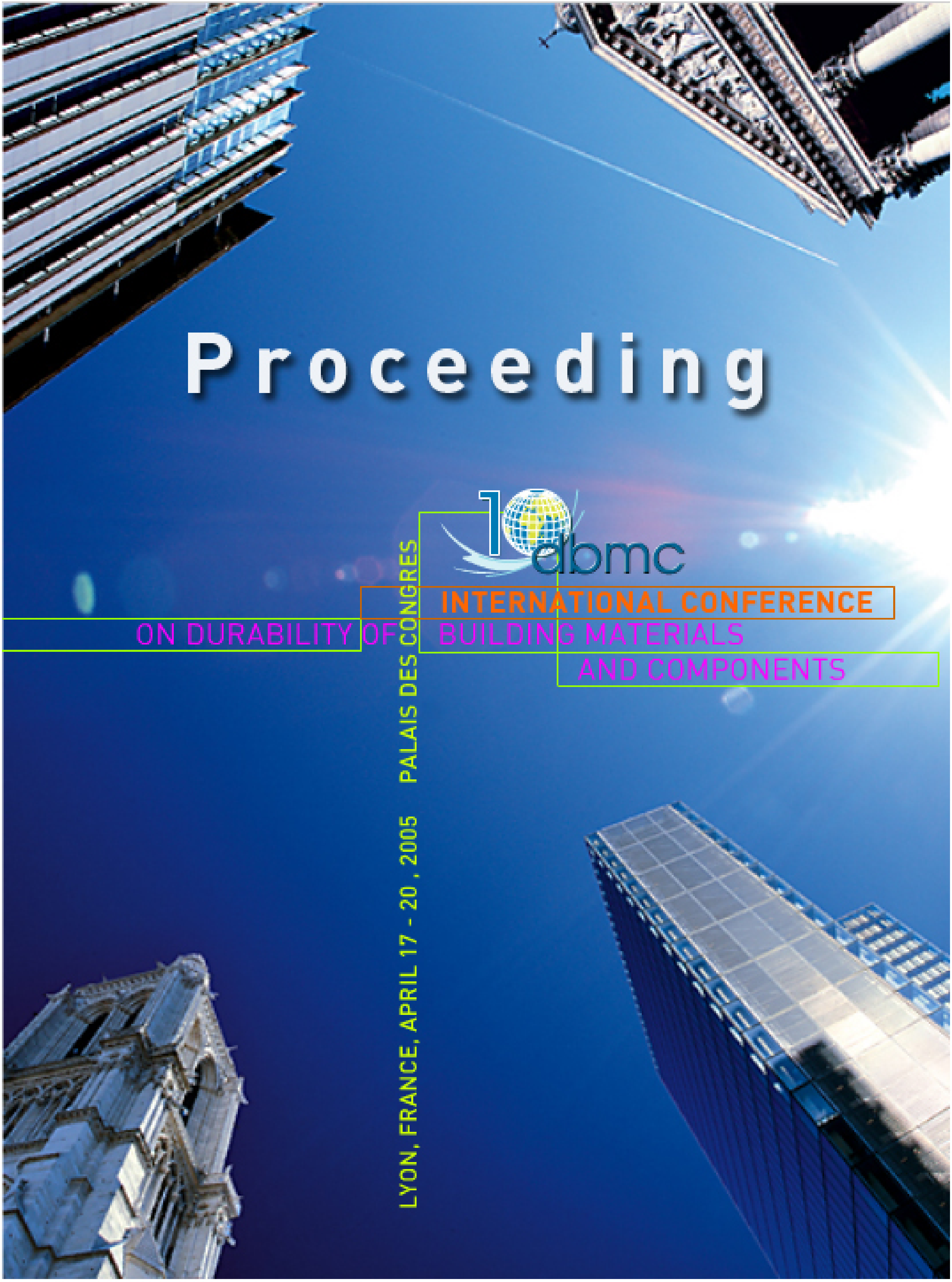
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Modelling of Corrosion induced Failure of Reinforced Concrete Structures

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ABSTRACT

Many reinforced concrete (RC) structures are suffering from corrosion related problems much earlier than their expected service life.
Steel reinforcement is subjected to corrosion due to carbonation and chloride attack. Carbonation ordinarily induces uniform corrosion. Chloride attack induces generally localised corrosion at cracks level. The consequences of the corrosion at local scale are a decrease of sound steel active cross section, a development of cracking in concrete cover, and a loss of steel-concrete bond.
To carry out structural assessment of corroded structure elements needs to evaluate the consequences of the corrosion at global scale. These consequences depend on the sort of corrosion, the corrosion rate, the localisation and the extension of corroded areas, and the crack density.
A Finite Element model has been developed to simulate the flexural behaviour of corroded RC elements. The FE model was calibrated with different experimental studies. For the computation, special elements were used to model the interface between steel and concrete. They are called rust elements. Their size is fixed and their characteristics infer from measurable parameters as corrosion localisation or rate. Their efficiency to compute flexural behaviour of corroded RC elements has been highlighted.
The FE model has been used to simulate different types of corrosion instance, varying nature, localisation, extension, rate, and cracking. For each case the ultimate load, the ultimate deflection, and the stiffness were computed. The final objective is to provide an efficient tool of evaluation of structural assessment and of failure prediction from deterioration assessment.

KEYWORDS

Reinforced Concrete, Degradation, Corrosion, Finite Elements, Failure

1 INTRODUCTION

Many structures are suffering from corrosion related problems much earlier than their expected service life. Reinforcing steel bars are subjected to corrosion due to carbonation and chloride attack. Carbonation ordinarily induces general corrosion whereas chloride attack induces localised corrosion at cracks level. The consequences of corrosion at local scale are a decrease of sound steel active cross section, a development of cracking in concrete cover, and a loss of steel-concrete bond. To carry out structural assessment of corroded structure elements needs to evaluate the consequences of the corrosion at global scale. These consequences depend on the type of corrosion, the degree of corrosion, the localisation and the extension of corroded areas, and the crack density.

A Finite Element study has been developed to simulate the flexural behaviour of corroded RC elements [Dekoster 2003]. The FE calculation was calibrated from different experimental studies [Dekoster *et al.* 2003, Castel *et al.* 2000, Lee *et al.* 1998]. For the computation special elements were used to represent the interface between steel and concrete. They are called “rust” elements. Their size was fixed and their characteristics were inferred from measured parameters representing the degradation of materials as degree and extension of corrosion. Their efficiency to compute flexural behaviour of corroded RC elements has been highlighted.

The “rust” elements were used to simulate various instances of corrosion, varying nature, localisation, extension, and degree. For each case the load, the deflection, and the stiffness were computed. The final objective is to provide an efficient tool of evaluation of global structural assessment from local deterioration assessment.

2 COMPUTATIONAL PROCEDURE

2.1 Reinforced concrete model

A damage elastic plastic model and volume elements are used to model the concrete. An elastic plastic model and bar elements are used to model the steel bars. The reduction of the steel cross section with the corrosion degree is supposed to be linear:

$$A_c = A*(1-\eta) \tag{1}$$

where A_c is the corroded steel cross section, A the original steel cross-section, and η the corrosion degree.
The computation is performed following the smeared crack approach. This choice induces that the model cannot take into account the longitudinal secondary cracking pattern generated by corrosion, and looses its accuracy near the bearings where the shear effects become predominant.

2.2 Rust element

The rust production x_r is considered to be a function of the corrosion penetration x, assuming that the increase of volume is two times the initial section (Equation 2). A parametric study [Dekoster 2003] had shown that the choice of this factor of expansion is not very relevant. Equation 3 gives the relationship between the rust production x_r, the original radius of reinforcement r_0, and the corrosion degree η (between 0 and 1).

$$x_r = 2x \tag{2}$$

$$x_r = 2r_0(1-\sqrt{1-\eta}) \tag{3}$$

In their model, [Molina *et al.* 1993] had simulated the corrosion simply with a linear variation of the materials properties from those of steel to those of rust. In our study [Dekoster 2003, Dekoster *et al.* 2003], rust and steel are considered as distinct materials and simply their section is changing. For the steel, the use of bar elements allows easily the decrease of the reinforcement section. To model the rust, volume elements (three nodes triangular) are used. The mechanical characteristics of the rust are assumed to be closed to these of water [Molina *et al.* 1993]. The real section of rust must be represented by fictitious layers of rust (Fig. 1).

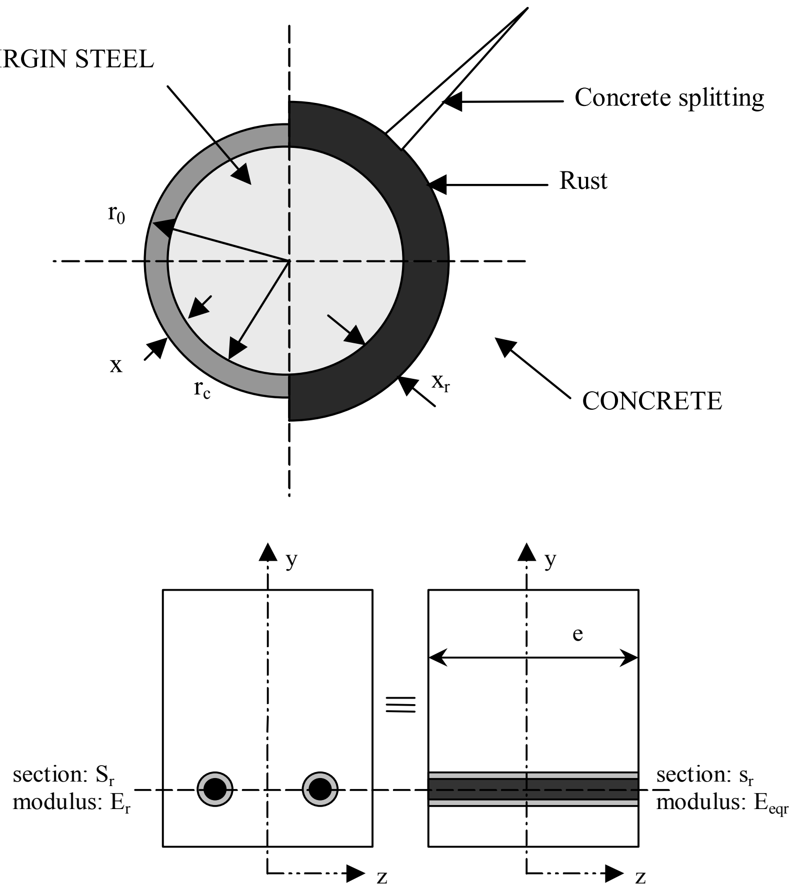

Figure 1. Real and reconstructed steel and rust section

As the corrosion is not homogeneously distributed along the reinforcement, the size of the rust element is defined with regard to the part of the beam with the lowest corrosion degree n_{min}. To fit the real conditions of corrosion and the needs of the modelling, an equivalent modulus E_{eqr} is used and is presented in equation 4. The real section of rust is presented in equation 5, the reconstructed section of rust in equation 6 and finally equation 7 give the relation between those parameters and the material and geometrical properties:

$$E_{eqr} = E_r \frac{S_r}{s_r} \tag{4}$$

$$S_r(\eta) = 2\pi[(r_0 + x)^2 - (r_0 - x)^2] = 8\pi r_0^2 (1 - \sqrt{1-\eta}) \tag{5}$$

$$s_r(\eta) = 2ex = 2r_0 e(1 - \sqrt{1-\eta_{min}}) \tag{6}$$

$$E_{eqr}(\eta) = E_r \frac{4\pi r_0}{e} \tag{7}$$

This use of a volume element rather than un-dimensional interface elements following stress-relative displacement laws is the main original aspect of this model. Only the knowledge of the corrosion degree needs to be correctly fitted.

2.3 Simulation parameters

The geometrical and mechanical parameters, which can be modified for simulation work, are presented in Fig. 2. For the present study, the same values as [Castel *et al.* 2000] are used and presented in Table 1.

Concrete								*Steel*		
l mm	a mm	e mm	d mm	h mm	E_b GPa	f_c MPa	f_t MPa	ϕ mm	E_s GPa	f_y MPa
2800	1400	150	258	280	36	65.3	6.8	12	250	500

Table 1. Geometrical and mechanical parameters

The corrosion instances are simulated by varying the location, the size, the degree of corrosion of the corroded zone, and the location of the loading point (Table 2 and Fig. 3).

Parameters	*Values*				
Size of the corroded zone (p_1) (mm)	100		300		500
Location of the loading point from the centre (p_2) (mm)	0		-400		-800
Location of the mid of the corroded zone from the centre (p_3)	0	-200	-400	-600	-800
Corrosion (%)	10			20	

Table 2. Parameters used for the simulation

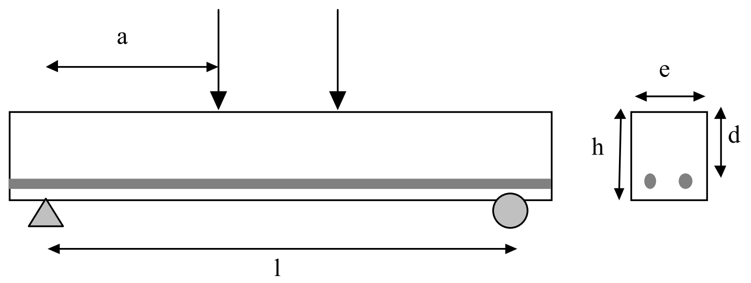

Figure 2. Beam dimensions

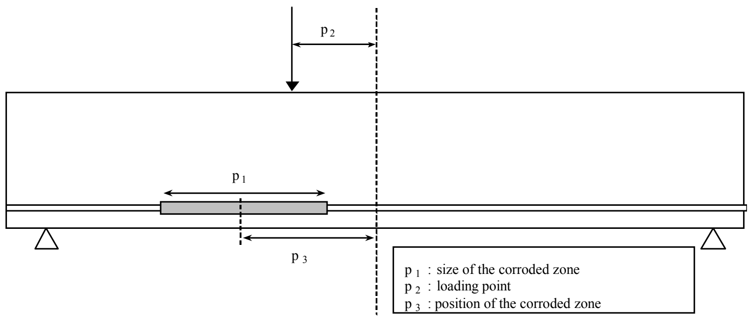

Figure 3. Presentation of the parameters used for the simulation

3 STUDY OF CORROSION INFLUENCE ON BEAM FLEXURAL BEHAVIOUR AT SERVICEABILITY CONDITIONS

For the selected geometric and mechanical values (Table 1,[Castel *et al.* 2000]), the service load given by the French code BAEL 91 is equal to 19.2 kN. In first, the beam presumed new is loaded until the service load F_{ser}. The service deflection (u_{ser}) is plotted at this point. Then, to take into account the various delayed deformations as shrinkage, swelling, fatigue, the beam is unloaded, corrosion is introduced by considering the degraded characteristics of materials and again the beam is loaded until service load [Dekoster *et al.* 2003-2].

Load decrease ratio							*Deflection increase ratio*						
		$p_2 = 0$							**$p_2 = 0$**				
		p_3							**p_3**				
p_1	**η**	**-800**	**-600**	**-400**	**-200**	**0**	**p_1**	**η**	**-800**	**-600**	**-400**	**-200**	**0**
100	**0.1**	*0.09*	*0.09*	*0.10*	*0.11*	*0.08*	**100**	**0.1**	*0.10*	*0.10*	*0.11*	*0.12*	*0.09*
	0.2	0.11	0.10	0.11	0.12	0.11		**0.2**	0.12	0.11	0.12	0.14	0.12
300	**0.1**	*0.13*	*0.12*	*0.13*	*0.12*	*0.11*	**300**	**0.1**	*0.15*	*0.14*	*0.15*	*0.14*	*0.13*
	0.2	0.14	0.14	0.14	0.15	0.16		**0.2**	0.16	0.17	0.17	0.17	0.19
500	**0.1**	*0.13*	*0.14*	*0.14*	*0.15*	***0.15***	**500**	**0.1**	*0.15*	*0.17*	*0.17*	*0.17*	***0.17***
	0.2	0.15	0.19	0.19	0.19	**0.20**		**0.2**	0.18	0.23	0.23	0.24	**0.25**
		$p_2 = -400$							**$p_2 = -400$**				
100	**0.1**	*0.10*	*0.11*	*0.10*	*0.11*	*0.10*	**100**	**0.1**	*0.11*	*0.12*	*0.11*	*0.12*	*0.11*
	0.2	0.11	0.12	0.11	0.12	0.11		**0.2**	0.12	0.13	0.12	0.14	0.12
300	**0.1**	*0.14*	*0.14*	*0.12*	*0.13*	*0.12*	**300**	**0.1**	*0.16*	*0.16*	*0.14*	*0.15*	*0.13*
	0.2	0.16	0.16	0.15	0.16	0.15		**0.2**	0.20	0.19	0.18	0.18	0.18
500	**0.1**	***0.17***	*0.15*	*0.15*	*0.13*	*0.13*	**500**	**0.1**	***0.21***	*0.17*	*0.18*	*0.16*	*0.14*
	0.2	**0.23**	0.20	0.20	0.19	0.18		**0.2**	**0.29**	0.25	0.25	0.23	0.23
		$p_2 = -800$							**$p_2 = -800$**				
100	**0.1**	*0.03*	*0.04*	*0.08*	*0.04*	*0.02*	**100**	**0.1**	*0.03*	*0.04*	*0.09*	*0.05*	*0.02*
	0.2	0.04	0.05	0.10	0.08	0.02		**0.2**	0.05	0.05	0.11	0.08	0.03
300	**0.1**	*0.10*	*0.10*	*0.15*	*0.14*	*0.05*	**300**	**0.1**	*0.11*	*0.10*	*0.18*	*0.16*	*0.06*
	0.2	0.14	0.15	0.18	0.17	0.06		**0.2**	0.16	0.18	0.23	0.20	0.08
500	**0.1**	*0.12*	*0.10*	***0.20***	*0.19*	*0.14*	**500**	**0.1**	*0.14*	*0.11*	***0.25***	*0.24*	*0.16*
	0.2	0.21	0.18	**0.24**	**0.24**	0.18		**0.2**	0.27	0.22	**0.31**	**0.31**	0.22

Table 3. Load decrease and Deflection increase ratios

The new deflection ($u_{ser\text{-}c}$) corresponding to the service load and the load ($F_{ser\text{-}c}$) corresponding to the previous service deflection are plotted. The results are presented as ratios by report to the sound beam values.

$$\text{Load decrease ratio} = \frac{F_{ser} - F_{ser-c}}{F_{ser}} \tag{8}$$

$$\text{Deflection increase ratio} = \frac{u_{ser-c} - u_{ser}}{u_{ser}} \tag{9}$$

The load decrease ratio is representing the loss of mechanical capacity of the structure, and at serviceability conditions is corresponding to the decrease of load for a given deflection linked to a critical opening of cracks. This decrease is related to the decrease of stiffness of the corroded structure. The deflection increase ratio is representing the consequence of the corrosion process in terms of deformation. The results presented in Table 3 show a similar evolution for load decrease and deflection increase but for a same simulation, the deflection increases higher than the load decreases.

The increase of the ratios is well related to the increase of the size of the corroded zone . But the maximal ratios are not systematically observed at the level of the application of the load, except at mid-span. For the point of application located at 400 mm, the maximal ratios are observed at the end of the corroded zone near the bearing. For the point of application located at 800 mm, the maximal ratios are observed at the middle between the point of application of the load and the mid-span.

4. STUDY OF CORROSION INFLUENCE ON BEAMS AT FLEXURE ULTIMATE STATE

In this case, the new beam is loaded until the service load (F_{ser}). Then, after the unloading of the beam, and the introduction of corroded characteristics, the beam is loaded until the ultimate state, plotting the ultimate deflection (u_{uc}) and the ultimate load (F_{uc}) of the corroded simulation. A loading of the sound beam at ultimate state has been simulated as a preliminary in order to determinate the ultimate deflection of the sound beam (u_{ut}) and its ultimate load (F_{ut}).The results are presented as ratio by report to the sound beam values.

$$\text{Ultimate load decrease ratio} = \frac{F_{ut} - F_{uc}}{F_{ut}} \tag{10}$$

$$\text{Ultimate deflection decrease ratio } = \frac{u_{ut} - u_{uc}}{u_{ut}} \tag{11}$$

The load decrease ratio is representing the loss of strength capacity of the corroded structure compared with the sound structure, and at ultimate conditions is corresponding to a decrease of safety factor. This decrease is mainly related to the reduction of the reinforcement steel cross section induced by corrosion. The deflection decrease ratio is representing the consequence of the corrosion process in terms of ductility. This decrease is related to a combination of the reduction of the steel cross section and the reduction of the stiffness of the corroded structure induced by the decrease of bond.

For the serviceability conditions, the ratios corresponding to a degree of corrosion of 0.2 are just a little higher than these corresponding to a degree of corrosion of 0.1. For the ultimate conditions, the difference is notably higher, sometimes of two or three times.

It can be observed in Table 4 that the evolutions of the deflection and load ratios are different from these corresponding to the serviceability conditions. First, for the simulation with 100 mm size of corroded zone, the ultimate deflection is strongly more reduced than the ultimate load. For the other sizes (300 and 500 mm), the reductions are quantitatively the same than for 100 mm size, but deflection reduction is located at the ends of the corroded zone, while load reduction remains located at the loading point [Dekoster 2003].

For the ultimate load and for all the sizes of corroded zone, the minimal ratio is observed at the loading point. The difference with the variation of the loading point is not very high.

For the ultimate deflection, the ratio decreases when the size of the corroded zone increases.

Those results confirm that deflection and load are not influenced in the same way by the loss of bond and the loss of steel cross section. It is principally the loss of steel cross section that affects the load reduction, while it is the loss of bond that affects the deflection reduction.

Table 4. Ultimate load and Ultimate deflection ratios

Ultimate load decrease ratio							*Ultimate deflection decrease ratio*						
		$p_2 = 0$							$p_2 = 0$				
		p_3							p_3				
p_1	η	-800	-600	-400	-200	0	p_1	η	-800	-600	-400	-200	0
100	0.1	0.00	0.00	0.02	0.06	0.11	100	0.1	0.00	0.00	0.09	0.18	0.55
	0.2	0.01	0.04	0.08	0.12	0.19		0.2	0.00	0.15	0.24	0.55	0.59
300	0.1	0.00	0.03	0.03	0.07	0.08	300	0.1	0.00	0.20	0.22	0.37	0.24
	0.2	0.01	0.05	0.05	0.18	0.19		0.2	0.00	0.23	0.24	0.58	0.30
500	0.1	0.02	0.02	0.03	0.11	0.10	500	0.1	0.04	0.04	0.21	0.35	0.11
	0.2	0.03	0.04	0.12	0.18	0.19		0.2	0.23	0.25	0.48	0.58	0.20
		$p_2 = -400$							$p_2 = -400$				
100	0.1	0.01	0.04	0.09	0.05	0.01	100	0.1	0.00	0.22	0.46	0.24	0.00
	0.2	0.03	0.11	0.22	0.13	0.06		0.2	0.02	0.46	0.60	0.53	0.18
300	0.1	0.03	0.07	0.10	0.08	0.03	300	0.1	0.03	0.41	0.14	0.40	0.06
	0.2	0.02	0.16	0.19	0.17	0.08		0.2	0.00	0.54	0.30	0.50	0.26
500	0.1	0.02	0.10	0.12	0.10	0.06	500	0.1	0.11	0.36	0.11	0.25	0.23
	0.2	0.05	0.18	0.19	0.17	0.13		0.2	0.35	0.44	0.19	0.50	0.43
		$p_2 = -800$							$p_2 = -800$				
100	0.1	0.08	0.03	0.00	0.00	0.00	100	0.1	0.38	0.13	0.00	0.00	0.00
	0.2	0.08	0.13	0.03	0.00	0.00		0.2	0.57	0.45	0.09	0.00	0.00
300	0.1	0.09	0.07	0.03	0.00	0.00	300	0.1	0.21	0.17	0.00	0.00	0.00
	0.2	0.17	0.16	0.13	0.00	0.00		0.2	0.29	0.31	0.13	0.00	0.00
500	0.1	0.08	0.08	0.03	0.00	0.00	500	0.1	0.14	0.02	0.13	0.00	0.00
	0.2	0.18	0.17	0.13	0.00	0.00		0.2	0.32	0.05	0.32	0.00	0.00

5 CONCLUSIONS

The developed tool of simulation presented in this study enables the prediction of the mechanical behaviour of corroded RC structures. It is possible to take into account the location, the expanse and the degree of corrosion of the corroded zone to simulate the behaviour at serviceability and ultimate conditions.

At serviceability conditions, the increase of the size and of the degree of corrosion of the corroded zone induces the decrease of the load ratio and the increase of the deflection ratio. The reduction of the bond between steel and concrete is the main factor of mechanical degradation for these conditions.

At ultimate conditions, the both load and deflection are reduced. It is principally the loss of steel cross section that affects the ultimate load decrease, and the loss of bond that affects the ultimate deflection decrease. This is confirmed by the fact that the main deflection decreases are observed at the end of the corroded zone.

The model presents a set of limitations:
- the corrosion is graduated by the degree of corrosion, which cannot be measured directly in situ;
- the use of the smeared crack approach gives greater place to the bending cracking at the expense of shear and secondary cracking;
- the model is fitted to global scale analysis and is not suited to local scale analysis;
- the model is not very accurate near the bearings where the shear effect becomes predominant.

But the model allows taking into account the degradation of the bond with the corrosion by using only one parameter, unlike other models as analytical ones, which need three or more parameters to characterise the bond or the tension-stiffening degradation [Buyle-Bodin & Rezaie 2000, Castel *et al.* 2002].

The model will be used soon in a French large benchmark leaning on the study of 40 years old control beams of French sea water dam "Barrage de la Rance" near Saint Malo. It will be in competition with analytical and FE models.

6 REFERENCES

Buyle-Bodin, F. & Rezaie, F. 2000, 'Modelling at global scale of mechanical effects of the corrosion of the steel reinforcement of RC beams', Life prediction and aging management of concrete structures, D.J. Naus ed., RILEM Proceedings 16, 43-52.

Castel, A., François, R. & Arliguie, G. 2000, 'Mechanical Behaviour of Corroded Reinforced Concrete Beams – Part 1: Experimental Study of Corroded Beams', *Materials and Structures*, **33**, 539-544.

Castel, A., François, R. & Arliguie, G. 2002, 'Mechanical modelling of reinforced concrete damaged by steel corrosion', *Comptes rendus de l'Académie des Sciences, serie IIb/mechanics*, 330, 45-50.

Dekoster, M. 2003, 'Etude du comportement mécanique des structures en béton armé dégradé par la corrosion', Thèse de Doctorat, Université de Lille 1.

Dekoster, M., Buyle Bodin, F., Maurel, O. & Delmas ,Y. 2003, 'Modelling of the flexural behaviour of RC beams subjected to localised and uniform corrosion', *Engineering Structures,* **25**[10], 1333-1341.

Dekoster, M., Buyle Bodin, F. & Maurel, O. 2003-2, 'Structural assessment of reinforced concrete elements subjected to localised and general corrosion', Life prediction and aging management of concrete structures, D.J. Naus ed., RILEM Proceedings 29, 115-126.

Lee, H.S., Tomosawa, F. & Noguchi, T. 1998, 'Fundamental Study on Evaluation of Structural Performance of Reinforced Concrete Beam Damaged by Corrosion of Longitudinal Tensile Main Rebar by Finite Element Method', *Journal of Structural and Construction Engineering,*, **AIJ**[506], 43-50.

Molina, F.J., Alonso, C. & Andrade C. 1993, 'Cover cracking as a function of rebar corrosion : Part 2 Numerical model', *Materials and Structures*, **26**, 532-548.

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A prediction model for concrete carbonation based on coupled CO_2-H_2O-ions transfers and chemical reactions

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ABSTRACT

It is a recognized fact that steel corrosion reduces the serviceability and safety performance of reinforced concrete. Usually high alkaline conditions in concrete lead to the formation of a passive layer at the steel surface. However the natural diffusion of the atmospheric carbon dioxide (CO_2) into the concrete induces a decrease of the pore water *pH* value after reactions with hydrates such as portlandite $Ca(OH)_2$ and calcium silicate hydrates C-S-H. Under low-*pH* conditions, the passive layer breaks down allowing the corrosion to start. Because the carbonation products (calcium carbonates $CaCO_3$) have higher molar volumes than the hydrates, there is a significant decrease of porosity during carbonation. Moreover, the carbonation releases water previously combined to the hydrates. In this research, a model accounting for CO_2-H_2O-ions transfers, carbonation reactions, *pH* fall, porosity reduction and water mass supply is developed.

The authors measure by mercury intrusion the drop of porosity after carbonation of Portland cement pastes with water-cement W/C ratios ranging from 0.35 to 0.6. By using thermal analysis, they correlate these results to the quantity of carbonated hydrates. Analytical expressions used in modelling are proposed to link the decrease of porosity due to carbonation with the amounts of consumed hydrates.

To simulate carbonation phenomena in concrete, dissolution-dissociation of CO_2 in pore water, dissolution of $Ca(OH)_2$ and precipitation of $CaCO_3$ are formulated in the framework of chemical equilibrium and kinetic laws. A macroscopic kinetic of C-S-H carbonation is proposed following a topotactic transformation. These chemical reactions are introduced in the mass balance equations as source or sink terms. The constitutive laws of transfer are diffusion of CO_2, electro-diffusion of species in solution and convection of liquid water. The capillary pressure is directly linked to the liquid phase (due to the low value of the gaseous phase pressure with respect to the liquid phase one) and the moisture transport is assumed to be controlled by the sole liquid phase, the contribution of the vapour transfer being neglected. We consider that carbonation leads to variations of porosity, as well as of water content, and consequently to modifications of transfer properties. 1D boundary value problems are worked out by a finite volume method. Unknown parameters of chemical kinetics are calibrated with experimental results of accelerated carbonation of a cement paste. Then, the model is validated with results of accelerated carbonation of a concrete. The model is thus able to reasonably predict the carbonation phenomena and *pH* profiles in concrete for accelerated conditions.

KEYWORDS

Carbonation - Cement based materials - Durability - Mass transport - Modelling

1 INTRODUCTION

In this paper, we introduce a generalized computational method which can deal with *pH* fluctuations, CO_2-H_2O-ions transfers, evolution of the microstructure and liquid water formation due to carbonation. The originality of the model developed consists firstly on the description of the chemical reactions in the framework of thermodynamic equilibriums or chemical kinetics, secondly on the prediction of *pH* and finally on the analytical functions introduced for linking the decrease of porosity and the supply of water to the chemical composition of the material. Moreover, following the performance approach proposed by Baroghel-Bouny *et al.* [2004], the input data of the model correspond to physico-chemical durability indicators identified as key-parameters for the carbonation process. The aim of the paper is mainly to model accelerated carbonation of concrete. The extension to other external conditions is easy due to the characteristics of the governing equations.

In the first part of this paper the physico-chemical mechanism of carbonation is detailed. Secondly, we focus on the modelling of CO_2-H_2O-ions transfers which are coupled to chemical reactions through mass balance equations. Eventually, accelerated carbonation tests on a cement paste are simulated to calibrate the model. To validate it, output data are compared with experimental results obtained after accelerated carbonation of a concrete: profile of carbonation, depth of carbonation detected by phenolphthalein pulverization, etc., so-called monitoring parameters.

2 PHYSICO-CHEMICAL MECANISM OF CARBONATION

2.1 Modelling of carbonation reaction

The overall process of lime carbonation consists of six steps described in table 1. Reaction **(a)** corresponds to CO_2 dissolution and reactions **(b)** and **(c)** are linked to the dissociation of H_2CO_3 consuming OH^- ions and decreasing the *pH* value of the pore solution. Dissolution of $Ca(OH)_2$ **(d)** enables to maintain a high *pH* around 12.4 in the pore solution. Then, calcium ions resulting from the dissolution of $Ca(OH)_2$ are assumed to precipitate with carbonate ions to form $CaCO_3$ **(e)**. While steps **(a)**, **(c)**, **(e)** and **(f)** are instantaneous, the rate controlling steps are the dissolution of $Ca(OH)_2$ **(d)** and dissociation of H_2CO_3 **(b)**. Consequently, reactions **(a)**, **(c)**, **(e)** and **(f)** are considered at equilibrium whereas **(b)** and **(d)** have an explicit kinetic law. The dissociation kinetic of H_2CO_3 **(b)** is given by Danckwerts [1970]:

$$\xi_1 = \phi S k_1 \left([H_2CO_3][OH^-] - \frac{[HCO_3^-]}{K_1} \right) \tag{1}$$

The reaction rate refers generally to an unit volume of the liquid phase of pores which are partially filled with water and air. So, it must be multiplied by the volume ϕS of liquid per unit volume of concrete (ϕ is the porosity accessible to water and S the saturation degree). k_1 is the rate constant which has to be calibrated.

Reactions	*Equilibrium equations*	*Equilibrium constants*	*Reaction rates*
(a) $CO_2(g) + H_2O \leftrightarrow H_2CO_3$	$K_H = [H_2CO_3]/[CO_2]$	$K_H = 0{,}94$	ξ_H
(b) $H_2CO_3 + OH^- \leftrightarrow HCO_3^- + H_2O$	$K_1 = \frac{[HCO_3^-]}{[H_2CO_3][OH^-]}$	$\log(K_1) = 7{,}66$	ξ_1
(c) $HCO_3^- + OH^- \leftrightarrow CO_3^{2-} + H_2O$	$K_2 = \frac{[CO_3^{2-}]}{[HCO_3^-][OH^-]}$	$\log(K_2) = 3{,}66$	ξ_2
(d) $Ca(OH)_2 \leftrightarrow Ca^{2+} + 2OH^-$	$K_P = [Ca^{2+}][OH^-]^2$	$\log(K_P) = -5{,}19$	ξ_P
(e) $Ca^{2+} + CO_3^{2-} \leftrightarrow CaCO_3$	$K_C = 1/[Ca^{2+}][CO_3^{2-}]$	$\log(K_C) = 8{,}36$	ξ_C
(f) $OH^- + H_3O^+ \leftrightarrow 2H_2O$	$K_W = [OH^-][H_3O^+]$	$\log(K_W) = -14$	ξ_W

Table 1. Mechanism of carbonation of $Ca(OH)_2$. The equilibrium constants are given at 298 K.

Groves *et al*. [1990] observe by transmission electron microscopy that the structure of $Ca(OH)_2$ is progressively surrounded by rims of $CaCO_3$ microcrystals forming a compact and tortuous coating through which ions transfer with difficulty [cf. Fig. 1]. It justifies that carbonation of $Ca(OH)_2$ is initially very rapid but, by the time roughly half of the original $Ca(OH)_2$ has reacted, it becomes much slower. By assuming that masses of $Ca(OH)_2$ crystals are spherical (initial radius R_0) [cf. Fig. 2], an analytical form for the dissolution rate of $Ca(OH)_2$ can be established [Thiery *et al.* 2004]. The following notations are adopted:

$$\kappa = 1 - \frac{n_{Ca(OH)_2}}{n^i_{Ca(OH)_2}} \qquad R(\kappa) = R_0(1-\kappa)^{1/3} \qquad R_c(\kappa) = R_0\left(1-\kappa+\frac{v_{CaCO_3}}{v_{Ca(OH)_2}}\kappa\right)^{1/3} \qquad s(\kappa) = \frac{3}{R_0} n^i_{Ca(OH)_2} v_{Ca(OH)_2}(1-\kappa)^{1/3}$$

$n^i_{Ca(OH)2}$ is the initial concentration of $Ca(OH)_2$ in moles per unit volume of concrete. κ is the carbonation rate of $Ca(OH)_2$, R the radius of the sphere of $Ca(OH)_2$ after carbonation, R_c the external radius of the coating of $CaCO_3$, v_k the molar volume of crystal k and s is the surface of $Ca(OH)_2$ per unit volume of concrete. It can be established that the rate of dissolution of $Ca(OH)_2$ is:

$$\xi_p = -\frac{dn_{Ca(OH)_2}}{dt} = \frac{s(\kappa)}{\frac{1}{h} + \frac{R(\kappa)}{R_c(\kappa)}\left(\frac{R_c(\kappa)-R(\kappa)}{D}\right)} \ln\left(\frac{K_p}{[OH^-]^2[Ca^{2+}]}\right)$$

The parameter D characterizes the diffusion of ionic species through the $CaCO_3$ coating and need to be calibrated and h=5,6.10^{-4} mol.m^{-2}.s^{-1} is a kinetic constant.

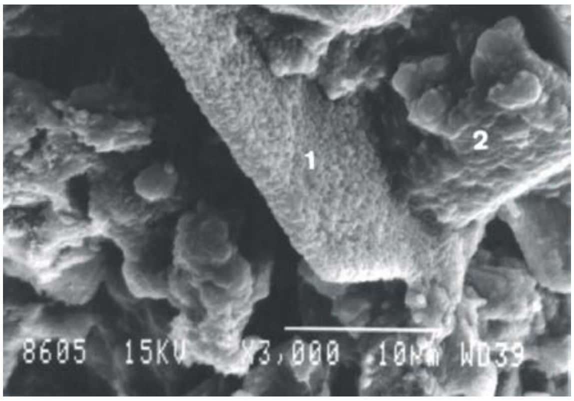

Figure 1. $Ca(OH)_2$ recovered with $CaCO_3$ (1), carbonation of C-S-H (2) [Rafai *et al.* 2002].

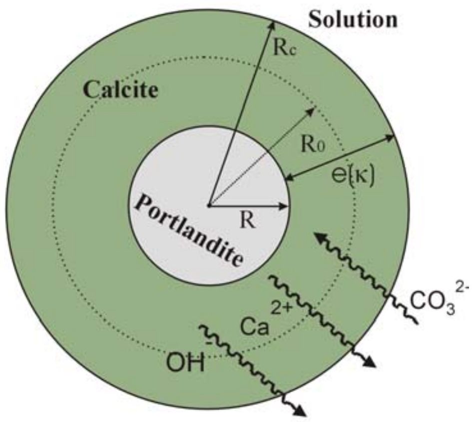

Figure 2. A spherical mass of $Ca(OH)_2$ surrounded by a $CaCO_3$ coating.

Concrete contains other phases: principally calcium silicate hydrates C-S-H. CO_2 also decomposes the C-S-H components producing $CaCO_3$ and a residual silica-gel [cf. Fig. 1]. The reaction of carbonation of C-S-H does not affect the alkalinity of the pore solution since the solubility of C-S-H is low compared with that of $Ca(OH)_2$, but C-S-H hydrates fix lots of CO_2 molecules. Consequently this reaction cannot be neglected, but a detailed descriptive analysis of the mechanism of carbonation of C-S-H is not required. We adopt the following reaction rate where τ_{CSH}=3000 s [Thiery *et al*. 2004].

$$\xi_{CSH} = -\frac{dn_{CSH}}{dt} = \phi S \frac{[CO_2]}{\tau_{CSH}} \qquad (3)$$

2.2 Effect of carbonation on the microstructure and water content

The $CaCO_3$ formation contributes to the clogging of the pore system, causing a decrease of porosity [Thiery et *al.* 2003a] and thus of effective diffusion coefficient of CO_2 and of ions. We consider changes in porosity with carbonation using a simplified model. Regarding the reduction of porosity $\Delta\phi$ as a function of amounts of carbonated $Ca(OH)_2$ and C-S-H, we consider the following equation:

$$\Delta\phi = v_{Ca(OH)_2}(n^i_{Ca(OH)_2} - n_{Ca(OH)_2}) - v_{CaCO_3} n_{CaCO_3} + \Delta v_{CSH}(n^i_{CSH} - n_{CSH}) \tag{4}$$

n_{CaCO3} corresponds to the quantity of calcium carbonates formed after $Ca(OH)_2$ carbonation. The molar volume of $Ca(OH)_2$ (33 $cm^3.mol^{-1}$) and of $CaCO_3$ (37 $cm^3.mol^{-1}$) are clearly identified in the literature, whereas the volume change after carbonation of C-S-H hydrates Δv_{CSH} is uncertain. We propose to determine it experimentally. For cement pastes of various W/C ratios, the porosity drop is measured after 40 days of accelerated carbonation of crushings (50% CO_2 and RH=53%). The materials are prepared with an ordinary Portland cement. Results are shown in Fig. 3. Moreover, we quantify by thermogravimetric analysis (TGA) all the $CaCO_3$ produced by carbonation of $Ca(OH)_2$ and C-S-H and, at the same time, by determining by TGA the content of $Ca(OH)_2$ which has disappeared, it is possible to evaluate the quantity of $CaCO_3$ associated to the C-S-H carbonation. By assuming that the C-S-H stoichiometry is $C_3S_2H_3$, we deduce that each C-S-H can fixe 3 molecules of CO_2. Relation (4) enables to calculate the decrease of porosity with a given value of Δv_{CSH}. We choose the best value of Δv_{CSH} which minimizes the discrepancy between experimental measurements and computed assessments of $\Delta\phi$ for all cement pastes [cf. Fig. 3]: we find Δv_{CSH}=39 $cm^3.mol^{-1}$.

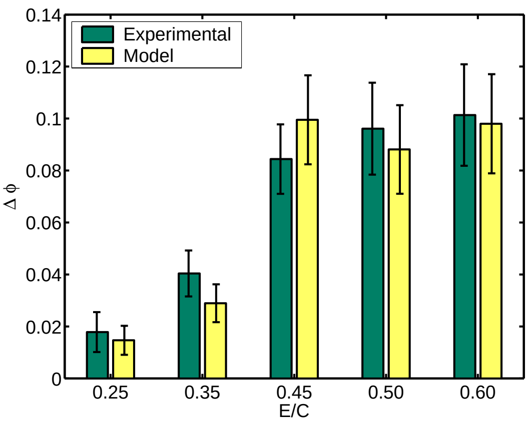

Figure 3. Drop of porosity experimentally estimated by mercury intrusion and theoretically calculated. Results obtained after 40 days of accelerated carbonation of cement pastes of various W/C ratios.

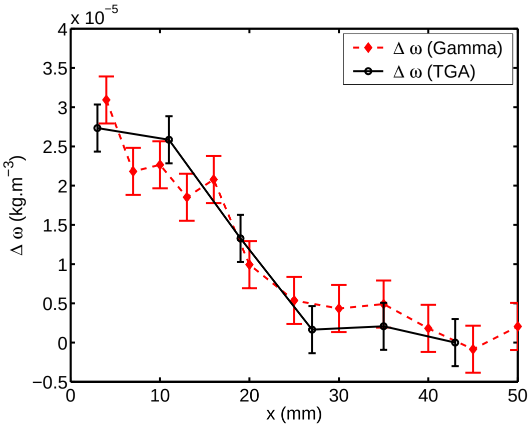

Figure 4. γ-ray measurements of water content increase following 14 days of accelerated carbonation of a concrete specimen. Comparison with the water only freed by carbonation of $Ca(OH)_2$ (TGA)

In the same way, we try to link experimentally the water content variation $\Delta\omega$ due to carbonation to the amount of hydrates carbonated. By γ-ray a depth-profile of $\Delta\omega$ is determined on a carbonated concrete specimen (50% CO_2 and RH=53%) [cf. Fig 4]. We estimate the increase of water content due to the sole carbonation of $Ca(OH)_2$ measured by TGA on sawed slices. It seems that mass supply of water is essentially linked to carbonation of $Ca(OH)_2$. Actually, $Ca(OH)_2$ carbonation corresponds with steps of dissolution, movements of ions in the pore solution and precipitation of carbonates carbonation, whereas carbonation of C-S-H follows a topotactic process without mass transport.

3 GOVERNING EQUATIONS OF TRANSPORT AND MASS BALANCE EQUATIONS

The diffusive flux of CO_2 is given by the following equation:

$$\mathrm{w}_{CO_2} = -D_g(\phi, S)\,\mathrm{grad}\,[CO_2] \quad \text{with} \quad D_g(\phi, S) = D_{g0} f(\phi, S) \tag{5}$$

D_{g0} is the diffusion coefficient of CO_2 in the air and $f(\phi,S)$ is the resistance factor describing the variation of space offered to the gaseous constituents and the tortuosity effects. Numerous expressions have been proposed by researchers for the reduction factor f; we retain the form derived from Millington's works [1959]: $f(\phi,S)=\phi^a(1\text{-}S)^b$. Thanks to measurements of diffusion coefficients of gas O_2

and CO_2 in mortars [Papadakis *et al*. 1991], we fit parameters a and b: a=2,74 et b=4,20 [Thiery *et al.* 2003b].

The velocity of liquid water under the effect of capillary suction is expressed *via* the Darcy law:

$$\mathrm{V_l} = -\frac{K(\phi)}{\mu}k(S)\mathrm{grad}\,p_l \quad \text{with} \quad \begin{cases} k(S) = \sqrt{S}[1-(1-S^{1/m})^m]^2 \\ K = K_i\,(\phi/\phi_i)^3\,[(1-\phi_i)/(1-\phi)]^2 \end{cases} \tag{6}$$

where p_l and $K(\phi)$ are the pressure of the liquid phase and the intrinsic permeability coefficient; μ denotes the dynamic viscosity of the water. The macroscopic function $k(S)$ affects K by accounting for variations of saturation degree due to moisture transfer, supply of water by carbonation and porosity changes. The form of $k(S)$ is based on a formulation proposed by Van Genuchten [1980]. m is a scalar, set to 0.45 for numerical applications. We assume that $K(\phi)$ depends on the total porosity ϕ [Van Genuchten 1980]. ϕ_i and K_i are respectively the initial porosity and intrinsic permeability coefficient of the noncarbonated material. Supposing that the pressure of the gaseous phase is negligible with respect to the liquid one, we have p_{cap}=-p_l where p_{cap} is the capillary pressure which is given as a function of saturation rate S by the desorption isotherm, characteristic of the cement based material. We assume that the moisture transport is controlled by the sole liquid phase, the contribution of the vapour transfer being neglected, as recently proposed by Mainguy [Baroghel-Bouny *et al*. 1999].

The flux of ions in the pore solution is given by the extended Nernst-Planck-Einstein relation with an advection term for the constituent i in solution:

$$\begin{aligned} &\mathrm{w_i} = -D_i(\phi,S)\,\mathrm{grad}\,[i] - \frac{z_iF}{RT}D_i(\phi,S)[i]\,\mathrm{grad}\,\Psi + [i]\mathrm{V_l} \\ &\text{with} \quad D_i(\phi,S) = 2.9\times10^{-4}D_{i0}\left[e^{9{,}95\phi}\right]/\left[1+625(1-S)^4\right] \end{aligned} \tag{7}$$

where D_i is the diffusion coefficient at the macro level through the porous medium and D_{i0} at the micro scale in solution. D_i depends both on ϕ and S *via* a relation proposed by Bary et Sellier [2004]. z_i is the valence number of the species, F is the Faraday constant, R the ideal gas constant, T the temperature and Ψ the electrical potential which is chosen to ensure that the electrolytic solution remains neutral, i.e. $F\sum z_i[i]=0$. Actually, it is possible to show that if the system is initially neutral the electro-neutrality is accomplished by enforcing zero total courant $F\sum z_i w_i=0$ [Thiery et al. 2004].

The mass balance equation for each constituent i takes the general form:

$$\frac{\partial n_i}{\partial t} = -\mathrm{div}\,\mathrm{w_i} + \sum_j \upsilon_{i,j}\xi_j \tag{8}$$

with n_i as the content of i per unit volume of material. $\upsilon_{i,j}$ is the algebraic stoichiometric coefficient of constituent i in chemical equation j. $\upsilon_{i,j}<0$ (resp. >0) if i is a reactive (resp. a product). $\upsilon_{i,j}=0$ if i does not appear in j. The numerical method used to solve the carbonation problem in unsaturated cement based materials is detailed by Thiery *et al*. [2003b & 2004].

5 SIMULATIONS OF ACCELARATED CARBONATION TESTS

A first accelerated carbonation test is performed on a W/C=0.45 cement paste (cement CEM I 52.5). After demoulding, cylindrical samples are wrapped in two superimposed adhesive aluminium foil sheets to guarantee a suitable water-tightness and are stored (T=20°C and RH=95%) during 8 months to ensure complete hydration prior to exposure to carbonation. After this curing, samples are wet-sawn by half, leaving a side face exposed to unidirectional drying and carbonation. The program continues with a 1 month drying at 45°C and a 8 months storing into sealed cells in which RH is controlled (RH=53%) and T=20°C. One considers that the distribution of water in the specimen is relatively homogeneous after such a pretreatment. Then the accelerated carbonation takes place during 14 days in a gas chamber where a continuous flow of a 50% CO_2 - 50% air mixture is imposed and where RH=53% and T=20°C are maintained. After 14 days of accelerated carbonation, the test is interrupted; a specimen is split to measure the carbonation depth by spraying a pH-indicator (phenolphthalein) whose colour changes at pH=9. Another specimen is sawn into 2-3 mm slices parallel to the CO_2-exposed surface and a TGA apparatus is used to quantify the solid concentration of $Ca(OH)_2$ and $CaCO_3$ in each slice.

The initial characteristics of the material used are given in table 2. The initial permeability coefficient K_i is estimated by the Katz-Thompson theory [Thompson & Katz 1987]. The initial hydric state after pretreatment is determined with the desorption curve (linking S to RH) by assuming that specimens have attained moisture equilibrium at RH=53%. The desorption curve is taken from results obtained on a cement paste of same W/C but containing a different Portland cement [Baroghel-Bouny *et al.* 1999]. The average radius R_0 of masses of $Ca(OH)_2$ is linked to W/C ratios by Chaussadent *et al.* [2000]. The content of C-S-H is calculated by assuming $C_3S_2H_3$ as stoichiometry of C-S-H and a complete hydration of anhydrous calcium silicates C_3S and C_2S.

The parameter D which characterizes limitation of accessibility of $Ca(OH)_2$ encased in carbonates crystals is fitted *via* the depth-profile of $Ca(OH)_2$ measured by TGA after 14 days of accelerated carbonation [cf. Fig. 5]. We find $D \approx 10^{-13}$ mol.m^{-1}.s^{-1}. The value of the rate constant k_1 of dissociation of H_2CO_3 is calibrated so that the carbonation depth X_C, experimentally determined with phenolphthalein, corresponds to the abscissa for which the model predicts a pH value of 9 [cf. Fig. 6]. We obtain k_1=150 mol^{-1}.L.s^{-1} while Danckwerts proposes k_1=6000 mol^{-1}.L.s^{-1}. This discrepancy is justified by the fact that this literature value has been measured in bulk solutions with dimensions larger than the dimensions of pores in a cement based material (a few nanometers). Furthermore, the saturation rate of the studied cement paste is relatively low after drying and the aqueous film thickness on the walls of larger pores walls is typically 1-3 molecular dimensions. In such aqueous environment, mass transport of ions at the micro scale follows different mechanisms than in bulk aqueous media, in particular when one takes into account the strong ion-pore wall interactions.

Initial durability indicator	*Measurement method*	*Cement Paste E/C=0.45*	*Concrete E/C=0.84*
$n^i_{Ca(OH)_2}$	TGA	5.6 mol.L^{-1}	1.35 mol.L^{-1}
ϕ	Porosity accessible to water	0.40	0.15
K_i	Katz-Thompson method [Thompson & Katz 1987]	2.10^{-19} m^2	1.10^{-19} m^2
S (hydric state)	-	0.49 Desorption curve [S=F(HR)]	Profile of S γ-ray measurement
n_{CSH}	Hydration model	2.4 mol.L^{-1}	0.43 mol.L^{-1}
R_0	Electron microscopy [Chaussadent *et al.* 2000]	30 µm	60 µm
Desorption curve	-	[Baroghel-Bouny *et al.* 1999]	[Thiery *et al.* 2004]

Table 2. Initial durability indicators used as input data for simulations.

Figure 6 shows the curve of the predicted depth of carbonation X_C as function of square root of time. We notice a good agreement between experiments and simulations after 7 days and 28 days of accelerated carbonation. We remark that a certain time (around 4 days) is necessary to sufficiently carbonate the specimen's surface, i.e. to make pH drop under 9. This phenomenon is directly linked to the effect of the kinetics of the chemical reactions. Actually, if the chemical reactions were at equilibrium, the pH would have instantaneously decrease under 9.

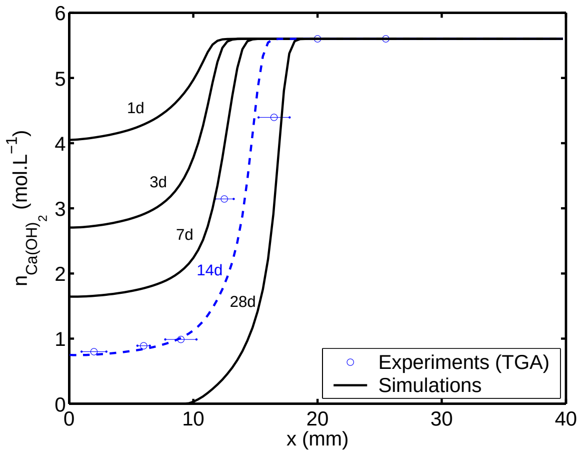

Figure 5. Depth-profiles of $Ca(OH)_2$ on the cement paste *W/C*=0.45.

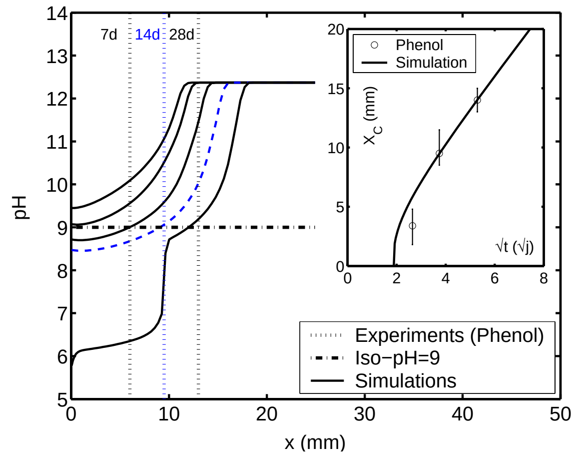

Figure 6. Depth-profiles of *pH* on the cement paste E/C=0.45.

As an example of validation, we have simulated an accelerated carbonation test on a concrete of W/C=0.84. The cement is the same as the one used for the cement paste. The initial characteristics are given in table 2. The desorption curve is exposed by Baroghel-Bouny in [Thiery *et al*. 2004]. The material is pretreated with an oven-drying at 45°C during 1 month. Since it is difficult to obtain a hydric equilibrium in a reasonable time, we prefer to measure the initial hydric profile by the gamma-ray transmission method and use it as an input data in the model. The simulated depth-profile of $Ca(OH)_2$ is shown on Fig. 7 and is consistent with TGA measurements. The depth of carbonation indicated by spraying phenolphthalein is compared with the calculated concrete depth X_C on Fig. 8. We note a good agreement. The study is completed with an analysis to investigate the sensitivity of the model to variations in material parameters. We numerically identify key parameters for the carbonation process: the $Ca(OH)_2$ content, the porosity, the permeability coefficient and the initial hydric state. They are consistent with the relevant indicators identified by Baroghel-Bouny [Baroghel-Bouny *et al.* 2004]. Small variations of these input data have a great incidence on output data such as depth-profiles of *pH* or $Ca(OH)_2$. Parameters are measured with a certain lack of precision. By taking into account the variability of these indicators, we show on Fig. 8 a curve which underestimates and another which overestimates the depth of carbonation. The experimental results, complemented with assessments of uncertainties, stay inscribed between the two curves which minimizes or maximizes X_C; this indicates the reliability of the developed model.

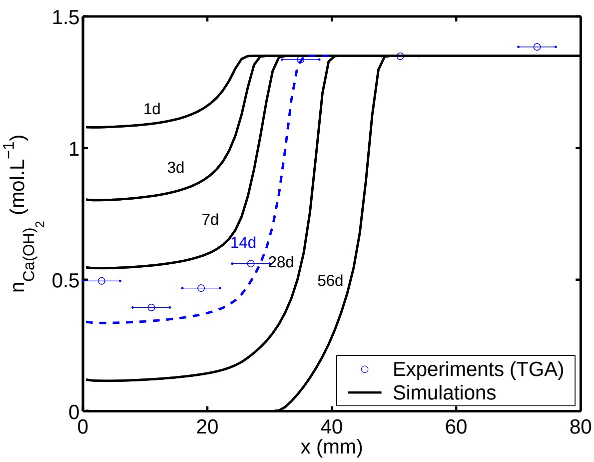

Figure 7. Depth-profiles of $Ca(OH)_2$ on the concrete *W/C*=0.84.

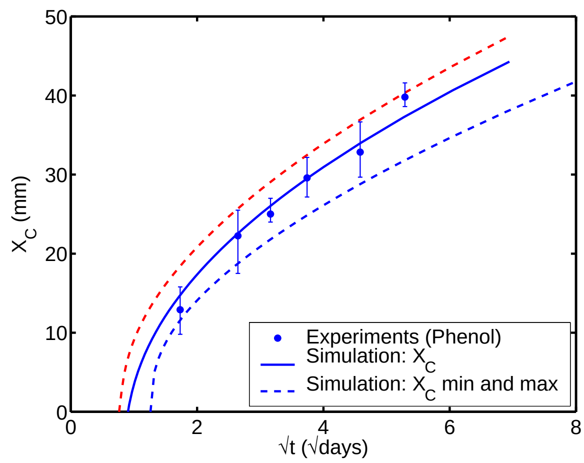

Figure 8. Depth of carbonation X_C as function of square root of time for the concrete *W/C*=0.84.

6 CONCLUSIONS

The approach for describing the carbonation of concrete takes into account CO_2, ions and water transfers. Chemical reactions are introduced using equilibriums or kinetics and constitute source or sink terms in mass balance equations. The diffusion coefficients and the water permeability are affected by the reduction of porosity and supply of water due to carbonation. The functions used for linking the porosity and the saturation degree variation to the chemical consumption of hydrates are fitted on experimental data.

In its current version, the proposed model gives interesting results, which permit to improve the global comprehension of the carbonation processes and constitutes a powerful prediction tool. It emphasizes the impact of kinetics of chemical reactions. Actually, it shows that neutralization does not occur immediately, even on the surface which is directly exposed to CO_2.

7 REFERENCES

Baroghel-Bouny, V., et al. 2004, 'Conception des bétons pour une durée de vie donnée des ouvrages - Maîtrise de la durabilité vis-à-vis de la corrosion des armatures et de l'alcali-réaction' (in french), Association Française du Génie Civil, Paris.

Baroghel-Bouny, V., Mainguy, M., Lassabatère, T. & Coussy, O. 1999, 'Characterization and identification of equilibrium and transfer moisture properties for ordinary and high-performance cementitious materials', Cement and Concrete Research, **29**, 1225-1238.

Bary, B. & Sellier, A. 2004, 'Coupled moisture-carbon dioxide-calcium transfer model for carbonation of concrete', Cement and Concrete Structure, **34**[10], 1859-1872.

Chaussadent, T., Baroghel-Bouny, V., Hornain, H., Rafaï, N. & Ammouche, A. 2000, 'Effect of water-cement ratio of cement pastes on microstructural characteristics related to carbonation process', Proc. of 5th CANMET/ACI Int. Conference on Durability of Concrete, Barcelone, 523-537.

Danckwerts, P.V. 1970, 'Gas-liquid reactions', Ed. McGraw-Hill Book Company, New York.

Van Genuchten, M.Th. 1990, 'A closed-form equation for predicting the hydraulic conductivity of unsaturated soils', Soil Sci. Soc. Am. J., **44**, 892-898.

Groves, G.W., Rodway, D.I. & Richardson I.G. 1990, 'The carbonation of hardened cement pastes', Advances in Cement Research, **3**[11], 117-125.

Lo, Y. & Lee, H.M. 2002, 'Curing effects on carbonation of concrete using phenolphtalein indicator and Fourier-transform infrared spectroscopy', Building and Environment, **37**, 507-514.

Millington, R.J. 1959, 'Gas diffusion in porous media', Science, **130**, 100-102.

Papadakis, V.G., Vayenas, C.G. & Fardis, M.N. 1991, 'Physical and chemical characteristics affecting the durability of concrete', ACI Materials Journal, **88**[2], 186-196.

Rafaï, N., Hornain, H., Villain, G., Baroghel-Bouny, V., Platret, G. & Chaussadent, T. 2002, 'Comparaison et validité des méthodes de mesure de la carbonatation' (in french), Revue Française du Génie Civil, **2**[6], 251-274.

Thiery, M., Villain, G. & Platret, G. 2003a, Effect of carbonation on density, microstructure and liquid water saturation of concrete, Proc. 9th Eng. Conf. on Advances in Cement and Concrete, Copper Mountain (USA), 481-490.

Thiery, M., Dangla, P., Villain, G. & Platret, G. 2003b, 'Modélisation du processus physico-chimique de carbonatation des bétons', Actes des Journées des Sciences de l'Ingénieur du réseau des LPC, december 9-11, Dourdan, France, 403-408.

Thiery, M., Dangla, P., Villain, G., Platret, G., Massieu, E., Druon, M. & Baroghel-Bouny, V. 2004, 'Atmospheric modelling of cement based materials', BLPC, **252**.

Thompson, A.H., Katz, A.J. & Krohn, C.E. 1987, 'The microgeometry and transport properties of sedimentary rocks', Adv. Phys., **36**[5], 625-693.

Impact of carbonation on microstructure and transport properties of concrete

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ABSTRACT

Physicochemical models of degradation of the reinforced concrete require in particular the microstructural and transport properties of cementitious materials as input data, in order to model the service life of engineering structures. However, the carbonation, which is one of the principal causes of deterioration, modifies the microstructure, decreases the porosity of the concrete containing Portland cement and consequently has a considerable influence on the gas and liquid transfers such as diffusion or permeation. These models should thus take into account the evolution of transport coefficients and with this intention, they need an experimental impact study.
The purpose of this paper is to study and quantify the influence of carbonation on the transport properties (i.e. the vapour diffusivity, the gas permeability and the liquid water permeability) of various concretes by explaining them by the microstructural modifications generated.
Several experimental campaigns are thus carried out on three concrete mixes, on carbonated concrete and non carbonated concrete as reference. Total bulk porosity is measured by water saturation, the pore structure estimated by mercury intrusion porosimetry and the gas permeability determined by Cembureau method. Electrical resistivity is recorded to obtain the concretes tortuosity and the liquid water permeability thanks to the Katz-Thompson theory and to the mercury intrusion porosimetry results. Moreover, for the most porous concrete, the vapour diffusion coefficient is measured at various relative humidities ranging from 90% to 64% in both carbonated and non carbonated cases.
Firstly, carbonation leads to flatten the concrete desorption isotherm curve. Furthermore carbonation involves an increase in the vapour diffusivity of the concrete what can be explained by the microstructural transformation.
Secondly, it is shown that the influence of carbonation on the gas and liquid water permeabilities is not clear: this influence can even cause a fall or a rise of permeability for the studied concretes. It is then supposed that this effect depends on the porous distribution of material and its alteration by carbonation. It is indeed observed that not only total porosity changes but also pore distribution. We assume that this is related to the size of the hydrates and the formed products after successive dissolution and precipitation processes. Consequently, these modifications of transport properties have an influence on the carbonation itself and on the wetting and drying cycles of structural concrete. Indeed, results show that a carbonated concrete dries more rapidly than a non carbonated concrete.

KEYWORDS

Carbonation, concrete, diffusion, permeability, tortuosity.

1 INTRODUCTION

In order to determine the impact of carbonation on transport properties of concrete, an experimental study is conducted on three different concretes. In the first part, the test methods are presented. In the second one, the results of drying and vapor diffusion experiments are shown and explained thanks to porosimetry curves obtained by mercury intrusion. In the third one, we comment on the measured oxygen permeability and the estimated liquid water permeability.

Vapor diffusivity rather than carbon dioxide diffusion coefficient is chosen because the measurement of the last one is not possible in the non carbonated concrete. And the future and ultimate aim of this research project is to evaluate this diffusion coefficient according to the water content of material for the carbonated concrete and the non carbonated concrete to try to estimate the carbonation impact on the carbon dioxide diffusion coefficient at different water content.

2 EXPERIMENTAL PROGRAMM

Three concrete mixes containing Portland cement and having average mechanical resistances at 28 days equal to 25, 40 and 50 MPa are tested [Table 1]. After a 3-month minimal cure under water, the samples of diameter equal to 11cm are sawn to obtain slices of different heights: 5cm for permeability and 5mm for diffusion measurements. The 5cm thick samples are then pre-dried during one month at 45°C in an oven and during one month at 20°C in a desiccator where relative humidity (RH) is equal to 53%. The 5mm thick samples of the most porous concrete (M25), set aside for carbonation, are directly put in the desiccator at 20°C and RH=53%. The half of the samples is totally carbonated in controlled conditions with carbon dioxide (CO_2) content equal to 50%, RH=53% and T=20°C. Meanwhile, the other half of the samples is maintained in the desiccator. Gamma-ray measurements enable to obtain the saturation rate of the bigger samples and to control if they are near an equilibrium state. Porosity by water saturation and porosimetry by mercury intrusion are measured.

		M25	M40	M50
Silico-calcareous sand (0/5) and gravel (4/20)	(kg/m^3)	1906	1898	1743
Cement CEM I 52,5 PM ES (Lafarge St Vigor)	(kg/m^3)	230	300	410
Total water	(kg/m^3)	193	187.4	197
Water-cement ratio W/C	(-)	0.84	0.62	0.48
Average compressive strength at 28 days	(MPa)	26.9	44.1	54.7

Table 1. Mix design of the studied materials

2.1 "Cupel" method to determine vapor diffusivity

The 5mm thick samples are kept in a desiccator above water until their mass remain constant to reach a saturated state. Then the desorption experiment is conducted following Baroghel-Bouny's recommendations [1994] but at 20°C. Step by step, the series of 4 samples are dryied at different relative humidities: 90% [Fig. 1], 75% and 64% and all the samples are regularly weighted. Thereafter, the desorption isotherm curve is plotted [Fig. 2]. This isotherm could be expressed in mass water content (w) by estimating the dried mass of the samples by porosity and density determined by water saturation of bigger samples.
To obtain the vapor diffusivity, "cupel" device is used [Delcelier 1989], [Godin 2000]. The mass flow of water vapour, between 2 different humidities (HR1 and HR2), is measured and the hydric diffusivity D calculated as in equation (1) [Baroghel-Bouny 1994], [Daïan 2001] with J the vapor mass flow, e the sample thickness, ρ_{dry} the density of the dry concrete, (dw/dRH) the isotherm slope.

$$D = \frac{J}{\rho_{dry}} \frac{e}{\Delta RH} \frac{dRH}{dw} \tag{1}$$

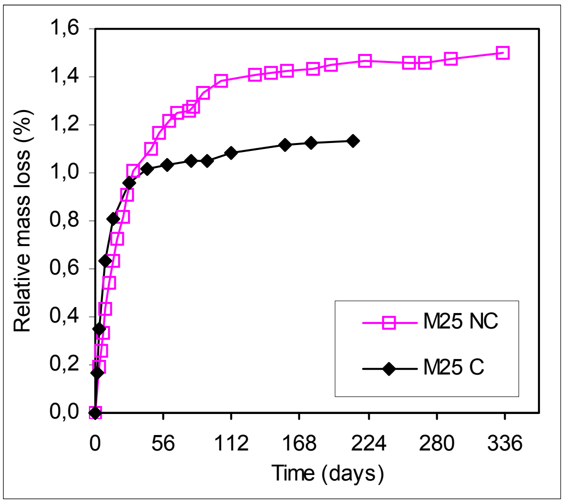

Figure 1. Mass loss of M25 carbonated C and non carbonated NC during drying at RH=90%

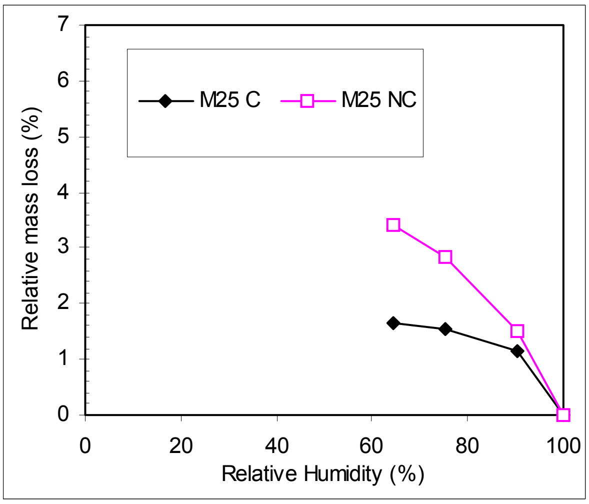

Figure 2. Desorption isotherm of carbonated and non carbonated concrete called M25

The 3 couples of RH presented here are 100-81%, 90-75% and 75-53%, 3 devices are built for each RH couple to dispose of a mean curve of the mass flow [Fig. 3]. The tests last 6 months and the diffusion coefficient given correspond to the average of 5 last weighings when the mass flow remains constant [Fig. 4]. Thus steady transfer is reached and diffusion is free from interactions between water vapor and cementitious material.
The main drawback of this test, used to determine the transport properties of concrete, is the slice thickness that is very low in comparison with the maximal size of gravel. However, vapor goes through a surface whose dimensions correspond to a representative sample of the concrete. Moreover slices without big air bubbles are carefully chosen. Meanwhile, the influence of the interfacial transition zone on diffusion can not be avoided, but it does not affect the comparison between carbonated samples and the others. Besides, this effect is slightly reduced with limestone aggregates.

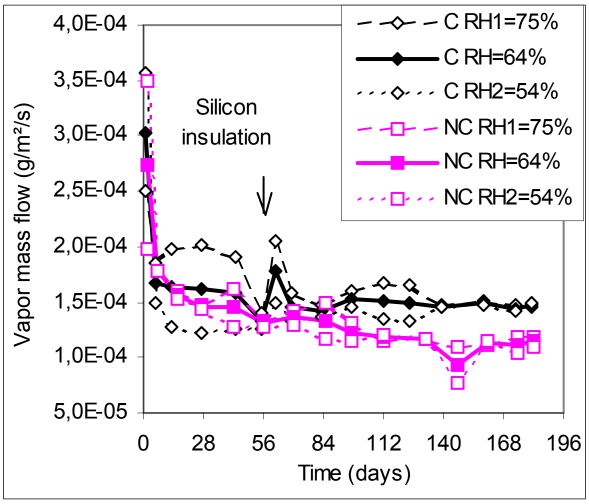

Figure 3. Vapor mass flow of M25 carbonated and non carbonated at the average RH=64%, between RH1=75% and RH2=54%

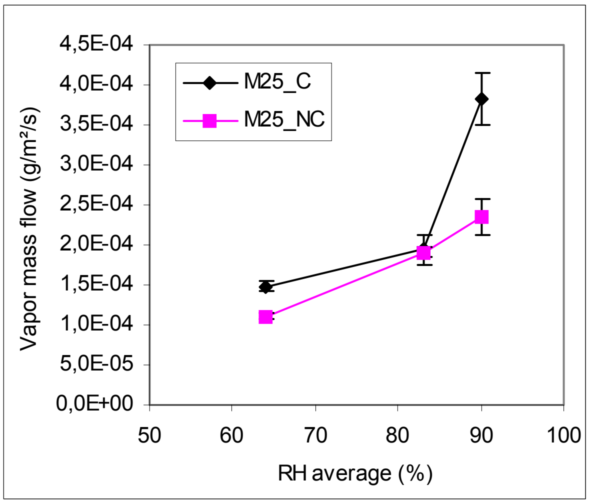

Figure 4. Vapor mass flow of carbonated and non carbonated concrete M25 versus average relative humidity

2.2 Investigation of the microstructure

The global porosity of the 3 studied concretes is measured by water saturation of 5cm thick samples. Their porosimetry is determined by mercury intrusion in a small sample of the concrete pastes. This method allows the evaluation of the pore distribution and the critical pore radius r_c corresponding to the smallest pore of a continuous path through the material in the Katz-Thompson theory [1987]. Nevertheless, mercury intrusion porosimetry is not suitable to measure the global porosity because it does take into account neither the bubbles nor the hydrate micropores.

2.3 "Cembureau" method to determine gas permeability

Gas permeability is measured by oxygen at 20°C in the 3 concretes dried at 105°C until constant mass. The device used is the one developed by Kollek [1989] and the method to obtain the intrinsic gas permeability according to Klinkenberg is described in [Villain *et al.* 2001]. The results of Jafaar [Thiery *et al.* 2003b] are confirmed by a second experimental program for M25 and M40 and the permeability is the average of 4 samples for both materials, carbonated or non carbonated.

2.4 Electrical resistivity test to determine liquid permeability and tortuosity

The electrical resistivity of the samples is measured according to the method described by [Nilsson 1996] using direct current ranging from 2 to 24V. Since it was not possible to extract the pore interstitial solution, an artificial one has been made for M25, the same as this used for cement pastes by [Quenard *et al.* 1999] and another one for M40 and M50 to respect the pH differences. They consists of 1.52g/L of $Ca(OH)_2$, 1.1g/L of NaOH and 12.5g/L of KOH for M25 and 1.76g/L of KOH for M50. The solution conductivities are measured by an electrode and are in a roughly good agreement with the conductivities measured by [Tumidajski *et al.* 1996] in extruded porewater of cement pastes. To carbonate the solutions, a gas mix containing 50% of CO_2 has blown bubbles until the conductivity remains constant.
The formation factor is the ratio of the concrete resistivity ρ by the solution resistivity ρ_0: $F=\rho/\rho_0$.
Once the formation factor and the critical pore radius r_c determined, liquid water permeability k_l can be calculated thanks to Katz-Thompson theory [1987] by equation (2):

$$k_l = \frac{4.r_c^{\;2}}{226.F} \tag{2}$$

Garboczi [1990] shows that this theory built for rocks can be applied for cement pastes. Tortuosity T is the inverse of porosity ϕ by the formation factor F: $T=1/(\phi.F)$. Table 2 presents the results.

	concrete resistivity ρ (Ω.m)	solution resistivity ρ_0 (Ω.m)	formation factor F (-)	porosity by water sat. ϕ (%)	tortuosity $T=1/(\phi.F)$	critical radius r_c (nm)	permeability k_l (m²)
M25_NC	105	0.278	377.9	15.1	$1.76.10^{-2}$	50	$1.17.10^{-19}$
M40_NC	107	0.169	630.7	14.7	$1.08.10^{-2}$	40	$4.49.10^{-20}$
M50_NC	145	0.169	856.3	13.6	$8.60.10^{-3}$	30	$1.86.10^{-20}$
M25_C	188	0.764	246.3	13.7	$2.97.10^{-2}$	300	$6.47.10^{-18}$
M40_C	322	0.588	547.0	11.1	$1.65.10^{-2}$	100	$3.24.10^{-19}$
M50_C	1164	0.588	1978.2	9.7	$5.20.10^{-3}$	400	$1.43.10^{-18}$
M50_C						*40*	*$1.43.10^{-20}$*

Table 2. Resistivity measurement results of non carbonated (NC) and carbonated (C) concretes

3 INFLUENCE OF CARBONATION ON VAPOR DIFFUSIVITY

3.1 Modification of the microstructure

Carbonation leads to a reduction of global porosity measured by water saturation [Table 2] because the products like calcium carbonates and silica gels occupy a bigger molar volume than the initial components like calcium hydroxide or C-S-H [Thiery *et al.* 2003a]. The global porosity obtained by mercury intrusion present the same trend. Figure 5 shows the pore distributions for M25 and M50. For all the studied concretes, carbonation fills in a part of the small pores, which are characterized by peaks centred around 10 to 50 nm. For the most porous concrete (M25), larger pores (200 nm peak) develop after carbonation. For the less porous concrete (M50), it is not so clear: pores are scattered from 10 to 1000 nm. Finally, pores are coarser in carbonated concretes than in non carbonated. Ngala & Page [1997] note also an increase of capillary porosity (pores bigger than 30nm) in carbonated cement pastes.

Chaussadent *et al.* [2000] show the relationship between water-cement ratio and hydrate size in cement pastes. Hydrates are bigger in porous concretes because they can well develop in capillary pores. So we can assume that big lime hydrates dissolution leave room to bigger pores and that carbonation products preferably develop in the smaller pores. Thus it is possible to explain the appearance of 200nm pores at the same time as the filling of the 30nm pores on the one hand, and the differences between M25 and M50 (or M40) on the other hand.

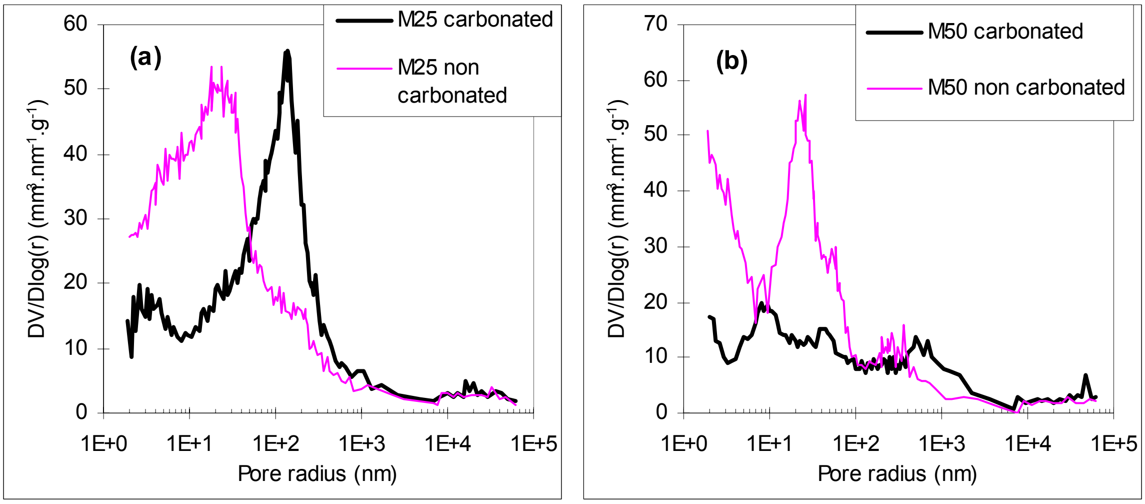

Figure 5. Pore distribution by mercury intrusion porosimetry: M25 (a) and M50 (b)

3.2 Hydric properties

Firstly, we note that M25 carbonated samples dry more rapidly than M25 non carbonated ones, whatever the hydric conditions in the desiccator, for RH greater than 64%. Figure 1 illustrates this for RH=90%. This can be due to the coarser porosity in the carbonated M25 samples as shown by Fig. 5a. Secondly, the desorption isotherm obtained [Fig. 2] for the non carbonated concrete M25 is correlated with the curve obtained by Baroghel-Bouny. Carbonation flattens the desorption isotherm, which means that the slope of the M25 carbonated isotherm is smaller than the one of the non carbonated M25 for each RH. These results are in agreement with the shape of the cement pastes isotherms of [Houst & Wittmann 1994].

3.2 Vapor diffusivity versus relative humidity

Whichever of the RH couple is used, the mass flow of water vapor and the hydric diffusivity of the carbonated concrete M25 are greater than the mass flow and the diffusivity of the non carbonated M25 [Fig. 4 & 6]. That means there is an easier path to go through the carbonated sample and once again the coarser porosity of the carbonated M25 can explain this result. Ngala & Page [1997] put the same reason forward to explain the increase of chloride diffusion coefficient due to carbonation of cement pastes.

Meanwhile, it does not agree the modelling equation proposed by Papadakis *et al.* [1991], which predicts a decrease of diffusion coefficient as global porosity drops due to carbonation. But the experimental results of this paper corresponding to diffusion of O_2 and CO_2 in cement pastes do not show such a clear decrease of the diffusion coefficient and the size of the molecule of the gas has certainly an influence. That is why this comparison leads to be careful to generalise the Papadakis's equation for all the gas transportation.

Besides hydric diffusivity is determined by dividing the flow by the slope of the desorption isotherm (equation (1)). That is why the curve obtained [Fig. 6] expresses the effect of interaction between water vapor and the matrix of the concrete M25 [Baroghel-Bouny 1994].

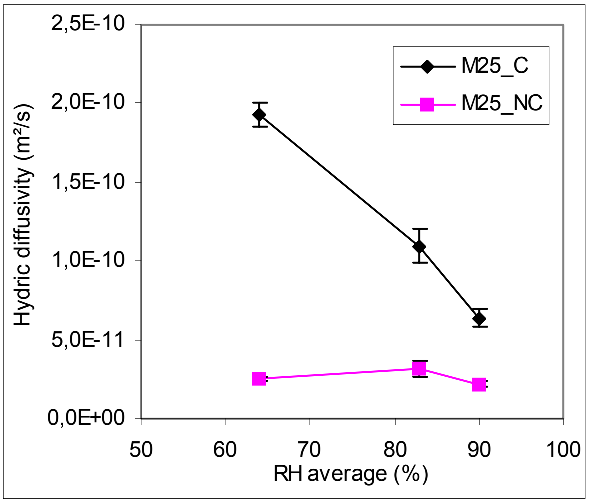

Figure 6. Hydric diffusivity of carbonated and non carbonated M25 versus average RH

4 INFLUENCE OF CARBONATION ON GAS AND LIQUID WATER PERMEABILITY

The water content of concretes has a great influence on its transport coefficients [Garboczi 1990], [Houst & Wittmann 1994] and [Villain et al. 2001]. But here, we choose to simplify the study and to present only the gas permeability in fully dried concretes and the liquid water permeability in saturated ones.

4.1 Gas permeability

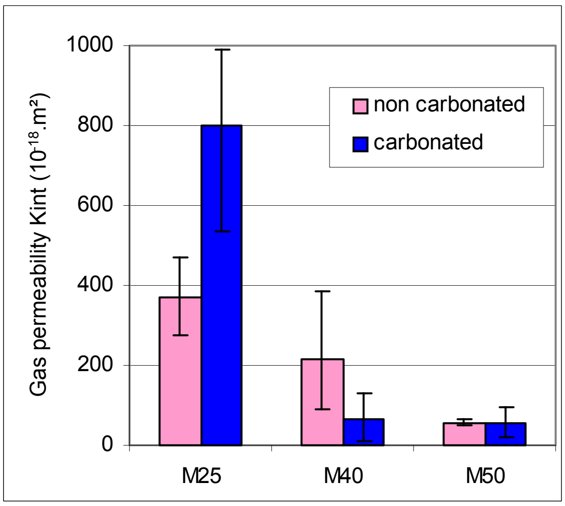

Figure 7. Intrinsic gas permeability of dried concretes, average, min and max values

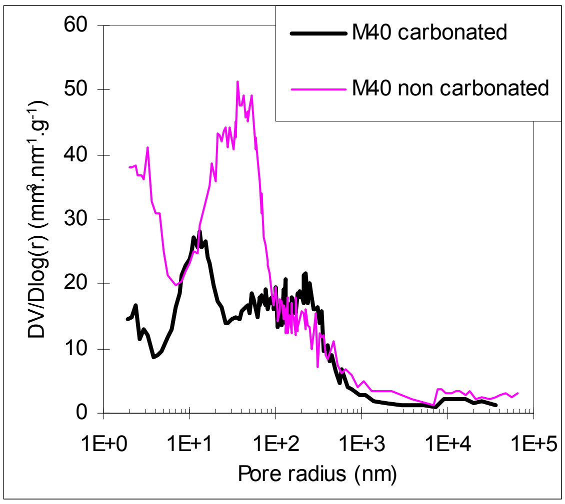

Figure 8. MIP pore distribution of M40

Figure 7 shows the mean intrinsic gas permeability calculated from permeability measurement at different pressures, in several samples for the 3 studied materials carbonated or not. We observe that carbonation leads to an increase of the intrinsic permeability of the most porous concrete M25 and a decrease of the permeability of M40. We could assume that it is due to the size of the lime hydrates that are well developed and bigger in M25 whose water-cement ratio is higher than for M40, what we explained in a previous paragraph. When these hydrates dissolve they leave more room and an easier path for gas. The 100nm peak of carbonated M40 [Fig. 8] is smaller than the 200nm peak of carbonated M25 [Fig. 5a].

4.2 Liquid water permeability and microstructure modifications

First, we note that carbonation increases the resistivity of all the tested concretes. Indeed the ion concentration (especially the one of hydroxide ions) drops during carbonation. So the interstitial solution is less conductive and, moreover, porosity decreases. Resistivity is a good criterion to characterise concretes [Andrade 2000], especially in our case because is independent of experimental bias like making an artificial solution. But this is a useful means to obtain durability and microstructural indicators such as tortuosity and water permeability.

Then for the formation factor and the tortuosity, carbonation seems to induce different effects on the 3 concretes. M25 tortuosity increases whereas M50 tortuosity decreases, because the evolutions of porosity and concrete resistivity are conflicting: $T = \rho_0/(\phi.\rho)$. For the less porous concrete (M50), resistivity grows so much that tortuosity decreases. For the most porous concrete (M25), the diminution of porosity has a stronger impact on tortuosity, which increases.

Meanwhile the microstructure modification due to carbonation mainly influences the liquid water permeability by means of the critical radius r_c (see equation (2)). For M25 and M40, we already observe that porosity is coarser, so the characteristic critical radius is bigger in carbonated concretes. Moreover, the formation factor decreases so water permeability increases noticeably. As for M50, it is not easy to determine the critical radius. In table 2, two options are presented. In case of r_c = 400 nm, permeability grows so drastically that it gets greater than carbonated M40 permeability. On the contrary, in case of r_c = 40 nm, permeability does not change because of carbonation.

5 CONCLUSIONS

In this paper, the influence of carbonation on microstructure and transport properties is evaluated. For the 3 studied concretes (M25, M40 and M50), global porosity is reduced, small pores (≈ 30 nm) are filled by carbonation products and bigger pores (≈ 300 nm) appear. This clearly influences transport properties of the most porous concrete M25. M25 vapor diffusivity, its gas and liquid water permeabilities increase due to carbonation. As a consequence, carbonated M25 dries more quickly than non carbonated M25, diffusion and convection phenomena being concomitant.

Besides, carbonation leads to several effects on permeability properties of the other concretes. M40 gas permeability decreases whereas M40 liquid water permeability increases. In less porous concretes, in their densified microstructure, the pores involved in gas and liquid transferts are certainly different. And the interactions between water and hydrates must play a prominent part.

For modelling carbonation, it is fundamental to dispose of carbon dioxide diffusion coefficient versus porosity and water content in carbonated and non carbonated concrete. But its measurement is not possible in the non carbonated concrete. Consequently the study deals with vapor diffusivity. Moreover we intend to determine this diffusivity according to the material water content to try to estimate the carbonation impact on the carbon dioxide diffusion coefficient at different water content. As our results show that carbonation increases vapor diffusion for M25 and for the mean relative humidities studied (90, 82 and 64%), whereas literature references report that carbonation induces a reduction of the carbon dioxide coefficient, we can not presently conclude and further research has to be carried out.

6 REFERENCES

TT1-62, Impact of carbonation on microstructure
and transport properties of concrete, G. Villain & M. Thiery

Andrade, C., Alonso, C., Arteaga, A. & Tanner, P. 2000, 'Methodology based on the electrical resistivity for the calculation of reinforcement service live', Proc. Suppl. Papers of 5th CANMET/ACI Int. Conf. on Durability of Concrete, Barcelona Spain, 4-9 june 2000, 899-915.

Baroghel-Bouny, V. 1994, Caractérisation des pâtes de ciment et des bétons – Méthodes, Analyse, interprétations, L.C.P.C., Paris, 468 p.

Chaussadent, T., Baroghel-Bouny, V., Hornain, H., Rafaï, N. & Ammouche, A. 2000, 'Effect of water-cement ratio of cement pastes on microstructural characteristics related to carbonation process', Proc. of 5th CANMET/ACI Int. Conf. on Durability of Concrete, Barcelone, 523-537.

Daïan, J-F. 2001, 'Evaluation des propriétés de transfert dans les matériaux cimentaires', Revue Française du Génie Civil (RFGC), 5[2-3], 179-202.

Delcelier, P. 1989, Transferts de vapeur, Cahiers du CSTB, 298[2322], 1-18.

Godin, J. 2000, Determination de la perméabilité et de la diffusivité à la vapeur d'eau d'un béton par la méthode à la coupelle, Proc. of tech. days of the LPC, Bordeaux (France), 8-9 march 2000, 65-71.

Garboczi, E.J. 1990, 'Permeability, diffusivity and microstructural parameters, a critical review', Cement and Concrete Research (CCR), 20, 591-601.

Houst, Y.F. & Wittmann, F.H. 1994, 'Influence of porosity and water content on the diffusivity of CO2 and O2 through hydrated cement paste', Cement and Concrete Research, 24[6], 1165-1176.

Kollek, J.J. 1989, 'The determination if the permeability of concrete to oxygen by the Cembureau method – a recommendation', Materials and Structures, 22, 225-230.

Ngala, V.T. & Page, C.L. 1997, 'Effects of carbonation on pore structure and diffusional properties of hydrated cement pastes', Cement and Concrete Research, 27[7], 995-1007.

Nilsson, L.O., Poulsen, E., Sandberg, P., Sorensen, H.E. & Klinghoffer O. 1996, Chloride penetration into concrete- State-of-the-art -Transport processes, corrosion initiation, test methods and prediction models, HETEK Report n° 53, ed. Frederiksen, Danish Road Directorate, Copenhagen, 50-56.

Papadakis, V.G., Vayenas, C.G. & Fardis, M.N. 1991, 'Physical and chemical characteristics affecting the durability of concrete', ACI Materials Journal, 8[2], 186-196.

Quenard, D., Kalifa, P. & Pardo, A.M. 1999, Mesure du facteur de formation et de la perméabilité dans les pâtes de ciment, Final research report, CSTB, février 1999, 24p.

Thiery, M., Villain, G. & Platret, G. 2003a, 'Effect of carbonation on density, microstructure and liquid water saturation of concrete', Proc. of 9th Eng. Conf. on Advances in Cement and Concrete, Copper Mountain (USA), 481-490.

Thiery, M., Villain, G. & Jafaar, W. 2003b, 'Estimation de la perméabilité des matériaux cimentaires par porosimétrie au mercure' (in french), 28th sci. days of GFHN, Dijon, 24-26 nov., 139-144.

Thompson, A.H., Katz, A.J. & Krohn, C.E. 1987, 'The microgeometry and transport properties of sedimentary rocks', Adv. Phys., 36[5], 625-693.

Tumidajski, P.J., Schumacher, A.S., Perron, S., Gu, P. & Beaudoin, J.J. 1996, 'On the relationship between porosity and electrical resistivity in cementitious systems', Cement and Concrete Research, 26[4], 539-544.

Villain, G. Baroghel-Bouny, V., Kounkou, C. & Hua, C. 2001, 'Measurement of gas permeability versus the saturation degree of concretes' (in french), RFGC, 5, 251-268.

Villain, G., Thiery & Roussel, P. 2004, 'Determination of drying and carbonation profiles within concretes by gamma-ray measurement', Bulletin des Laboratoires des Ponts et Chaussées, 248, 49-71.

Effect of High Temperature on the Mechanical Properties and Ultrasound Velocity of Cellular Gas Concrete

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ABSTRACT

Exposure to high temperatures affects various properties of concrete and mortar, including their mechanical strength. Susceptibility to damage under high temperatures depends mostly on the properties of the matrix and the dispersed phase of the material. Because cellular gas concrete has very fine aggregate admixtures, evaluating its response to high temperatures requires testing the material as a whole, rather than considering the phases separately.

In the case of fire damage, tests generally can be done on specimens taken from the affected areas. In addition, ultrasound velocity test is often used to evaluate the degree of damage indirectly. In this experimental research, the change of mechanical properties such as compressive strength and splitting tensile strength according to high temperature has been examined by ultrasound velocity. In these tests, the applied temperature changes from 100°C to 965°C and the results of these changes were determined.

As a result, this paper tries to find out if there is a meaningful relationship between the mechanical properties of the material under high temperature and if testing the material with ultrasound velocity is an efficient and adequate option in order to examine and determine the damage occurs in the material after exposure to temperatures from 100°C to 965°C. In this context the relationship between each property such as compressive, flexural, splitting tensile strength and ultrasound velocity was determined. Under these observations it was tested that if there is meaningful correlation between these properties and ultrasound velocity.

KEYWORDS

Cellular Gas Concrete, High Temperature, Ultrasound Velocity, Mechanical Properties.

1 INTRODUCTION

Cellular gas concrete is used widely in building construction. Especially since the second half of the twentieth century, this inorganic material has been used for non-load-bearing purposes, as a filling material in floors, wall blocks, and panels in partition walls. In addition, in the last quarter-century, load-bearing gas concrete reinforced roof and floor elements have been introduced. Gas concrete components also have been used to make small prefabricated buildings. General properties of this type of material and the related research done on this subject are listed in Narayanan & Ramamurthy [Narayanan & Ramamurthy 2000].

Its inorganic, porous structure gives this material particular importance from the point of view of building physics. Its thermal insulation qualities give it considerable importance. It also protects other building elements, like structural steel, from fire, and may be used as a fire wall with its inherent structural quality. Unfortunately, while it is noncombustible, fire may still destroy gas concrete irreversibly. All building materials, especially the load bearing structural materials are affected by this important phenomenon [Baltzer 1988].

The Modulus of Elasticity (MoE) is an important parameter for structural stability together with the other mechanical properties like compressive and flexural strengths. It defines the load-bearing capacity and dimensional change of a structure under load and should be assessed after exposure to high temperatures regarding the reliability of the material. This subject related to concrete is evaluated from different points of view in various publications [Ashby & Jones 1993] [Bingöl & Gül 2004] [Farage et al. 2003] [Neville 2000] [Pundit Manual 1989] [Vodak et al. 2004].

This study determines the changes in the mechanical properties of gas concrete after exposure to high temperature. The main aim is to assess its reliability and safety after exposure to high temperatures. It also relates the changes in mechanical properties, and in the ultrasound velocity, to each other and to physical changes in the material.

2 THERMAL TREATMENTS AND EXPERIMENTAL PROCEDURES

For this experimental study, gas concrete specimens were obtained from the producer. The material is classified as G4. Table 1 gives typical properties [Borhan 1986].

Type of the Cellular Gas Concrete Block	(G4)		*Physical Properties*		
Mechanical Properties			*Specific Density*	[gr/cm^3]	2.60
Modulus of Elasticity (MoE)	[kN/mm^2]	1.950-2.0	*Unit Weight*	[gr/cm^3]	0.59±1
Compressive Strength	[N/mm^2]	~ 3.3	*Pore Proportion*	[%]	75
Tensile Strength	[N/mm^2]	~ 0.5	*Pore size*	[mm]	0.5-1.5
Flexural Strength (MoR)	[N/mm^2]	~ 0.7	*Thermal Conductivity*	[W/m^{o}K]	0.14
Shear Strength	[N/mm^2]	~ 1.1	*Thermal Expansion Coefficient*	[m/m ^{o}C]	$0.8 \cdot 10^{-5}$

Table 1. Properties of the material used during the experiments

The dimensions of the specimens are 4x4x8 cm. The samples were kept at room temperature (20-21°C) and constant humidity of (60±5% R.H) until they reached constant moisture content and weight. For heating, a specimen was placed in an oven, and the oven temperature increased to the desired level: 100, 200, 400, 600, 800 and 965°C, respectively. After reaching the temperature setpoint, the samples were soaked for 30 minutes, in order to homogenize the temperature throughout the material. After this time the heating was stopped and specimens were left in the oven for slow cooling down to room temperature. Then they were removed from the oven and replaced into desiccators, to prevent further uptake of humidity. Dimension, weight and ultrasound velocity of each material were measured before and after high temperature exposure and thus found volume, unit weight and modulus of elasticity (MoE). In order to prevent dimensional effect that may arise from cutting the specimens, each material that belongs to each heating regime were measured separately. Three specimens were prepared for each temperature effect and the volume, unit weight, MoE, and compressive and splitting strengths were measured for each specimen.

Weights were measured with a PRECISA 4000C electronic scale, which has a 10 kg capacity and 0.01 gr. precision. Ultrasound pulse velocities were determined by CNS Electronic Ltd. PUNDIT nondestructive ultrasound equipment. The mechanical tests were done by Amsler Type 6DB7F120 Hydraulic test equipment with a capacity of 6-60 kN, and by a Losenhausenwerk Hydraulic test equipment with a capacity of 20-200 kN. A NUVE MF100 oven was used to obtain high temperatures; this oven can reach a maximum of 1000°C, with 1°C precision. Table 2 gives the average measurement results, both before and after heating.

Temp [˚C]	*Volume [cm^3]*		*Unit Weight [g/cm^3]*		*MoE [kN/mm^2]*		*Compresive Strength [KN/mm^2]*	*Splitting.Strength [kN/mm^2]*
	before	*after*	*before*	*after*	*before*	*after*		
20	123,6		0.69		2,36		3,29	0,71
100	136,1	137,3	0,56	0,56	2,37	2,40	3,48	0,90
200	135,1	135,8	0,57	0,55	2,28	1,98	3,70	0,55
400	135,8	131,6	0,57	0,55	2,29	1,54	3,43	0,39
600	133.8	128,9	0,59	0,55	2,27	1,17	3,66	0,50
800	134,1	127,6	0,59	0,53	2,39	0,55	3,18	0,29
965	134,3	124,0	0,59	0,55	2,47	0,55	2,12	0,19

Table 2. Measurement results for the different temperature treatments.

3 RESULTS AND DISCUSSION

The specimen properties were tested both before and after exposure to high temperature. Hence, differences in the properties could be evaluated.

3.1 Relationship between Physical Properties and Applied Temperature

As mentioned, temperature is elevated up to 965°C. The thermal expansion coefficient of gas concrete is estimated as $0.8x10^{-5}$ m/m °C [Borhan 1986]. Presumably when the elements are exposed to high temperatures, they expand. However, after cooling they become relatively shorter than their original size. Figure 1 shows that the shrinkage depends on the applied temperature. It also shows the theoretical thermal expansion.

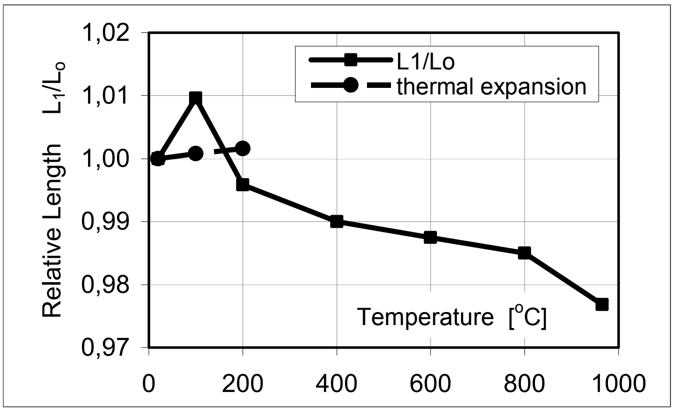


Figure 1. Effect of temperature on the relative length of materials.

The dimensional changes directly affect the volume of the material. 'Figure 2.1' shows the relative volumes of the specimens, determined from their dimensions subsequent to cooling. For comparison, 'Fig 2.2' shows equivalent data for selected gypsum paste and lightweight mortars [Tanacan & Ersoy 2001]. While the relative volume of tested gas concrete after heating to 965°C is 0.97-0.98, it is around 0.65 for the gypsum paste and mortars. Possibly the superior volume stability of gas concrete makes it a better choice for use as a fire wall or screen, because it may limit the propagation of cracks during cooling. However, if the material is cooled by water during the fire, the rapid shrinkage may lead to strong crack propagation regardless of the volume stability. This subject requires further research.

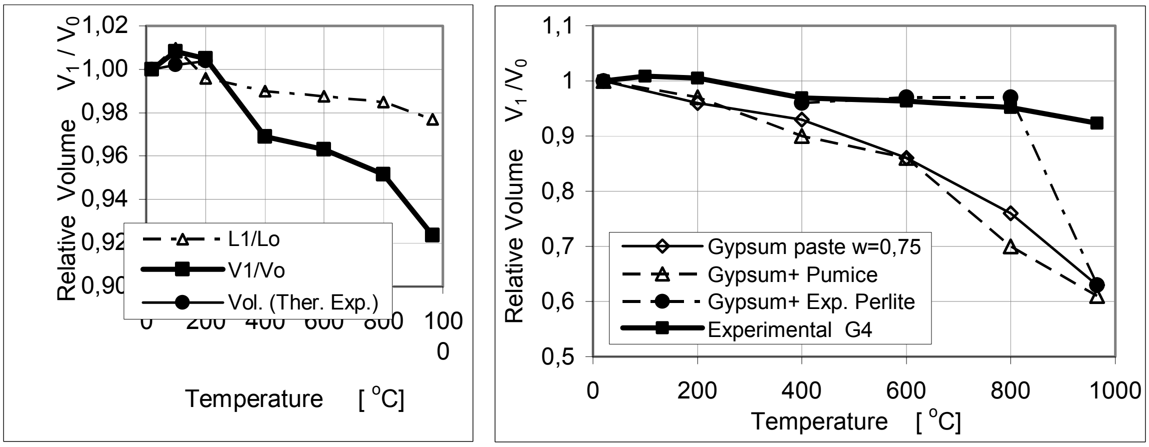


Figure 2.1. Effect of temperature on the relative volume of samples (left).
Figure 2.2. Effect of temperature on the relative volume of different gypsum paste and lightweight mortars (right).

'Figure 3' gives the variations in weight and unit weight of the material depending on the applied temperature. The measured values are given on the left and relative values on the right. The behavior of gypsum paste and gypsum mortars is also added to 'Fig 3.2', for comparison.

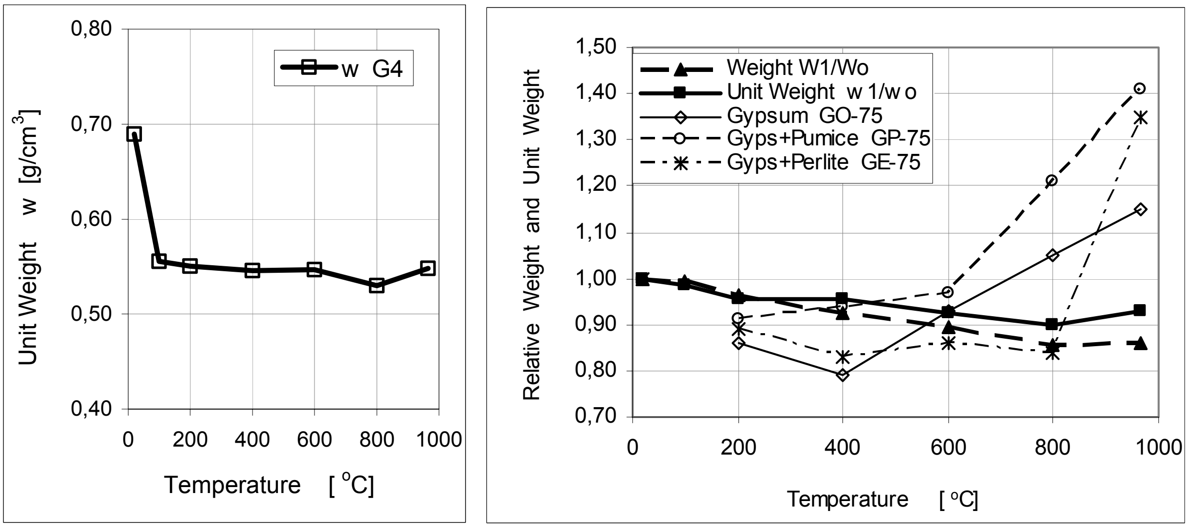
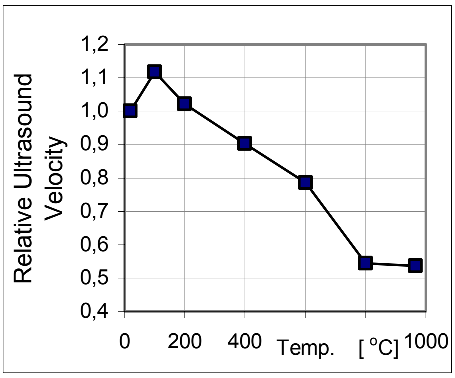


Figure 3.1. Unite Weight of gas concrete (G4) at different temperatures (left)
Figure 3.2. Effect of high temperature on the relative weight, unite weight of samples and of different gypsum paste and lightweight mortars (right)

3.2 Relationship between Modulus of Elasticity, Mechanical Properties and Applied Temperature

'Figures 4.1 and 4.2' show the change in ultrasound velocity depending on different temperatures. The ultrasound velocity increases after soaking at 100°C, but by 200°C it returns approximately to its initial value, and continues to decrease with higher temperature exposure. The main reason in achieving approximate results from MoE and strength tests is the evaporation of the physically bonded water from the material at the first 100°C. The compound water of the concrete starts to evaporate at ascending temperatures and this continues approximately up to 400°C. From this temperature level upwards shrinkage occurs depending on dehydration of Ca $(OH)_2$ in the compound.



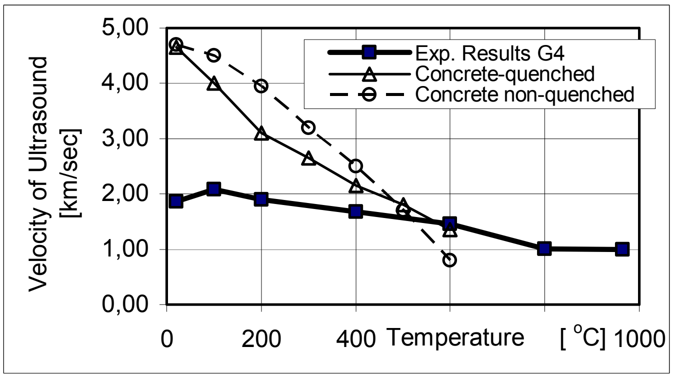


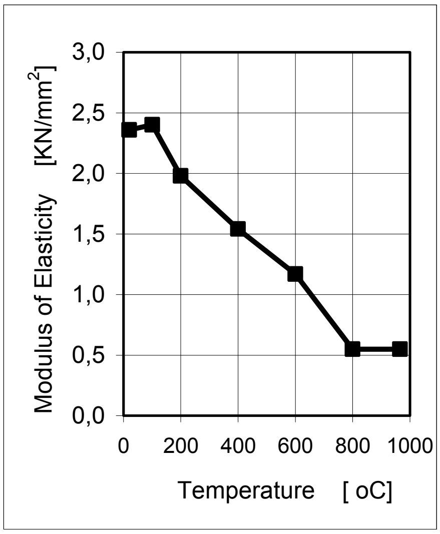
Figure 4.1. Relative ultrasound velocity of gas concrete (G4) at different temperatures (left)
Figure 4.2. Effect of temperature on ultrasonic pulse velocity in gas concrete (G4) and concrete (right).

The difference in behavior for exposure to 100°C versus higher temperatures relates to the evaporation of physically-bonded water from the material at 100°C. At temperatures of 300-500°C, the chemically-bonded water gets driven off. Thus from about 400°C, shrinkage depends on the dehydration of Calcium in the compound. Lime-rich mortars like Portland cements are highly affected. On the other hand the $CaCO_3$ in gas concrete is more resistant to high temperatures than $Ca(OH)_2$. Hence the mortars and mixtures in which carbonation is not completed are more susceptible to high temperatures [Neville 2000].

In 'Fig 4', note that the reference data, taken from research by Watkey, state the behavior of the concrete after the material is cooled by water and by air in normal atmospheric conditions [Pundit Manual 1989]. In this study, the materials are cooled in air. Therefore the values reported from this study probably are higher than what could be expected after cooling by water, as is usually the case in real life.

'Figure 5.1' shows the changes in MoE depending on different temperatures. Like the values obtained for ultrasound velocity, the MoE values also decrease gradually but at different slope than the curve achieved for ultrasound velocity. MoE values increase at the first 100°C and then from this temperature up to 800°C the curve gradually decreases. 800°C can be accepted as the approximate temperature where sintering starts [Borhan 1986]. From this temperature to 965°C, the relative values of MoE remain same. On the other hand, the variation of modulus of elasticity in cement paste between 20-300°C is in similar characteristics [Farage et al 2003].

MoE values obtained for gypsum and cement mortar composites are also given in 'Fig 5.2' for comparison [Neville 2000] [Tanacan & Ersoy 2001]. The variation of the data up to 400°C is akin to the data variation obtained for gas concrete. The data of the experimental study done with similar technique to this study gives the behavior of gypsum and lightweight perlite-gypsum mortars under the heat effect of maximum 800°C [Tanacan & Ersoy 2001]. Here, it is seen that gypsum mortar gives its physically bonded water at the first 100°C, hereafter particularly at the first 200°C it gives its chemically bonded water and dehydrates until 400°C.



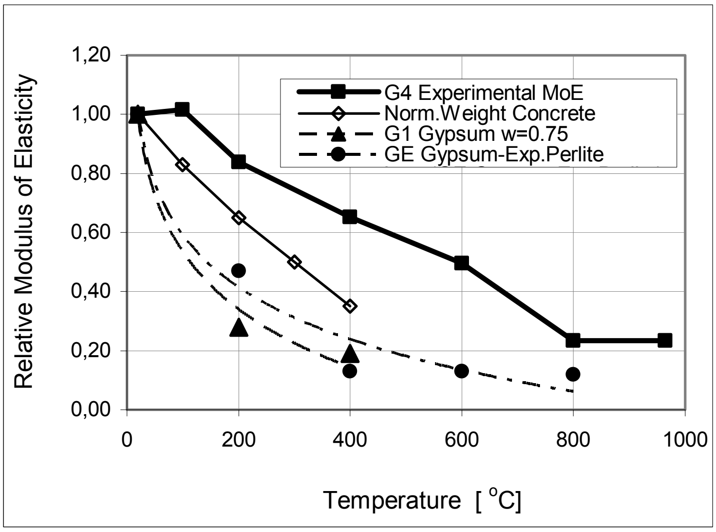
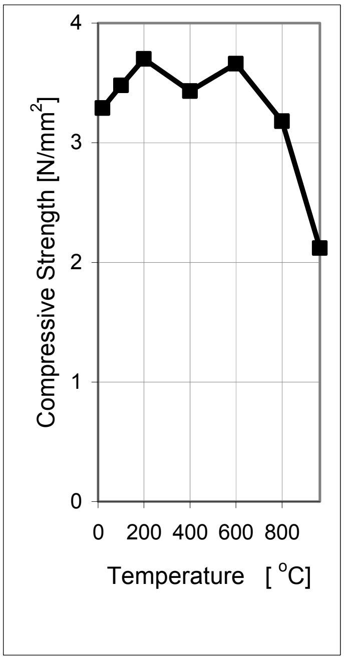


Figure 5.1. Modulus of Elasticity of gas concrete (G4) at different temperatures (left)
Figure 5.2. Effect of temperature on relative Modulus of Elasticity of gas concrete class G4; normal weight concrete and Gypsum (right)

'Figure 6.1' shows the results of the compressive tests depending on the elevated temperature. Compressive strengths increase at the first 400-600°C, decrease gradually until 600°C, but descends rapidly upwards.



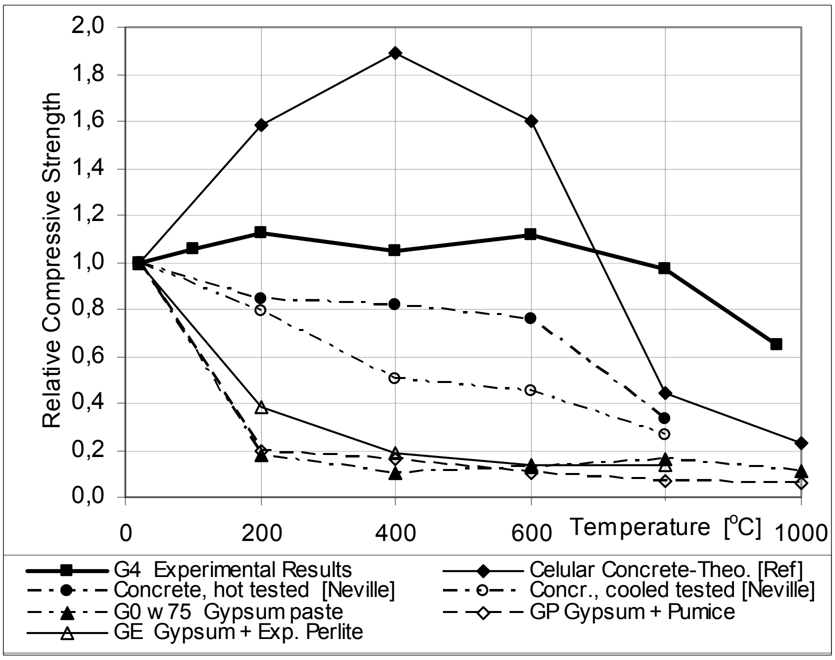
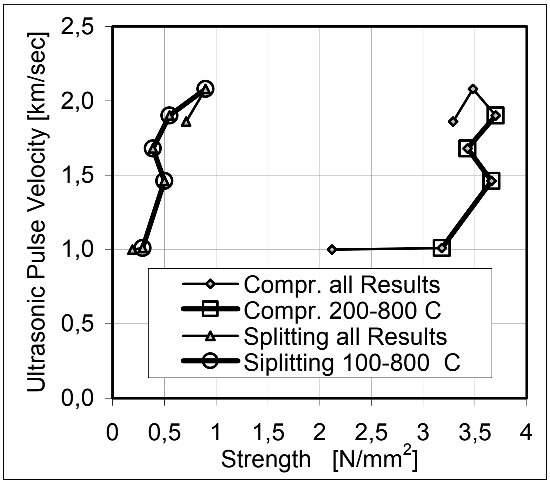


Figure 6.1. Compressive Strength of gas concrete (G4) at different temperatures (left)
Figure 6.2. Effect of temperature on relative Compressive Strength of gas concrete class G4; normal weight concrete, Gypsum, and theoretical values of gas concrete (G4) [Borhan 1986] (right)

There is no continuous similarity between experimental results of ultrasound velocity, MoE and compressive strength in this respect. Although the values of ultrasound velocity and MoE increase at the first 100°C and then descend until 800°C, the values of compressive strength decrease apparently only after 500-600°C. The related data of compressive strength and temperature relationship obtained from [Borhan 1986] is also given in 'Fig 6.2'. Here also, there is seen an apparent similar tendency between the tested results. In both different data compressive strength values fall lower than their initial values approximately at 700°C.

Experimental results related to normal concrete and gypsum mortars are added to the 'Fig 6.2' [Neville 2000]. Compressive strengths of concrete are hot and cooled tested. They both descend distinctively when the temperature is raised. While the compressive strengths of the cooled tested concrete which has same experimental conditions of this study are stable, test results of hot tested concrete decrease steeply particularly right after 600°C. The compressive tests of gypsum pastes and lightweight gypsum mortars have also similar tendencies.



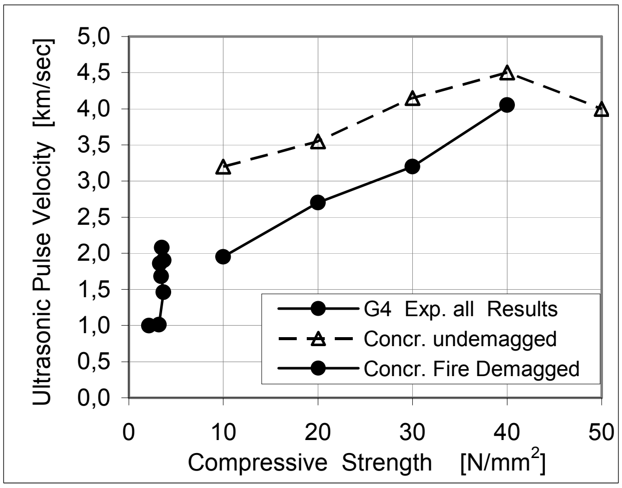
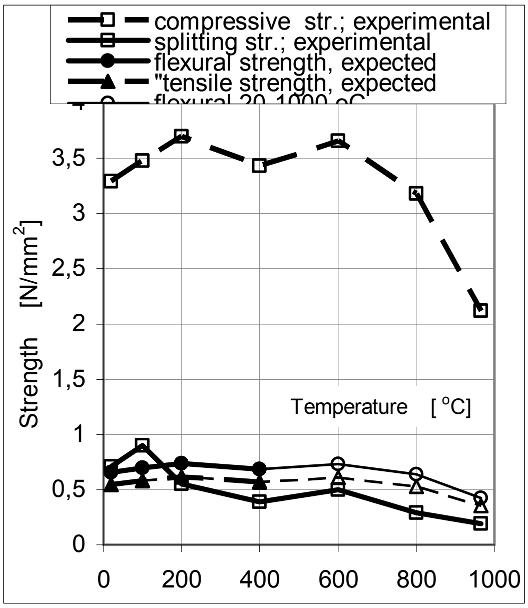


Figure 7.1. Relationship between strengths and ultrasonic pulse velocity according to the test results of gas concrete G4 (left)
Figure 7.2. Relationship between compressive strength and ultrasonic pulse velocity by celular concrete G4 according to the test results and two normal weight concrete series, undemaged and fire demaged according to Watkeys [Pundit Manual 1989] (right)

It is assumed that there is a relationship between ultrasound velocity and compressive strength. The different expressions related to the relationship between the MoE values and compressive strengths of the material were given in literature [Narayanan & Ramamurthy 2000] [CEB 1977]. Here, 'Fig 7.1 and 7.2' gives the relationship between strengths and ultrasound velocity for the G4 gas concrete and for the damaged and undamaged concretes left to high temperature exposure [Pundit Manual 1989][Ashby & Jones 1993]. 'Figure 7.1', gives the relationship between compressive and splitting test results and determined ultrasound velocity of G4 class gas concrete. Here, the first remarkable point is the disparity between the values achieved before and after high temperature exposure. Before, the reasons of the alteration in the properties of the material according to the elevating temperature were mentioned. Starting from approximately 100-200°C up to 800°C, there is a disruption and accelarating crack formation in the internal structure of the body. The splitting strength which is very sensitive to these cracks attains it's minimum value at 800°C. After this temperature is exceeded, these properties no longer vary significantly, but the compressive strength decreases until the level of 965°C related to its characteristic. Because of this reason, it can be said that, the temperature between 200-800°C is especially important on the relationship between mechanical strengths and ultrasound velocity.

There is an approximate relationship between compressive strength and flexural and tensile strength of gas concrete [Borhan 1986]. It is assumed that tensile strength of the material is 1/6 of its compressive strength and the flexural strength of the material is 1/5 of its compressive strength. In order to make an evaluation theoretical approximate tensile and flexural strengths that are calculated with the existing compressive strength test results are added to 'Fig 8'. As a general evaluation the relative reductive variation of the four basic properties depending on different temperatures are given altogether in 'Fig 9'.



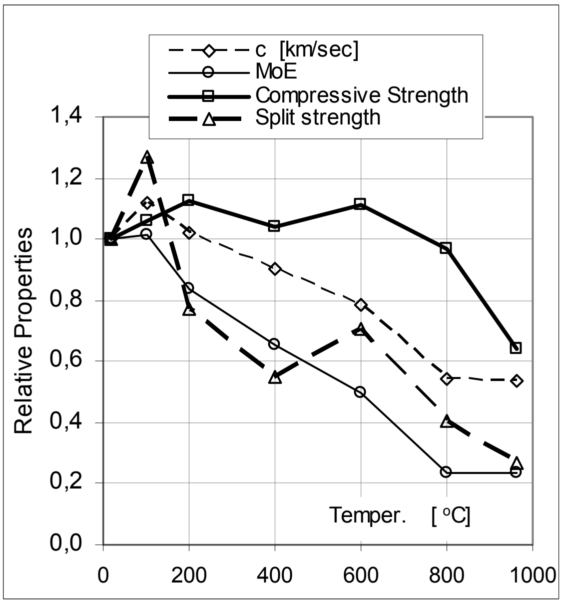


Figure 8. Experimental results of Compressive and Splitting Strengths and theoretical values of Tensile and Flexural Strengths depending on different temperatures (left)
Figure 9. The relative values of Ultrasound Velocity, MoE, Compressive Strength, Splitting Strength depending on different temperatures (right)

Splitting test which can give an idea related to the tensile strength of gas concrete has similar behavior with compressive strength depending on different temperatures 'Fig 9'. The maximum value (R_s~0.9 N/mm^2) is obtained around 100°C where presumably physically bonded water is evaporated from the structure. After relatively sharp reduction (~40 %) in splitting strength until 200°C, there is a stable and gradual reduction upwards. At 950°C the splitting strength is approximately 20% of the splitting strength achieved initially (R_s= 0.19 N/mm^2). The splitting test is influenced from high temperatures more than the compressive strength, because proportional but not quantitative values of compressive strength decrease approximately 57% at the same interval. The variation in the ratio of splitting strength/compressive strength is given in 'Fig.10'. The first 100°C has more positive affect on the splitting strength than on compressive strength, but at 200°C while splitting strength decreases, the compressive strength continues to increase which shows that splitting strength is more sensitive to the high temperature effect than compressive strength. The difference in the behaviours of the properties is getting diminished approximately after around 400°C and disappears after 800°C.

4 CONCLUSIONS

The possibility of using non-destructive test of ultrasound velocity in declaration of different properties of gas concrete under the effect of high temperature is researched. The determination of MoE by means of ultrasound velocity is a widespread application. It will be also useful to inter relate ultrasound velocity with the compressive and splitting test in evaluating gas concrete material. This study has a particular importance regarding to obtain reliable, confirmed results from non-destructive tests in the research of the relations among the properties of the material after high temperature exposure.

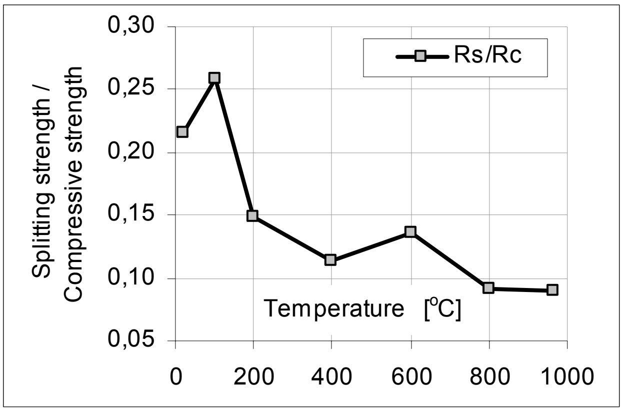


Figure 10. Variation of the ratio of splitting strength / compressive strength

The evaluations of the properties related to the high temperatures show that results achieved in this study are close to the results of concrete tests done under similar conditions. Evaluation of the test results comparatively with the test results achieved for those of concrete, gypsum and gypsum mortars which have similar characters like gas concrete strengthen the study. Gas concrete is used as a fire resistant material besides used as a thermal or a sound control material attributed to its lightweight body. Fire effect changes the dimensions and mechanical properties of the material in a different but similar way. Although the volume of the material increases at the first temperature levels due to the thermal expansion of the material, it increasingly shrinks depending on the rising temperature level. During the heating process, initially the physical bonded water and then the chemical bonded water release from the body. So that $CaCO_3$ and $Ca\ (OH)_2$ in cement are mainly affected from the high temperatures which inevitably influence the mechanical strengths.

Experiments show that between 20°C and 965°C weight and unit weights reduce approximately 15% where the similar reduction in volume is approximately 8%.

Flexural and splitting strength in which tensile strength is effective on them are more sensitive to the crack formation in the structure than compressive strength. Hence, the capillary crack formation that is assumed to appear between 200°C and 450°C causes splitting strength decreases strongly on the contrary to compressive strength which increases at this interval. Because of this reason it is assumed to be more reliable to search the relationship between compressive and tensile strengths at the temperature region particularly up to 100°C.

In ‘Fig 9’ it is seen that MoE values which are determined by the values of ultrasound velocity are affected from high temperatures one time (100%) more than the ultrasound velocity starting from approximately the lower temperatures. This is in fact the reflection of the changes in unit weight of the material. It is observed from the general evaluation of ‘Fig 9’ that high temperatures mostly affect the MoE values. This can have an unfavorable effect on the behavior of the material exposed to various mechanical stresses under the determined conditions.

Compressive and Splitting strengths both increase at the temperature level of 100°C, later especially splitting strength decreases rapidly to 27% of its initial strength at approximately 950°C.

In this respect, there is a close relationship between compressive and splitting strengths of the material. According to the literature, tensile strength of the material is approximately 1/6 of its compressive strength; this is approved by the splitting test results achieved approximately up to 100°C where the internal structure of the material is not destroyed by high temperature effect.

At the end of this step of the research, a close relationship is found between the ultrasound velocity and compressive and splitting strengths, especially between temperatures 200-800ºC. To conclude, it is seen possible to develop a foresight on the mechanical behavior of the gas concrete from the value of ultrasound velocity.

This study is planned to be developed by the research that will be done on the specimen right after the heat exposure (when they are hot) and after they are cooled.

5 REFERENCES

Ashby, M.F. & Jones, D.R.H. 1993, '*Engineering Materials 1; An Introduction to their properties and applications*', Pergamon Press, Oxford, New York.

Baltzer, H. 1988, 'Schäden verhüten statt vergüten', *Baugewerbe,* 68 [11], pp. 26-33

Bingöl, A.F. & Gül, R. 2004, 'Compressive strength of lightweight aggregate concrete exposed to high temperatures', *Indian Journal of Engineering and Materials Sciences*, 11 [1], pp. 68-72.

Borhan, B. 1986, *Ytong Handbook-1*, Türk Ytong Sanayi A.Ş., (*in Turkish*) Istanbul.

CEB Manual of Design and Technology 1977, *Autoclaved Aerated Concrete*, The Construction Press.

Farage,M.C.R., Sercombe, J. and Galle, C. 2003, 'Rehydratation and microstructure of cement paste after heating at temperatures up to 300 °C', *Cement and Concrete Research*, vol. 33, pp. 1047-1056.

Narayanan, N. & Ramamurthy, K. 2000, 'Structure and properties of aerated concrete: a review', *Cement and Concrete Composites,* vol. 22, pp. 321-329.

Neville, A.M. 2000, *Properties of Concrete*, (fourth edition) Prentice Hall, Pearson Education Ltd., Edinburg.

PUNDIT Manual, 1989, Mark IV, C.N.S. Electronics Ltd., London.

Tanacan, L. & Ersoy H.Y. 2001, 'Effect of high temperatures on the properties of gypsum-pumice and gypsum- perlite mortars', The First International Conference on Ecological Building Structure, San Rafael, California, July 5-9 (CD).

Vodak, F., Trtik, K., Kapickova, S., Hoskova, S and Demo, P. 2004,'The effect of temperature on strength-porosity relationship for concrete', *Construction and Building Materials* vol..18, pp. 529-534.

Comparison of Neutralization Depth Determined by Thermal Gravimetric Analysis of CaCO3 and A Phenolphthalein Method in An Accelerated Carbonation Test

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ABSTRACT

Protective performance against carbonation of concrete is one of the important performance for surface finishing materials. Such protective performance is generally evaluated by conducting an accelerated carbonation test of concrete specimens with surface finishing materials. After the accelerated carbonation test, neutralization depth is determined usually by use of a phenolphthalein method. However, carbonation depth values determined by the phenolphthalein method are not so accurate especially in case that specimens undergo little carbonation in the accelerated carbonation test.

In this context, thermogravimetric analysis (TGA) of cement mortar substrates was carried out after the accelerated carbonation test of cement mortar specimens with various finishing materials in order to evaluate the anti-carbonation performance of the finishing materials in this study. Quantitative analysis of calcium carbonate in the cement mortar substrates was performed in the depth direction from the surfaces of the specimens by TGA. The phenolphthalein method was also applied for determining neutralization depth of the specimens.

The main results obtained can be summarized as follows:
1) TGA clarified $CaCO_3$ concentration profiles of specimens in the depth direction and the $CaCO_3$ concentration curves demonstrated that $CaCO_3$ lies in even in non-newtralizated zones dermined by the phenolphtalein method and that $Ca(OH)_2$ lies in even in the newtralizated zones.
2) Protective performance of various types of finishing materials against carbonation could be evaluated and compared by the TGA method and the phenolphthalein method.
3) It is well known that a parabolic law can be obtained between carbonation periods and neutralization depth values determined by the phenolphthalein method. It was confirmed that such parabolic law was also obtained between carbonation periods and the total amounts of $CaCO_3$ formed in the specimens. The coefficients obtained from the both parabolic relations were reasonably corresponding to each other.

KEYWORDS

Carbonation, Newtralization, Parabolic Law, Finishing Material, Thermal Gravimetric Analysis

1 INTRODUCTION

A suppression of neutralization (carbonation) is expected for finishing materials for concrete. Such a suppression effect can be evaluated by an outdoor exposure test, an accelerated neutralization test, or an investigation of the actual conditions of buildings, etc. The neutralization of concrete substrates is usually evaluated by neutralization depth, which can be determined by spraying a 1 % ether alcohol solution of phenolphthalein (called a phenolphthalein solution here after).

The neutralization of concrete is a phenomenon where calcium hydroxide ($Ca(OH)_2$), produced by the hydration reaction of cement, changes to calcium carbonate ($CaC0_3$) by reacting with carbon dioxide in air. It is known that $Ca(OH)_2$ and $CaCO_3$ are mixed in the boundary of the coloring region due to the phenolphthalein solution. Namely, unreacted $Ca(CO)_2$ is mixed in the neutralization region determined and $CaCO_3$ is mixed in the un-neutralized region by the phenolphthalein solution. Thus, it is necessary to quantitatively examine the distribution of $CaCO_3$ produced and the total amount of $CaCO_3$ to stoichiometrically determine the degree of neutralization of concrete.

The suppression effect of neutralization of various types of finishing materials was evaluated from this standpoint by applying an accelerated neutralization test to test specimens with mortar base substrates. Moreover, the TGA method was used to evaluate the degree of neutralization of mortar, by which the amount of $CaCO_3$ produced in the mortar was estimated quantitatively. The neutralization depth was also determined by the phenolphthalein method, and this was compared with the results of the quantitative analysis of $CaCO_3$.

2 EXPERIMENTAL METHOD

2.1 Preparation of materials and mortar specimens

Cylindrical test specimens 100 mm in diameter and 400 mm in height were made of cement mortar prepared as in Table 1. The mortar test specimens were aged with a wet compress and water spray for the first two weeks and then kept at 20±2□ and RH 65±5% for the next two weeks. To completely avoid a laitance layer appearing on the cast mortar surface, as shown in Fig. 1, the part from the surface to 20 mm depth was cut off using a concrete cutter. For the surface for applying finishing materials, the cut surface was polished with an abrasive paper with a grain size of P180 (prescribed by [JIS R 6252:1999])

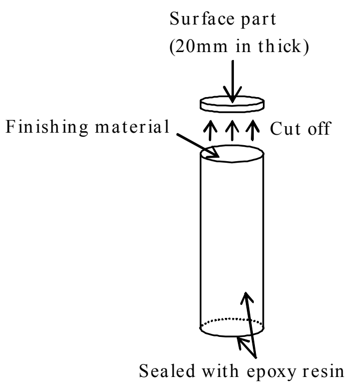


Figure 1. A mortar test specimen.

	Materials and mix proportion
Cement	Ordinary portland cement (Specified in JIS R 5201:1997)
Fine aggregate	River sand, Absolute dry specific gravity 2.56, Water absorption 2.22%
Cement / Sand	1:2.8
Water cement ratio	60% (45% and 75% in some specimens)

Table 1. Materials and mix proportion.

Mortar test specimens were prepared with the finishing materials shown in Table 2. The application specifications of the finishing materials were based on the standard specifications of the material manufactures. Epoxy resin was also carefully applied twice to the side and bottom surfaces of the specimens so that the coating thickness was 1 mm.

Code No.	W/C (%)	Surface finishing material
1-1	75	None (Bare mortar)
1-2	60	None (Bare mortar)
1-3	45	None (Bare mortar)
2	60	Polymer modified cement mortar (SBR latex, PL 5%, 10mm thick)
3	60	Polymer modified cement mortar (SBR latex, PL 14%, 10mm thick)
4	60	Polymer modified cement mortar (SBR latex, PL 20%, 10mm thick)
5	60	Epoxy resin mortar (5mm thick)
6	60	Epoxy resin mortar (10mm thick)
7	60	Acrylic emulsion paint (2 coats)
8	60	Polyurethane paint (2 component type, 1 coat)
9	60	Polyurethane paint (2 component type, 2 coats)
10	60	Acrylic paint (Solvent type, 2coats)
11-1	75	Multi-layer textured emulsion coating (Water proofing type)
11-2	60	Multi-layer textured emulsion coating (Water proofing type)
11-3	45	Multi-layer textured emulsion coating (Water proofing type)
12	60	Thin textured emulsion coating (Water proofing type)
13	60	Multi-layer textured emulsion coating (General type)
14-1	75	Thin textured emulsion coating (General type)
14-2	60	Thin textured emulsion coating (General type)
14-3	45	Thin textured emulsion coating (General type)
15-1	75	Water repellent material (Silane compound)
15-2	60	Water repellent material (Silane compound)
15-3	45	Water repellent material (Silane compound)
16	60	Water repellent material (Acrylic resin)

Table 2. List of mortar test specimens.

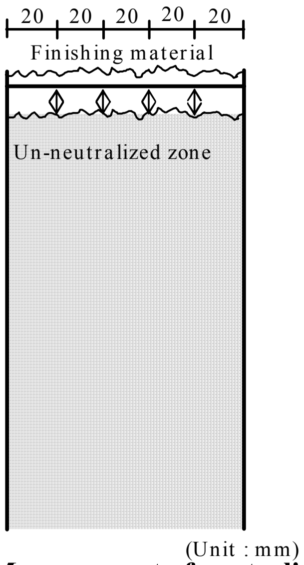
2.2 Accelerated neutralization processing

After the application of finishing materials, the mortar test specimens were aged at room temperature for one month. Then accelerated neutralization processing was carried out at a temperature of 30℃ at RH 60%, and a CO_2 concentration of 2%, for 6 or 24 months.

2.3 Measurement of neutralization depth and quantitative determination of $CaCO_3$ with the TGA method

The mortar test specimens to which the 6 or 24 month accelerated neutralization processing had been applied were divided into two parts by cutting. First, with one of the divided parts, the neutralization depth was determined by the phenolphthalein method. No neutralization regions were observed on the side and bottom surfaces sealed by epoxy resin, and thus the average of the depth of the four parts indicated in Fig. 2 was used as the neutralization depth of each mortar test specimen.

Next, the other divided part was further cut off using a finishing cutter as shown in Fig. 3. The 5 mm thick gray sections shown in Fig. 3 are the tab for the cutter. Moreover, the mortar test specimens were cleaved to get the central part, which were disintegrated using a boll mill or a mortar.
Scanning thermogravimetric analysis equipment (TGD-3000, Shinkuriko Co., Ltd.) was used for the thermogravimetric analysis. Mortar powder samples of about 45 mg were weighed exactly using a quartz cell. The thermogravimetric analysis was done at a heating rate of 5℃/min at temperatures between room temperature and 1000℃.



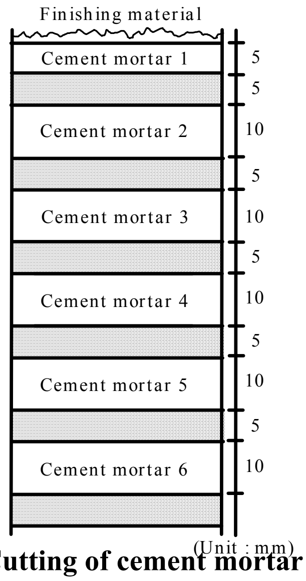


Figure 2. Measurement of neutralization depth. Figure 3. Cutting of cement mortar specimens.

3 RESULTS AND DISCUSSION

3.1 Estimation of $CaCO_3$ contents from thermogravimetric curves

The TGA curves for all the specimens decrease greatly at around 100□. Such a decrease in mass is considered to be due to evaporation of the water contained in the specimens. Also, at temperatures above 200□, each curve shows a stable base line coming from the completion of water evaporation.

Each thermogravimetric curve shows a great decrease in mass of the surface part (cement mortar 1) of the mortar test specimen at temperatures below and above 600□. Such a decrease in mass may be due to the thermal degradation of $CaCO_3$ produced in mortar shown in Eq. 1, from which the $CaCO_3$ contents were estimated.

$$CaCO_3 \quad \rightarrow \quad CaO \quad □ \quad CO_2\uparrow \qquad (1)$$

Additionally, as clearly seen from the comparison of the thermogravimetric curves of cement mortars 1, 2, 3, and 4 for example, with receding from the finishing surface, the decrease in mass around 600□ due to the degradation of $CaCO_3$ decreases, and the decrease in mass observed around 400□ increases. The decrease in mass around 400□ is expected to be due to the dehydration reaction of $Ca(OH)_2$ shown in Eq. (2), from which the contents of $Ca(CO)_2$ can be estimated quantitatively.

$$Ca(OH)_2 \quad \rightarrow \quad CaO \quad □ \quad H_2O\uparrow \qquad (2)$$

Thus, from Fig. 4, it was found that (1) in cement mortar 1 $Ca(OH)_2$ does not exist because the neutralization progresses, (2) inversely, in cement mortar 4, so far from the surface, there is a large amount of $Ca(OH)_2$ and a small amount of $CaCO_3$, because the neutralization progresses very little, and (3) for cement mortars 2 and 3 the amounts exhibit values between those of cement mortar 1 and 4.

Also, the idea that the decreases in mass around 600 and 400□ may be caused by the decreases in $CaCO_3$ and $Ca(OH)_2$, respectively, was confirmed by additionally performing the thermogravimetric analysis of $CaCO_3$ [JIS K 8617:1996] and $Ca(OH)_2$ [JIS K 8575:1994] at the same conditions. At the same time, the thermogravimetric analysis of fine aggregates was carried out, and no presence of $CaCO_3$ or $Ca(OH)_2$ was determined.

Moreover, as mentioned above, although the contents of $CaCO_3$ and $Ca(OH)_2$ were quantitatively estimated, only the contents of $CaCO_3$ were targeted in this paper.

3.2 Estimation of concentration gradient and integrated amount of $CaCO_3$ in mortar test specimens

Figure 5 shows the in-depth concentration gradients of $CaCO_3$ for the 6 month accelerated neutralization processing test specimens. Figure 6 shows the results of the 24 month accelerated neutralization processing test specimens. As can be seen from Fig. 5, neutralization does not reach the deep region of the 6 month processing test specimens. Thus, for the 6 month neutralization processing mortar specimens, the thermogravimetric analysis of the sections from mortar 1 to mortar 4 (which is located at a region 40-50 mm from the surface, as shown in Fig. 3) were carried out. However, for mortar test specimens 1-1, as only a small amount of $CaCO_3$ was observed as shown in Fig. 4, a thermogravimetric analysis of the sections from mortar 1 to 5 was performed.

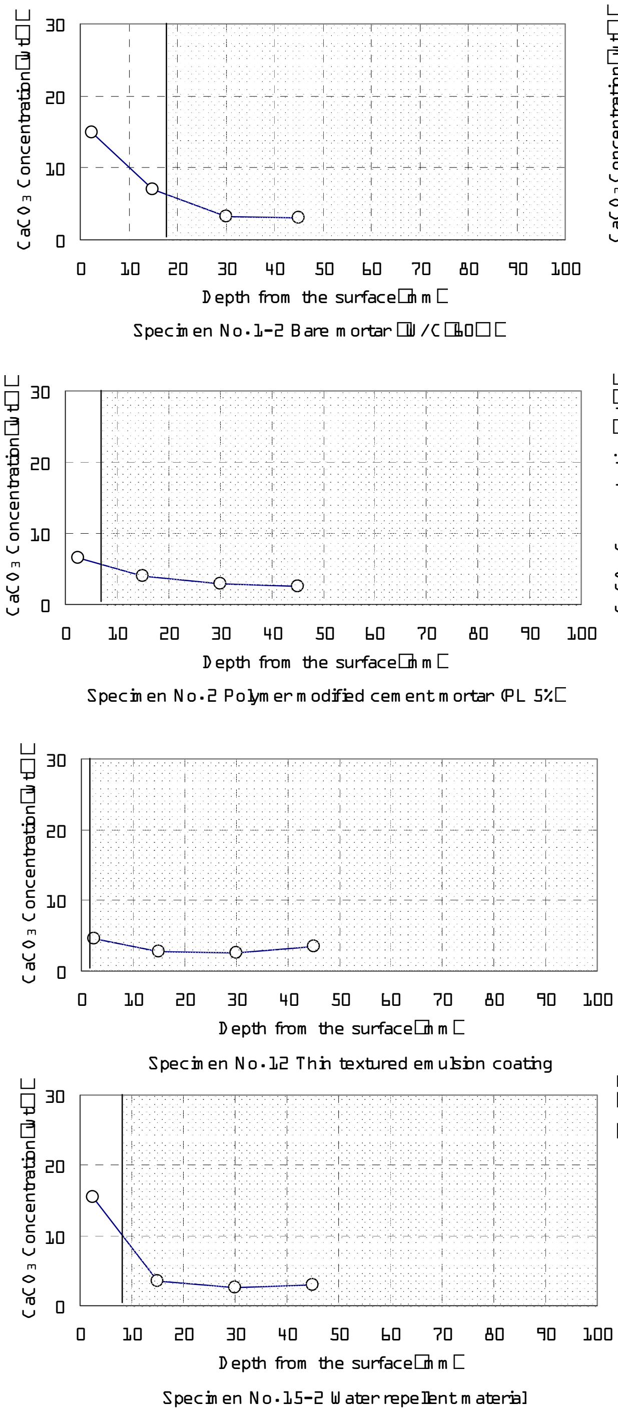
Also, the amount of $CaCO_3$ for each mortar test specimen is indicated at each depth from the surface, based on the cutting pattern of the mortar test specimens as shown in Fig. 3, e.g., for mortar 1, the amount of $CaCO_3$ at a depth of 2.5 mm, and, for mortar 2, the amount of $CaCO_3$ at a depth of 15 mm. The amounts of $CaCO_3$ are expressed by their ratio to the absolute dry mass of mortar (dried at 110□ for 24 hr) as a percentage.

As clearly shown in Figs. 5 and 6, for mortar test specimens in which neutralization has progressed, the amount of $CaCO_3$ gradually decreases going toward the deeper region from the surface. Moreover, it was found that $CaCO_3$ exists in the un-neutralized region determined by the phenolphthalein method, and that $Ca(OH)_2$ exists in the neutralization region.

Also, Figs. 5 and 6 show that several % of $CaCO_3$ exists even in the deep region far from the surface which is expected not to be neutralized. This can be considered to be an error induced by the sloping of the base line of the thermogravimetric curves. Thus, in the estimation of the integrated amount of $CaCO_3$, if the $CaCO_3$ contents show a certain low value in the deep region, the $CaCO_3$ contents in the region were compensated to be zero.

The integrated amount of $CaCO_3$ described here indicates a measure of the total amount of $CaCO_3$ produced in the accelerated neutralization processing mortar test specimens, corresponding to the areas of the shaded parts in the curves of Figs. 5 and 6. The integrated amounts of $CaCO_3$ were determined by adding the all shaded parts of the plotting paper.

Figures 5 and 6 clearly show that the $CaCO_3$ content apparently increases up to approximately 20 % during the process of neutralization. However, as mentioned above, when the $CaCO_3$ content in the un-neutralized region was corrected to make it zero, the $CaCO_3$ content in the neutralization-completed region was 12 % at maximum.
According to one reference,[5)] the mass of $Ca(OH)_2$ produced by the hydration of standard portland cement is considered theoretically to be 25-29 % of hardened substances produced. In this study, as the cement-aggregate mass ratio is 1: 2.8, the mass fraction of $Ca(OH)_2$ in whole mortar is 6.6-7.6 %. Moreover, considering the mass increment when the entire amount of $Ca(OH)_2$ (74g/mol) changes to $CaCO_3$ (100g/mol) by complete neutralization, the content of $CaCO_3$ in the mortar can be estimated to be 9-10 %.



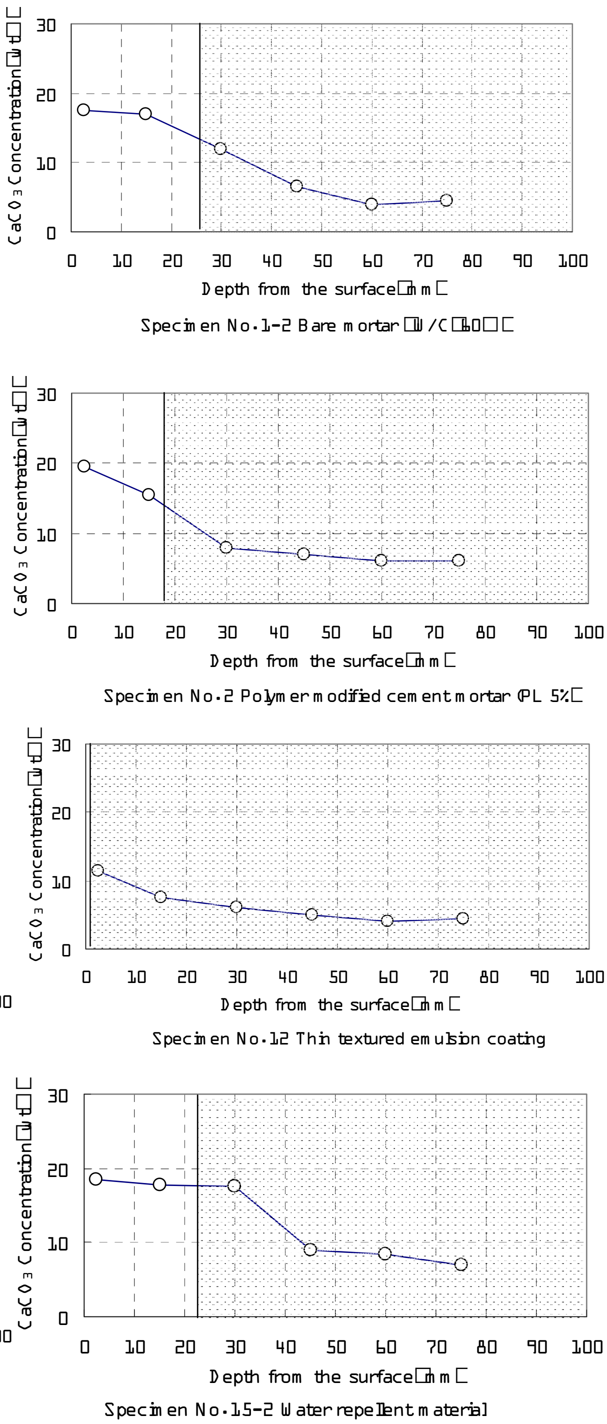


Figure 4. Concentration gradients of $CaCO_3$ (6 month accelerated newtralization)

Figure 5. Concentration gradients of $CaCO_3$ (24 month accelerated newtralization)

Comparing with the 9-10 % $CaCO_3$ content estimated theoretically, the compensated $CaCO_3$ content, 12 % at maximum as mentioned above, although it is a little bit larger, agrees well.

3.3 The relationship between neutralization depth by the phenolphthalein method and the amount of integrated $CaCO_3$ in mortar

Generally, the neutralization depth of concrete is formulated as in Eq.3,

$$C = A(t)^{0.5} \qquad (3)$$

where C is neutralization depth ((mm), t is the time (month), and A is a coefficient of the neutralization rate estimated from neutralization depth [mm/(month)$^{0.5}$].

Assuming that the neutralization rate equation for the neutralization depth determined by the phenolphthalein method is also valid for the integrated amount of $CaCO_3$, the following is obtained.

$$S = B(t)^{0.5} \qquad (4)$$

where S is the integrated content of $CaCO_3$ (%mm), t is the time (months), B is a neutralization rate coefficient estimated from the integrated content of $CaCO_3$ [%mm/(month)$^{0.5}$].

Figure 6 shows the relationship between the neutralization rate coefficient determined from the neutralization depth and that from the integrated amount of $CaCO_3$; both coefficients were found to be strongly correlated with each other, and the former coefficient was approximately 11 times the latter (Y = 11.1X, R =0.96). That is, from such a relation shown in Fig. 6 and Eq. 4, the following equation can be derived,

$S = 11.1\ (\%) \times C, \qquad (5)$

where S is the integrated amount of $CaCO_3$ (% mm), and C is the neutralization depth (mm).

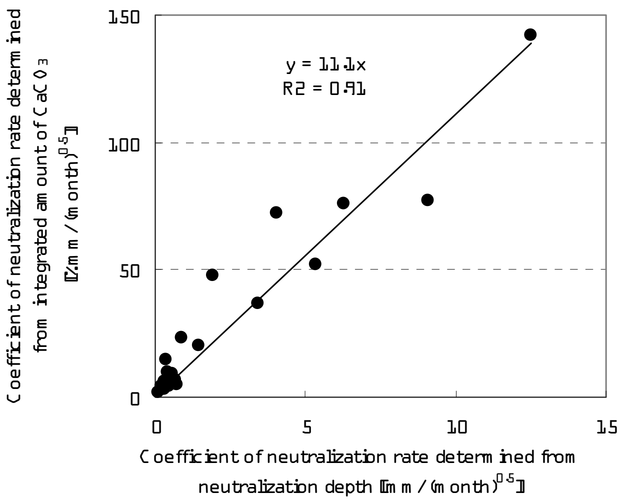


Figure 6. Relation between coefficients of neutralization rate determined from neutralization depth and integrated amounts of CaCO3.

As described above, in the region where neutralization is expected to be completed, the $CaCO_3$ content was 12 % in mortar from the thermogravimetric analysis. This value is consistent with the value of 10 % estimated using a report from Kasai [1983]. Additionally, Eq. 5 indicates that the integrated $CaCO_3$ content (% mm) can be obtained by multiplying the neutralization depth by a constant value of 11.1 (%). That is, when assuming that the $CaCO_3$ content is uniformly 11.1 % in the neutralization region determined by the phenolphthalein method, and $CaCO_3$ does not exist in the un-neutralized region, the total amount of $CaCO_3$ estimated was found to well agree with the integrated amount of $CaCO_3$ determined by the thermogravimetric method. Moreover, the constant of 11.1 % is close to the maximum value of $CaCO_3$ content of 12 % in the surface region where neutralization was completed.

In other words, although the produced $CaCO_3$ is distributed actually with a concentration gradient in mortar test specimens, the total amount (integrated $CaCO_3$ amount) can be approximated by the amount of $CaCO_3$ calculated by assuming that the specimens are completely neutralized in the region above the neutralization depth determined by the phenolphthalein method, and are not absolutely neutralized in the un-neutralized region.

Figures 4 and 5 show that the neutralization depth determined by the phenolphthalein method corresponds approximately to a depth at the intermediate value of the $CaCO_3$ contents in each curve.

Also, the conditions for the accelerated neutralization processing are fixed in this study, and thus it is not clear whether the results obtained are valid or not for other conditions.

4 CONCLUSION

The neutralization suppression performance of various types of finishing materials was evaluated by the neutralization depth determined from the amount of $CaCO_3$ produced, instead of the phenolphthalein method. For this purpose, an accelerated neutralization test was applied to materials with cement mortar as their base substrate, and the amount of $CaCO_3$ produced was analyzed quantitatively by the thermogravimetric method.

The integrated amount of $CaCO_3$ produced in the mortar test specimens by the accelerated neutralization processing was measured, and the relationship between the integrated amount and the neutralization depth determined by the phenolphthalein method was investigated. The integrated amount of $CaCO_3$ was found to be almost the same as the amount of $CaCO_3$ estimated by assuming that all $Ca(OH)_2$ in the neutralization region determined by the phenolphthalein method is changed to $CaCO_3$, and that $Ca(OH)_2$ in the un-neutralized region is not absolutely neutralized. Thus, in this study, it was found that the $\sqrt{t}$ law is also valid for the integrated amount of $CaCO_3$.

5 REFERENCES

JIS K 8617:1996, 'Calsium carbonate', Japanese Standards Association
JIS K 8575:1994, 'Calsium hydroxide', Japanese Standards Association
JIS R 6252:1999, 'Abrasive papers', Japanese Standards Association
Kasai, J. 1983, 'Chemistry of cement part 3 (in Japanese)', *Concrete Journal* published Japan Concrete Institute, Vol. 21, No. 21, pp. 100-106

Modelling of Crack Width in Concrete Structures Due to Expansion of Reinforcement Corrosion

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ABSTRACT

Crack width is a parameter of the most practical significance for the design and assessment of reinforced concrete [RC] structures. Practical experience and observations suggest that corrosion affected reinforced concrete structures are more prone to cracking than other forms of structural deterioration. Determination of corrosion induced crack width is essential to the prediction of the serviceability of corrosion affected RC structures and to the instigation of repairs for the structures. Although considerable research has been undertaken on corrosion induced cracking process, a study of research literature suggests that little work has been carried out directly on corrosion induced crack width both numerically and experimentally, and no analytical model for the crack width has been published to date. Moreover, for corrosion affected RC structures, repairing costs due to concrete cracking and spalling exceed those from other forms of deterioration by a substantial margin. It is therefore imperative to accurately predict the crack width in order to achieve the cost effectiveness in the asset management of RC structures.

This paper attempts to investigate the corrosion induced cracking process in an analytical manner, in which a theoretical model for corrosion induced crack width in RC structures is derived. In this paper, fracture mechanics is employed for the analysis of stress and strain in the concrete surrounding the reinforcing bar. The proposed model is considered the volumetric expansion of the rusting reinforcement through the material moduli. The validity of the derived stress equation is demonstrated by considering the model as a thick-wall cylinder subjected to an internal radial rust pressure distribution. As the corrosion progresses, the induced stress surrounding the rebar and crack evolution are determined. The prediction of the times to onset of cracking of the cover will be based on a combination of an expansive rust pressure analogy and a fracture mechanics approach. A merit of the derived model is that it is directly related to critical factors that affect the corrosion induced cracking process, such as the corrosion rate, concrete geometry and property. The derived model is also verified with both experimental and numerical data obtained from research literature. The model derived in the paper can serve as a useful tool for engineers, operators and asset managers in decision-making regarding the maintenance and repairs of corrosion affected RC structures.

KEYWORDS

Concrete structure, Crack width, Reinforcement corrosion, Serviceability.

1 INTRODUCTION

Corrosion of reinforcing steel in concrete is the predominant causal factor in the premature degradation of reinforced concrete [RC] structures [Broomfield 1997]. Practical experience and observations suggest that, although many RC structures are seen as “badly” deteriorated, characterized by mass concrete cracking and spalling, they are still structurally sound [Dhir & McCarthy 1999]. The reason for this is attributed to the nature of the problem; the corrosion products exert an expansive stress on concrete the tensile strength of which is usually low. It is also partially due to the fact that the safety factors used in structural design for strength are usually larger than those for serviceability since the paramount importance of structural safety. As a result, corrosion affected RC structures are more prone to cracking [than, e.g., loss of strength], incurring considerable costs of repairs and inconvenience to the public due to interruptions. This gives rise to the need for thorough investigation on corrosion induced cracking process in order to achieve cost-effectiveness in maintaining the serviceability of the RC structures.

Considerable research has been undertaken on corrosion induced cracking process, with perhaps more numerical and experimental investigations than analytical ones. Numerical investigations use mainly finite element methods with various models for the growth of corrosion products [i.e., expansive pressure] and concrete behavior once cracked. For example, Dagher & Kulendran [1992] and Pantazopoulou & Papoulia [2001] assume that the cracks in concrete are smeared and employ fracture mechanics to determine the stress in concrete and hence the cracking. Molina *et al*. [1993] model the concrete as linear softening material and the corrosion products as a layer with reduced modulus of elasticity. With this model, the length of the crack evolution in concrete can be determined. Noghabai [1996] and Coronelli [2002] also combine the corrosion-induced pressure [stress] with the pressure produced by bond action in determining the stress in concrete. Ueda *et al*. [1998] use finite element method to examine the factors that affect corrosion induced cracking in concrete and find that the tensile strength and creep of concrete are important factors.

In experimental investigations on corrosion induced cracking in concrete, the corrosion process is usually accelerated by various means so that concrete cracking can be achieved in a relatively short time [Alonso *et al*. 1998; Andrade *et al*. 1993; Liu & Weyers 1998; Francois & Arliguie 1998]. Most of the experiments appear to focus on surface cracking of concrete rather than on direct measurement of crack width over time [Liu & Weyers 1998]. Once experimental results are produced, empirical models can be readily developed for corrosion induced concrete cracking based on mathematical regression, including both deterministic models, such as Alonso *et al*. [1998], Rodriguez *et al*. [1996], and probabilistic models, such as Thoft-Christensen [2001] and Vu & Stewart [2002]. The factors that affect the corrosion induced cracking have also been studied in experiments. For example, Alonso *et al*. [1998] find that, in addition to concrete properties, the corrosion rate and cover to bar diameter ratio are critical factors.

Determination of corrosion induced crack width is essential to the prediction of the serviceability of corrosion affected RC structures and to the instigation of repairs for the structures. Although considerable research has been undertaken on corrosion induced cracking process, a study of research literature [see about references] suggests that little work has been carried out directly on corrosion induced crack width both numerically and experimentally, and no analytical model for the crack width has been published to date. While acknowledging that empirical models exist, e.g., Alonso *et al*. [1998], for widespread application of cracking models to corrosion affected RC structures by a variety of users, it is more desirable to have analytical models [e.g., Bažant 1979; Liu & Weyers 1998]. Moreover, for corrosion affected RC structures, repairing costs due to concrete cracking and spalling exceed those from other forms of deterioration by a substantial margin [Dhir & McCarthy 1999]. It is therefore imperative to accurately predict the crack width in order to achieve the cost effectiveness in the asset management of RC structures. It is in this regard that the present paper attempts to investigate the corrosion induced cracking process in an analytical manner, in which a theoretical model for corrosion induced crack width in RC structures is derived. In this paper, fracture mechanics is employed for the analysis of stress and strain in the concrete surrounding the reinforcing bar. Corrosion induced cracks in concrete are assumed to be smeared and the concrete is considered to be quasi-brittle material. A merit of the derived model is that it is directly related to critical factors that affect the corrosion induced cracking process, such as the corrosion rate, concrete geometry and property. The derived model is also verified with both experimental and numerical data obtained from research literature.

2 CORROSION INDUCED CONCRETE CRACKING

As is well known, concrete with embedded reinforcing steel bars can be modeled as a thick-wall cylinder [Bažant 1979; Tepfers 1979]. This is shown schematically in Fig. 1[a], where D is the diameter of reinforcement bar, d_0 is thickness of the annular layer of concrete pores [i.e., a pore band] at the interface between the reinforcing bar and concrete, and C is the concrete cover. Usually d_0 is constant once concrete has hardened. The inner and outer radii of the thick-wall cylinder are $a = (D+2d_0)/2$ and $b = C + (D+2d_0)/2$. When the reinforcing steel corrodes in concrete, its products [i.e., rusts, mainly ferrous and ferric hydroxides] fill the pore band completely. As the corrosion propagates in concrete, a ring of corrosion products forms, the thickness of which, $d_s(t)$ [Fig. 1[b]], can be determined from [Liu & Weyers 1998]

$$d_s(t) = \frac{W_{rust}(t)}{\pi(D+2d_0)}\left(\frac{1}{\rho_{rust}} - \frac{\alpha_{rust}}{\rho_{st}}\right) \qquad [1]$$

where α_{rust} is a coefficient related to the type of corrosion products, ρ_{rust} is the density of corrosion products, ρ_{st} is the density of the steel and $W_{rust}(t)$ is the mass of corrosion products. Obviously, $W_{rust}(t)$ increases with time and can be determined from [Liu & Weyers 1998]

$$W_{rust}(t) = \left(2\int_0^t 0.105(1/\alpha_{rust})\pi D i_{corr}(t)dt\right)^{1/2} \qquad [2]$$

where $i_{corr}(t)$ is the corrosion current density [in μA/cm^2] which is a measure of corrosion rate.

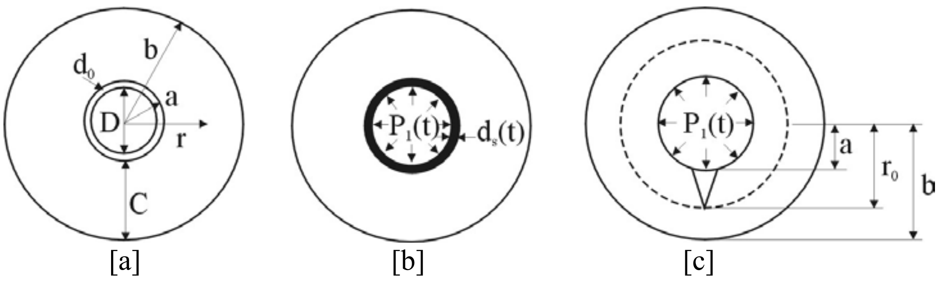

Figure 1. Schematic representation of cracking process.

The growth of the ring of corrosion products [known as rust band] exerts an outward pressure on the concrete at the interface between the rust band and concrete. Under this expansive pressure, the concrete cylinder undergoes three phases in terms of cracking: [i] no cracking; [ii] partially cracked; and [iii] completely cracked. In the phase of no cracking, the concrete cylinder can be considered to be elastic isotropic so that the theory of elasticity can be used to determine the radial stress $\sigma_r(r)$ and tangential stress $\sigma_\theta(r)$ at any point (r) in the cylinder [Timoshenko & Goodier 1970]. From the radial stress $\sigma_r(r)$, the expansive pressure at the interface between the rust band and concrete can be obtained as

$$P_1 = -\sigma_r(a) = \frac{E_{ef} d_s(t)}{a\left(\dfrac{b^2+a^2}{b^2-a^2}+\upsilon_c\right)} \qquad [3]$$

where E_{ef} is the effective elastic modulus of concrete and υ_c is Poisson's ratio of concrete. It may be noted that Equation [3] is the same as that was reported in Liu & Weyers [1998]. From the tangential stress $\sigma_\theta(r)$ at $r = a$, the initial cracking time can be determined by satisfying the condition $\sigma_\theta(a) = f_t$, where f_t is the tensile strength of concrete.

After cracking initiation, the crack in the concrete cylinder propagates along a radial direction and stops arbitrarily at r_0 [which varies between the radii *a* and *b*] to reach a state of self-equilibrium. The crack divides the thick-wall cylinder into 2 co-axial cylinders: inner cracked and outer uncracked ones, as shown in Fig. 1[c]. For the outer uncracked concrete cylinder, the theory of elasticity still applies. For the inner cracked concrete cylinder, let it now be assumed that the cracks are smeared and uniformly distributed circumferentially in the cracked cylinder [Pantazopoulou & Papoulia 2001]. Also let it be assumed that the concrete is a quasi-brittle material. With these assumptions, fracture mechanics can be applied to determine the stress distribution in the cracked cylinder [Kanninen & Popelar 1985]. According to Bažant & Jirasek [2002] and Noll [1972], there exists a residual tangential stiffness in the cracked concrete. Since the residual tangential stiffness at each point on the cracked surface along the radial direction is dependent on the tangential strain of that point, it is a function of the radial co-ordinate r. In view of the lack of knowledge of the residual stiffness of the cracked concrete, it will be assumed in this paper that the residual tangential stiffness is constant along the cracked surface, i.e., on the interval $[a, r_0]$, and represented by αE_{ef}, where α $[<1]$ is tangential stiffness reduction factor. Based on Bažant & Planas [1998], the stiffness reduction factor α is dependent on the average tangential strain $\overline{\varepsilon_\theta}$ over the cracked surface and can be determined as follows [also see Sheng, *et al*. 1991]

$$\alpha = \frac{f_t \exp\left[-\gamma\left(\overline{\varepsilon_\theta} - \overline{\varepsilon_\theta^c}\right)\right]}{E_{ef}\,\overline{\varepsilon_\theta}} \qquad [4]$$

where $\overline{\varepsilon_\theta^c}$ denotes the average tangential cracking strain and γ is a material constant. The cracking in radial direction makes the concrete an anisotropic material locally in the vicinity of cracks. That is, the elastic modulus in the radial direction is different from that in the tangential direction. Li *et al*. [2003] developed a formula for crack width based on this concept of fracture mechanic and the well-known of thick-wall cylinder as shown in Fig. 1. The derivation of crack width formulation is referred to Li *et al*. [2003]. The corrosion induced concrete crack width (w_c) can be expressed as follows

$$w_c = \frac{4\pi d_s(t)}{(1-\upsilon_c)(a/b)^{\sqrt{\alpha}} + (1+\upsilon_c)(b/a)^{\sqrt{\alpha}}} - \frac{2\pi b f_t}{E_{ef}} \qquad [5]$$

In Equation [5], the key variables are the thickness of corrosion products d_s and the stiffness reduction factor α. d_s is directly related to the corrosion rate as shown in Equations [1] and [2]. α is related to concrete geometry and property. Obviously, with the accumulation of corrosion products, the crack width increases. This makes sense both theoretically as shown in Equation [5] and practically as experienced and observed [Andrade *et al*. 1993; Liu & Weyers 1998]. It needs to be noted that, due to the random nature of crack occurrence, there may be more than one crack occurring either simultaneously or within a short period of time. In this case, the assumption that the crack width of all cracks is equal could be made according to Molina *et al*. [1993]. Thus Equation [5] is still applicable but w_c should be divided equally by the number of cracks. In any event, Equation [5] represents the maximum crack width on the surface of concrete. By using the values of basic variables in Table 1 for illustration, the size of a typical crack as a function of time can be determined using Equation [5] and shown in Fig. 2. As can be seen the crack width increases with time as expected. At the time that the concrete cylinder completely cracks, i.e., at time to surface cracking, there is an abrupt increase in crack width, which reflects the assumed quasi-brittle nature of the concrete.

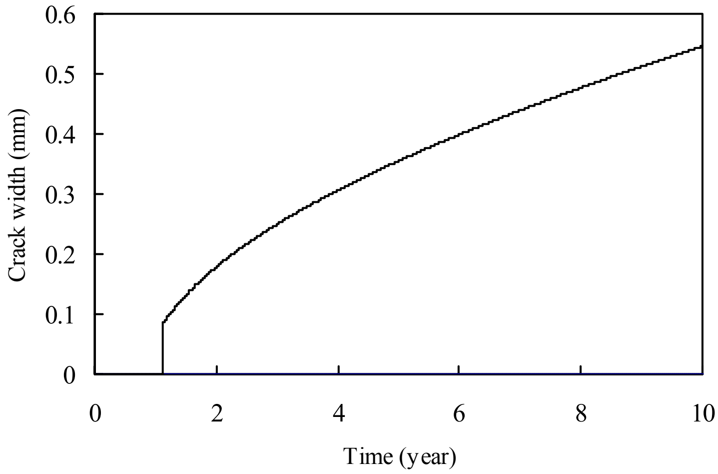

Figure 2. Corrosion induced crack width (w_c).

Symbol	*Values*	*Sources*
C	31 mm	Li (2003)
D	12 mm	Li (2003)
d_0	12.5 μm	Liu & Weyers (1998)
E_c	41.62 GPa	Li (2003)
f_t	5.725 MPa	Li (2003)
i_{corr}	$0.3686 Ln(t)+1.1305$ μA/cm^2	Li (2003)
α_{rust}	0.57	Liu & Weyers (1998)
υ_c	0.18	Liu & Weyers (1998)
ρ_{rust}	3600 kg/m^3	Liu & Weyers (1998)
ρ_{st}	7850 kg/m^3	Liu & Weyers (1998)

Table 1. Values of basic variables used in cracking computation

3 MODEL VERIFICATION

As discussed in previous sections, most of the current research on the corrosion induced cracking process focuses on corrosion induced surface cracking [e.g., Liu & Weyers 1998; Pantazopoulou & Papoulia 2001]. For this reason, data on time to surface cracking were collected from the literature. To investigate the time to surface cracking of concrete structures damaged by corrosion induced internal pressure, Liu & Weyers [1998] carried out a comprehensive experiment on RC slabs subjected to chloride induced corrosion. They observed the surface cracking behavior of corrosion affected RC slabs with various concrete geometry and properties [up to five years]. Their results for time to surface cracking are shown in Fig. 3. Using the same values of their test variables, the calculated time to surface cracking from the proposed model is also shown in Fig. 3. As can be seen, the analytical results are in good agreement with experimental results, with a maximum difference of about 10% for a range of different concrete covers.

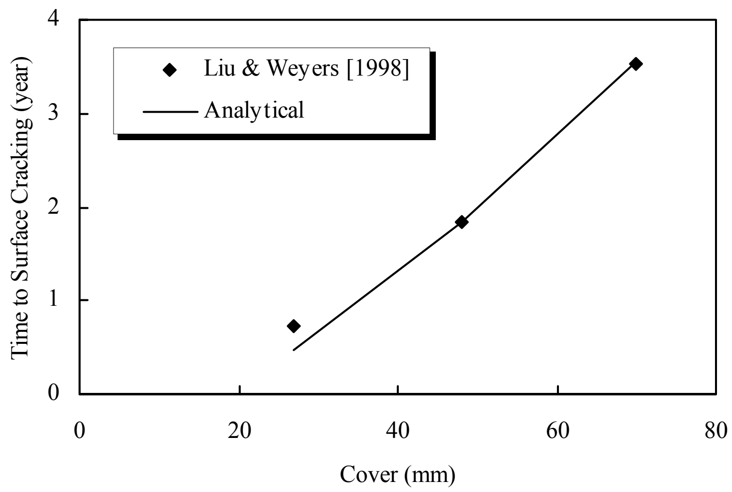

Figure 3. Experimental verification of time to surface cracking.

As noted earlier, Pantazopoulou & Papoulia [2001] developed a numerical algorithm to determine the time to surface cracking of concrete structures subjected to chloride induced corrosion. In their algorithm, the problem of corrosion induced concrete cracking is modeled as a boundary value problem and solved using a finite differences method. Also the cracks in the concrete are assumed to be smeared and the concrete is assumed to be quasi-brittle and anisotropic material. These are the same assumptions adopted in this paper and hence their results can be used for comparison. As shown in Table 2, the difference between the numerical and analytical results for crack width is about 1%.

Model	*Time to surface cracking [in year]*	*Difference [in %]*
Analytical	3.53	-
Experimental (Liu & Weyers 1998)	3.54	0.3
Numerical (Pantazopoulou & Papoulia 2001)	3.50	0.8

Table 2. Comparison of time to surface cracking

A further comparison may be made with experimental results on crack width. It may be appreciated that data on direct measurement of crack widths either from the laboratory or field are scarce. Few data are available for laboratory specimens of a practical size [Vu & Stewart 2002]. In this regard, data reported by Andrade *et al*. [1993] appear to be the only data useable. In their test, the specimens were 15 x 15 x 38 cm. The corrosion was accelerated by imposing electric current [as high as 100 $\mu A/cm^2$] so that the measurable crack width can be achieved within a test period of up to 100 days. Some results of their measured crack width are shown in Fig. 4. Using the same values of their test variables, e.g., corrosion rate, concrete geometry and properties, the calculated crack width is also shown in Fig. 4. As can be seen the analytical results are in reasonable agreement with experimental results. Of interest here is that almost all measured crack widths are smaller than [or equal to] the calculated crack widths, indicating that the derived model indeed gives maximum crack width as implied in Equation [5].

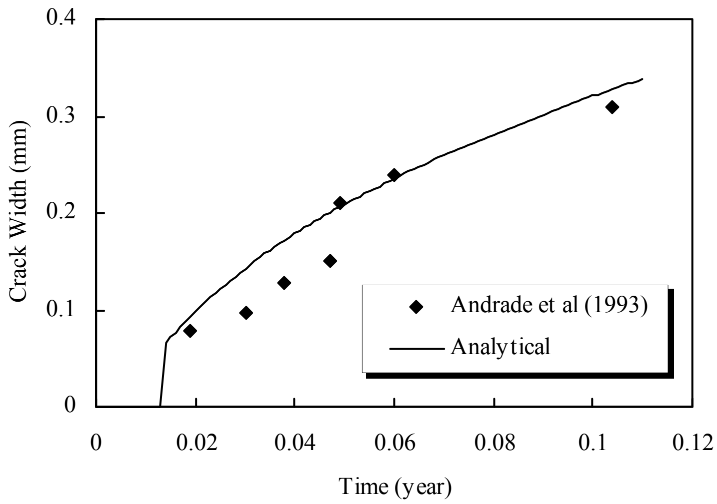

Figure 4. Experimental verification of crack width.

4 CONCLUSIONS

An analytical model for corrosion induced crack width in reinforced concrete structures has been proposed based on the concept of smeared cracks and verified against both experimental and numerical results. The model is directly related to critical factors that affect the corrosion induced cracking process, namely the corrosion rate, the concrete geometry and property. It can be concluded that the model presented in the paper can predict corrosion induced crack width with reasonable accuracy and therefore can serve as a useful tool for engineers, operators and asset managers in decision-making regarding the maintenance and repairs of corrosion affected RC structures. Timely maintenance and repairs have the potential to prolong their service life.

5 ACKNOWLEDGEMENTS

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6 REFERENCES

Andrade, C., Molina, F.J. & Alonso, C. 1993, 'Cover cracking as a function of rebar corrosion: part I – experiment test', *Materials and Structures*, **26**, 453–464.

Alonso, C., Andrade, C., Rodriguez, J. & Diez J.M. 1998, 'Factors controlling cracking of concrete affected by reinforcement corrosion', *Materials and Structures*, **31**[8], 435–441.

Bažant, Z.P. 1979, 'Physical model for steel corrosion in concrete sea structures – theory', *Journal of Structural Division, ASCE*, **105**[ST6], 1137–1153.

Bažant, Z.P. & Planas, J. 1998, *Fracture and Size Effect in Concrete and Other Quasibrittle Materials*, CRC Press.

Bažant, Z.P. & Jirasek, M. 2002, 'Nonlocal integral formulations of plasticity and damage: survey and progress', *Journal of Engineering Mechanics, ASCE*, **128**[11], 1119−1149.

Broomfield, J. 1997, *Corrosion of Steel in Concrete, Understanding, Investigating & Repair*, E & FN Spon, London.

Coronelli, D. 2002, 'Corrosion cracking and bond strength modeling for corroded bars in reinforced concrete', *ACI Structural Journal*, **99**[3], 267–276.

Dagher, H.J. & Kulendran, S. 1992, 'Finite element modeling of corrosion damage in concrete structures', *ACI Structural Journal*, **89**[6], 699–708.

Dhir, R.K. & McCarthy, M.J. (Eds). 1999, *Concrete Durability and Repair Technology,* Thomas Telford, London.

Fenner, R.T. 1989, *Mechanics of Solids*, Blackwell Scientific Publications, Oxford.

Francois, R. & Arliguie, G. 1998, 'Influence of service cracking on reinforcement steel corrosion', *Journal of Materials in Civil Engineering, ASCE*, **10**[1], 14–20.

Kanninen, M.F. & Popelar, C.H. 1985, *Advanced Fracture Mechanics*, Oxford University Press, New York.

Lekhnitskii, S.G. 1963, *Theory of Elasticity of an Anisotropic Elastic Body*, Holden-Day, San Francisco.

Li, C.Q. 2003, 'Life cycle modelling of corrosion affected concrete structures –propagation', *Journal of Structural Engineering, ASCE*, **129**[6], 753–761.

Li, C.Q., Zheng, J.J., Lawanwisut, W. & Chen, Z.J. 2003, 'An analytical model for corrosion induced crack width in reinforced concrete structures', *ACI Structural Journal* (submitted).

Liu, Y. & Weyers, R.E. 1998, 'Modeling the time-to-corrosion cracking in chloride contaminated reinforced concrete structures', *ACI Materials Journal*, **95**[6], 675–681.

Molina, F.J., Alonso, C. & Andrade, C. 1993, 'Cover cracking as a function of rebar corrosion: part 2 – numerical model', *Materials and Structures*, **26**, 532–548.

Noghabai, K. 1996, 'Environmental effects on bond in reinforced concrete structures', Proc. of the 7th Int. Conf. Durability of Building Materials and Components, Stockholm, Sweden, 605–614.

Noll, W. 1972 'A new mathematical theory of simple materials', *Arch. Ration. Mech. Anal.*, **48**, 1–50.

Plaat, O. 1971, *Ordinary Differential Equations*, Holden-Day, San Francisco.

Pantazopoulou, S.J. & Papoulia, K.D. 2001, 'Modeling cover-cracking due to reinforcement corrosion in rc structures', *Journal of Engineering Mechanics, ASCE*, **127**[4], 342–351.

Rodrigues, J., Ortega, L.M., Casal, J. & Diez, J.M. 1996, 'Corrosion of reinforcement and service life of concrete structures', Proc. of the 7th Int. Conf. Durability of Building Materials and Components, Stockholm, Sweden, 117-126.

Sheng, J. M., Wang, Z.Z. & Jiang, J.J. 1991, *Finite Element Methods for Reinforced Concrete and Limit Analysis of Plates and Shells*, Tsinghua University Press, Beijing.

Tepfers, R. 1979, 'Cracking of concrete cover along anchored deformed reinforcing bars", *Magazine of Concrete Research*, **31**[106], 3–12.

Timoshenko, S.P. & Goodier, J.N. 1970, *Theory of Elasticity*, McGraw-Hill Book Company, New York.

Thoft-Christensen, P. 2001, 'Corrosion crack based assessment of the life-cycle reliability of concrete structures', Proc. of Int. Cong. Structural Safety and Reliability, Corotis et al. (eds), Swets & Zeitlinger, 1–7.

Ueda, T., Sato, Y., Kakuta, Y. & Kameya, H. 1998, 'Analytical study on concrete cover cracking due to reinforcement corrosion', Proceedings of an Int. Conf. Concrete Under Severe Condition 2, Tromso, Norway, 678–687.

Vu, K.A.T. & Stewart, M.G. 2002, 'Spatial variability of structural deterioration and service life prediction of reinforced concrete bridges", Proc. of Int. on Bridge Maintenance, Safety and Management - IABMAS 2002, Barcelona, CD-ROM.

Chloride Penetration into Fiber Reinforced Concrete under Static and Cyclic Compressive Loading

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ABSTRACT

The effect of loading on the chloride penetration into plain concrete and fiber reinforced concrete was studied experimentally by using modified NT Build 492 – Non-steady state chloride migration test that include the application of loading on the specimen during the chloride penetration test. Three types of polypropylene fibers with different lengths and shapes were used to make fiber reinforced concrete. Under static compressive loading, plain concrete and three types of fiber reinforced concrete were tested for chloride penetration at different stress ratios. Under cyclic loading, plain concrete and one type of fiber reinforced concrete were tested for chloride penetration under compressive loading after subjecting the specimen to cyclic loading of different stress levels and number of cycles.

The results from the static loading showed that there was a slight reduction in the chloride penetration under low level of compressive stress while an increase in the chloride penetration into concrete was found at higher level of compressive stress. Plain concrete gave the highest increase in chloride penetration at higher compressive loading. Different fiber types had different resistance to chloride penetration under loading. Shorter fibers had better performance against chloride penetration under loading as they were better dispersed in concrete. Chloride penetration increased even more when the specimen was subjected to cyclic loading. For low level cyclic loading, the chloride penetration increased compared to static loading, however there was no increase in chloride penetration with increase in the loading cycles. Under moderate level of cyclic loading, fiber reinforced concrete showed no increase in the chloride penetration while the plain concrete showed a continuous increase with increase in the loading cycles. The chloride penetration rates for plain and fiber reinforced concrete increased with increase in the number of loading cycles when the concrete was subjected to higher level of cyclic loading.

KEYWORDS

Chloride ions, migration, diffusion, loading effect and polypropylene fiber.

1 INTRODUCTION

Concrete structures in marine environment are subjected to chloride penetration that could initiate corrosion of the reinforcement. The corrosion can significantly degrade the structural performance and their lifetime. In order to have a better prediction on the service life of the concrete structures under marine environment, the effect of loading on the chloride penetration into concrete needs to be studied. Loading on the concrete was investigated because it represents the actual condition of concrete structures at service time. During the service period, concrete structures are subjected to its self weight, service load such as traffic load or wind load and also accidental loading such as earthquake or impact load. These loading conditions could change the chloride penetration rate into the concrete structure and consequently change the service life time of the structure.

Inclusion of fibers into concrete has some benefits that include increased energy absorption, fracture toughness, and so on. The combination of polypropylene short fiber with steel reinforcement is expected to improve the behavior of concrete material by increasing its ductility and energy absorption capacity. Moreover, fiber reinforced concrete could perform better to resist chloride penetration under loading as the fibers have a crack bridging effect that could reduce the appearance of microcracks.

2 METHODOLOGY

2.1 Non-steady state migration

The chloride penetration test used in this study is based on the standard of NordTest Build 492 - Non-Steady State Migration Test [Nordtest 1997]. The principle of the test is to subject the concrete to external electrical potential applied across the specimen and to force chloride ions to migrate into the specimen. After certain test duration, the specimen is split and chloride penetration depth could be measured by a colorimetric method. The correlation of this test to the prediction of real structure had been discussed by Tang and Sorensen (2001)

The effect of loading on the chloride penetration was studied by conducting Modified NT Build 492 test that introduces loading at the time of testing, as shown in Fig. 1. The concrete specimen was loaded by using an external frame and then a vacuum pre-treatment procedure was conducted as described in the standard. The non-steady state migration test was then conducted while the specimen was still under load. After the migration test which normally takes 24 hours, the specimen was released from loading and split at the center of the specimen. The chloride penetration depth was measured by the change of color after spraying with silver nitrate solution.

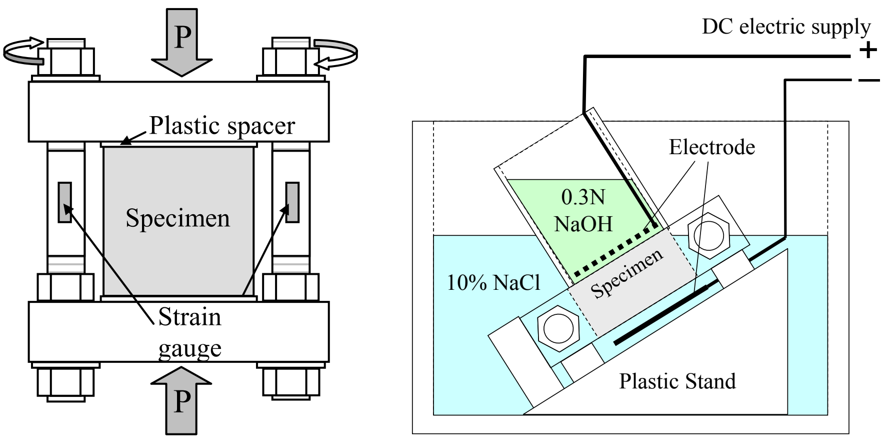


Figure 1 Migration test setup for chloride penetration under compressive loading. Loading was applied on the specimen using external stainless steel frame.

The chloride non-steady state migration coefficient can then be calculated from the equation (1) given in NT Build 492:

$$D_{nssm} = \frac{0.0239(273+T)L}{(U-2)t}\left(x_d - 0.0238\sqrt{\frac{(273+T)L\cdot x}{U-2}}\right) \quad (1)$$

where

D_{nssm}	= diffusion coefficient, × 10^{-12} m^2/s;
T	= temperature of solution, °C;
L	= specimen thickness, mm;
U	= applied potential, V;
t	= test duration, h.; and
x_d	= average penetration depth, mm.

2.2 Microcracks evaluation

The microcracks evaluation conducted in this study was based on the indirect microcracks area measurement proposed by Loo [1992]. The method to measure the crack area was done with the assumption that the change of cross-sectional area of a prismatic concrete specimen under uniaxial compression can be resolved into two parts, i.e. the elastic change of cross sectional area due to Poisson's ratio effects and the dilation due to microcracking. In this method, the increase in the area of the crack per unit cross-sectional area was calculated from the measured longitudinal and transversal strain using a simple formula which involves the elastic Poisson's ratio determined a priori from the strain data. This method gives the advantages as it calculates the crack area for whole area of concrete compared only measuring the crack area from external surface. The increase of crack area per unit cross-sectional area or the specific crack area is given by:

$$\varepsilon_c = 2(\varepsilon_x - \nu\varepsilon_y) \quad (2)$$

where

ε_x	= the transversal strain;
ε_y	= the longitudinal strain; and
ν	= the elastic Poisson's ratio.

2.3 Outline of test specimens

Three types of fiber were used to make fiber reinforced concrete in this study. Figure 2 shows mesh type 12 mm polypropylene fiber (PP12M), mono-filament type 10 mm polypropylene fiber (PP10S) and mono-filament type 30 mm stiffer polypropylene fiber (PP30S) used. Fiber diameters are 0.31 mm (equivalent), 0.23 mm and 1.0 mm for PP12M, PP10S and PP30S, respectively.

Figure 2 Three types of polypropylene fiber used and their designations.

For all cases, the water to cement ratio (w/c), water content and sand by aggregate volume fraction were fixed at 0.5, 175 kg/m3 and 0.52 respectively. The maximum aggregate size used was 12 mm. Addition of superplasticizer and air entrainment agent were used to achieve target slump of 10 cm and air content of 5 to 7%. The mix design of the concrete and physical properties are shown in Table 1.

Concrete type	Fiber type	G_{max} [mm]	V_f [%]	W	C [kg/m^3]	S	G	Slump [cm]	Air [%]	Bulk [kg/m^3]	f_c' [MPa]	D_{nssm} [$\times 10^{-12}$ m^2/s]
NPC1	-	12	0	175	350	927	878	10.0	4.9	2383	42.24	16.712
FRC12M	PP12M		0.1	175	350	926	877	10.2	5.9	2397	39.04	16.935
FRC10S	PP10S		0.5	175	350	921	872	11.0	6.8	2367	35.89	13.070
FRC30S	PP30S		1.0	175	350	913	865	10.3	6.2	2358	37.36	15.532

Table 1 Mix proportion and physical properties of concrete.

The specimens were cured in water for 60 days. After water curing, the prismatic specimens were cut by water-cooled concrete cutter perpendicular to its axis into 50±2 mm thick specimens. All specimens were then placed in room condition for additional 30 days before performing the chloride migration test.

For static compressive loading, migration test was conducted on all of the concrete types. Loading variations performed were 30%, 50% and 80% stress level of the maximum compressive strength.

For cyclic compressive loading, the test was conducted on NPC1 and FRC12M. Loading variations were conducted at 30%, 50% and 70% stress level for 1,000, 10,000 and 100,000 cycles. And then the chloride penetration test was conducted at 30% compressive stress level under static loading. Three replications were conducted for all condition.

3 RESULTS AND DISCUSSION

3.1 Effect of static compressive loading

Chloride penetration into the concrete specimen was measured at 0%, 30%, 50% and 80% of the maximum compressive strength. The normalized result of chloride penetration into plain concrete and three types of fiber reinforced concrete under compressive loading are presented in Fig. 3.

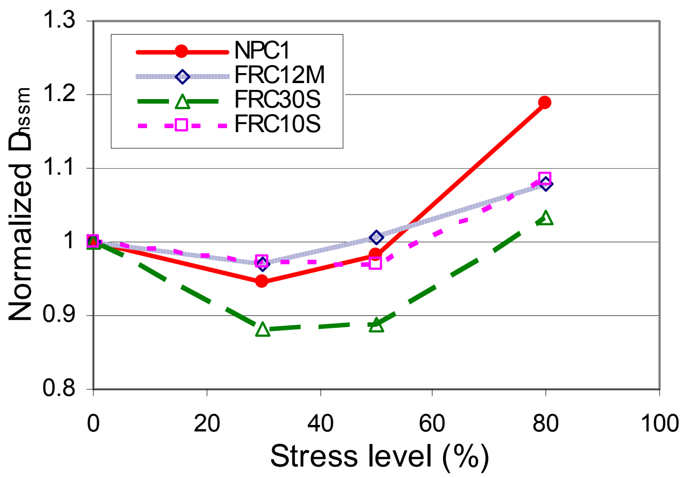


Figure 3 Normalized chloride penetration under compressive loading.

All of the concrete showed a reduced chloride penetration rate for lower stress level. Up to 30% stress level, there is a reduction of chloride penetration compared to non-loading conditions. This condition could be attributed to the increase in the density of concrete due to loading before any microcracks could occur. Similar trend of reduced chloride penetration under lower compressive loading was also reported by other researchers [Heideman 1995, Sugiyama 1995].

At 50% stress level, there was practically no increase for the FRC10S and FRC30S while there was a slight increase for NPC1 and FRC12M. However, from 50% to 80% stress level, there was some increase in the chloride penetration for all types of concrete. The increase is highest for NPC1. FRC30S shows the similar increase of NPC1 at this loading range. However, due to the reduction of the chloride penetration at lower loading, the chloride penetration rate is still lower than plain concrete.

The polypropylene fiber reinforced concrete showed a smaller increase of chloride penetration at loading of 80% stress level. The increase was less than 10% for the polypropylene fiber reinforced concrete, and about 20% for the plain concrete. This result showed that fiber reinforced concrete has better performance when subjected high compressive loading.

Figure 4 shows the specific cracks area of the concrete. It is shown in the figure that there is a difference rate of the increase of crack area with the increase of loading. FRC with shorter fiber (FRC12M and FRC10S) showed a reduced increase of the specific crack area compared to NPC1 and FRC30S. This result shows that the shorter fiber was quite effective to reduce the increase of crack area and thus reducing the increase of chloride penetration at the higher load ratio. NPC1 showed an increase of crack width at 40% stress level while FRC30S showed increase at 50% stress level. Longer fibers, in this case PP30S, have a lower influence of reducing the chloride penetration rate at higher loading condition. The reason could be due to the distribution of the fibers in the concrete matrix. Smaller fibers (PP10S and PP12M) are better distributed in the concrete matrix resulting in better crack bridging properties. Small microcracks could occur in the area between the longer fibers thus the crack bridging effect of longer fiber was lower. In agreement with Lim et al [2000], there are correlation of increase of microcracks area with that of chloride penetration when concrete was under compressive loading. However, the change of chloride penetration should be measured when the concrete was under loading.

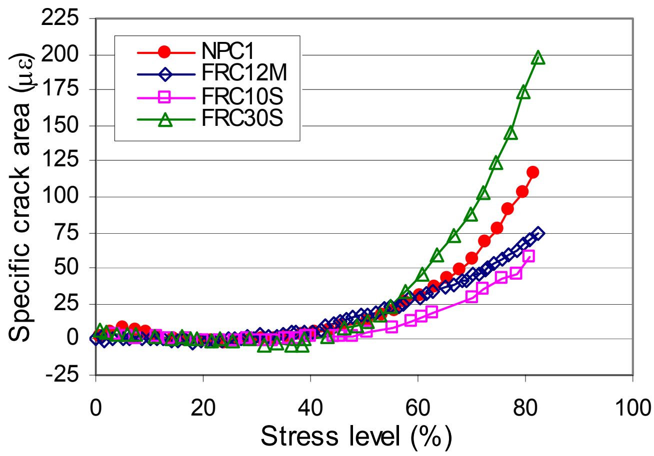


Figure 4 Increase of specific crack area with increase in the stress level.

3.2 Effect of cyclic compressive loading

Figures 5, 6 and 7 show the chloride penetration of NPC1 and FRC12M at cyclic loading of 30%, 50% and 70% stress level at different number of cycles. There was an increase of chloride penetration for both type of concrete under cyclic loading compared to static loading. The increase in chloride penetration shows that there was slight damage to the concrete specimen under cyclic loading compared to the non loading or static loading condition. However, with increase of cycles, the chloride penetration does not appear to be increasing.

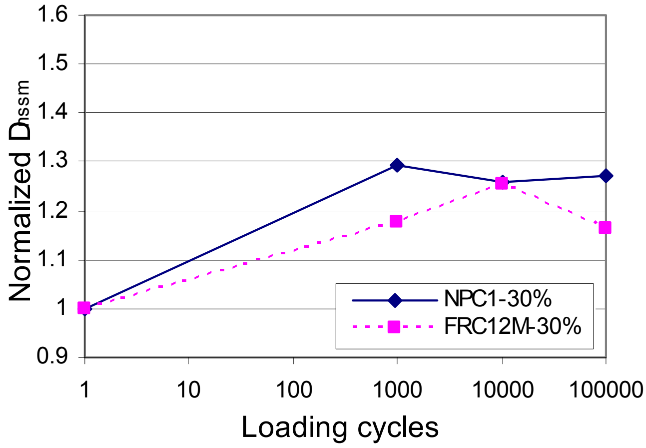


Figure 5 Chloride penetration of 30% cyclic loading.

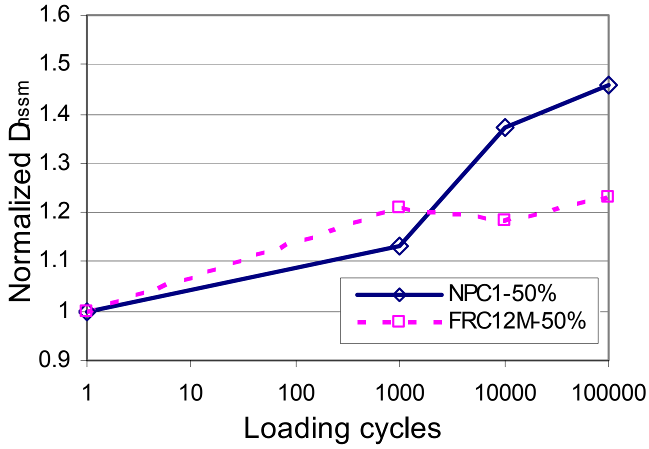


Figure 6 Chloride penetration of 50% cyclic loading.

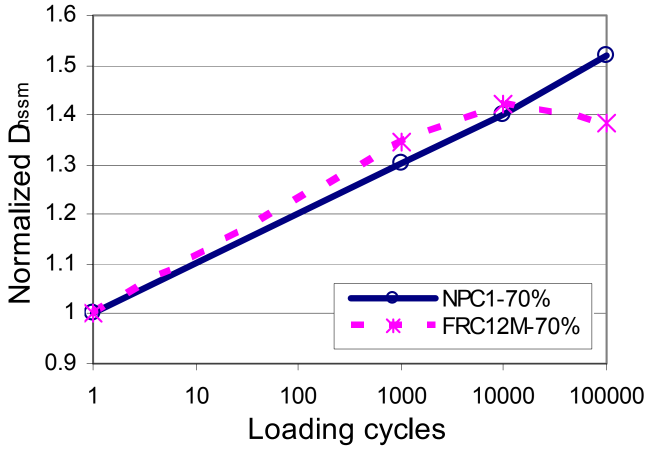


Figure 7 Chloride penetration of 70% cyclic loading.

PP12M polypropylene fiber shows its performance when the concrete was loaded at 50% of maximum compressive strength (Fig. 6). It was shown that the inclusion of fiber was effective to prevent the increase of chloride penetration with the increase of loading cycles. The result shows that the bridging effect of the fiber was effective to prevent the increase of damage in the concrete when the concrete was loaded at 50% stress level.

When the concrete was subjected to cyclic loading of 70%, both NPC1 and FRC12M showed an increase of chloride penetration with the increase of loading cycles (Fig. 7). The concrete showed an increase of chloride penetration logarithmically with the increase of loading cycles.

The measurement of microcrack area was not done for cyclic loading because the change of modulus of elasticity of concrete with increase of cycles. Longitudinal (ε_y) and transversal strain (ε_x) was measured with the increase of number of cycles instead. Increase of the longitudinal with the increase of the number of cycles was due to creep due to loading while the increase of transversal strain was an indication of the cumulative damage such as an increase of crack area.

Figures 8 and 9 show the change of strain with the increase of cyclic number for NPC1 and FRC12M at 50% cyclic loading. It was shown that there is an increase in the transversal strain (ε_x) at 5000 cycles for NPC1. This means that there was an increase of damage in the NPC1 due to cyclic loading at 50% of loading while for FRC12M there was no significant increase of damage in the concrete.

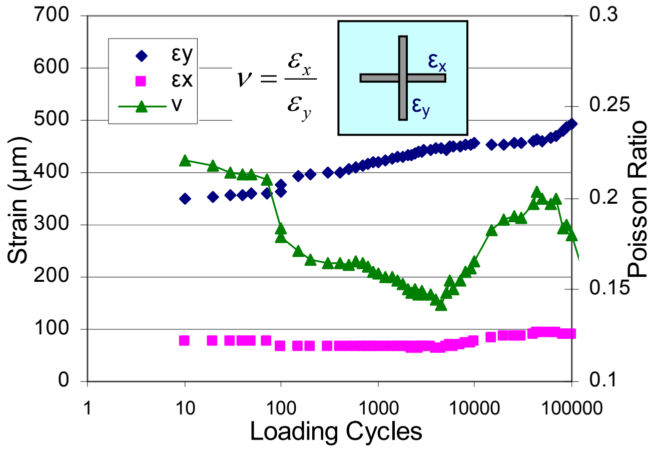


Figure 8 Strain change of 50% cyclic loading NPC1

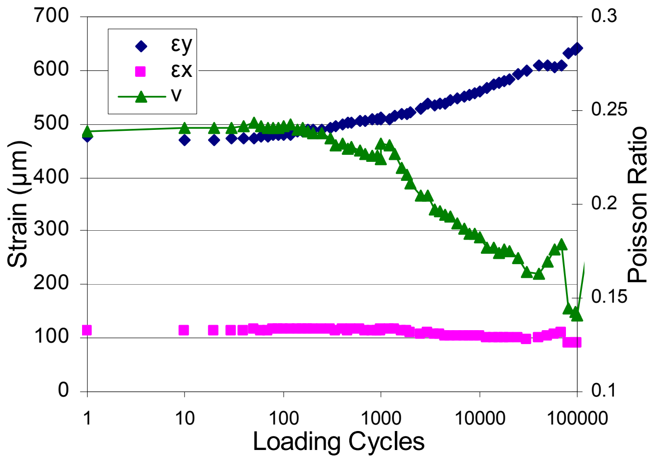


Figure 9 Strain change of 50% cyclic loading FRC12M

Increase of transversal strain was observed also for the cyclic loading of 70% since lower cycles showing that the damage started earlier. When comparing the changes in the transversal strain and the changes in chloride penetration, there are close correlation between them. Increase of the transversal strain could be used as a parameter to see the changes in the chloride penetration.

4 CONCLUSIONS

1. Loading on concrete had a significant effect on the chloride penetration rate into concrete. Under low static compressive loading, there could be a reduction in the chloride penetration depending on the type of concrete. At higher static compressive loading, an increase in chloride penetration was found for all concrete.
2. Increase in the chloride penetration is more pronounce when the concrete is subjected to cyclic compressive loading. Small and insignificant damage in the concrete material could accumulate under cyclic loading and increasing the chloride penetrability.
3. Low level of cyclic loading does not show increasing chloride penetration with the increase of the number of cycles, however when comparing to static or non loading condition, there is still some increase.
4. Moderate stress level loading shows the different behavior of plain concrete and fiber reinforced concrete. Chloride penetration in NPC1 increased with increase in the number of cycles, but there was no increase in the chloride penetration with increase in the number of cycles for FRC12M.
5. At higher stress level, there was an increase in the chloride penetration with the increase of number of cycles showing cumulative damage. NPC1 and FRC12M showed logarithmic increase in the chloride penetration with increase in the number of cycles.
6. The increase of the chloride penetration under static compressive loading was found to be related to the increase of specific microcracks area under compressive loading. With the increase of the specific crack area, the chloride penetration is increased in a similar manner.
7. Changes in the transversal strain in the concrete could be an indication of the changes in chloride penetrability with the increase of cyclic loading. Increase in the transverse strain shows that there is an increase of damage in the concrete and would increase the chloride penetration into concrete.

5 REFERENCES

Antoni, Horiguchi, T., and Saeki, N., "Influence of stress on chloride penetration into fiber reinforced concrete", *Proc. of the Japan Concrete Institute*, v.25, 2003, pp. 779-784.

Antoni, Horiguchi, T., and Saeki, N., "Performance of fiber reinforced concrete against chloride penetration under loading", *Proc. of the Japan Concrete Institute*, v. 26, 2004, pp. 921-926.

Heideman, H, "The effects of stress on chloride penetration into concrete, *Concrete under Severe Conditions: Environment and Loading*, 1995, pp. 1399-1407.

Lee, M. K. and Barr, B. I. G, "An overview of the fatigue behavior of plain and fiber reinforced concrete", *Cement and Concrete Composite*, v. 26, 2004, pp. 299-305.

Lim, C.C., Gowripalan, N., and Sirivivatnon, V., "Microcracking and chloride permeability of concrete under axial compression", *Cement and Concrete Composite*, v. 22, 2000, pp. 353-360.

Loo, Y H, "New method for microcracks evaluation in concrete under compression", *Materials and Structures*, v. 25, 1992, pp. 573-578.

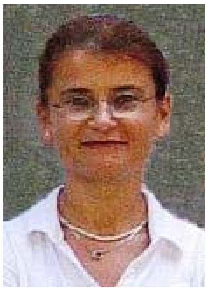
Nordtest NT BUILD 492, "*Chloride migration coefficient from non-steady state migration experiment*", Nordtest, Finland, 1999.

Saito, M. and Ishimori, H., "Chloride permeability of concrete under static and repeated loadings", *Cement and Concrete Research*, v. 25, 1995, pp. 803-808.

Sugiyama, T, Bremner, T W And Holm, T A, Chloride permeability of concrete under compressive stress, *Concrete under Severe Conditions: Environment and Loading*, 1995, pp. 1389-1398.

Tang L and Sørensen H E, Precision of the nordic test methods for measuring the chloride diffusion/migration coefficients of concrete, Materials and Structures, v. 34, 2001, pp.479-485.

Temperature and moisture conditions in materials – Effects on risk for degradation of rendered autoclaved aerated concrete



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ABSTRACT

Temperature and moisture conditions are, in general, the two major factors influencing the long-term performance of external walls made of porous mineral building materials. Degradation of wall components is accelerated by temperature and moisture induced stresses which lead to cracks and in turn a surface more vulnerable to other degradation agents. The degradation rate depends on both the environmental conditions and the material-inherent and component design properties. Extreme and rapid temperature fluctuations as well as moderate diurnal and seasonal temperature cycles cause thermal stresses and strains in the material, resulting in expansion or contraction and eventual deformation such as cracking or fracture. Material properties such as thermal expansion, elasticity and tensile strength determine if cracking occurs either immeadiately when the surface temperature drops below the initial temperature after rapid cooling or after a period of time if alternating or repeated stresses result in creep and fatigue.

In this paper an attempt is made to evaluate the temperature effects on the risk for degradation of external walls made of rendered autoclaved aerated concrete (AAC) based on temperature measurement data and the material properties. The measurement results are obtained from the continuous microenvironment monitoring carried out on a test cabin built on the roof of the Centre for Built Environment building in Gävle, Sweden. A finite element model (FEM) is used to simply calculate the temperature induced stresses in two different cases; with and without creep and relaxation in the material. According to the microenvironment measurement results the test panels attain maximum surface temperatures up to about 60 °C during summer and experience surface temperature fluctuations between day and night up to about 55 °C during winter. Rapid changes in surface temperatures frequently occur particularly throughout late spring and early summer. The preliminary calculated results indicate that the tensile forces built up during cold spells may be sufficient to crack the surface of AAC panels but the risk for fatigue damages due to combined moisture and temperature cycles induced by radiation from the sun seems to be small. Further studies are needed for better knowledge and reliable information on the degradation mechanisms related to temperature by complementary measurements of stress-strain, stress relaxation, creep and fatigue behaviour of AAC panels under different and cyclic temperature loading.

KEYWORDS

Degradation, microclimate, rendered autoclaved aerated concrete, temperature, moisture.

1 INTRODUCTION

Cement renderings and autoclaved aerated concrete (AAC) are brittle materials with modest tensile strengths and show significant movements due to temperature and moisture changes. Besides diurnal and seasonal cycles, rapid temperature and moisture changes in general cause strains in exposed building materials as a result of expansion and contraction, and these cycles may in turn, if the stress level is high enough, result in fatigue damage. Mainly due to radiation from the sun, the surface temperature may change quickly while the bulk temperature of material changes only slowly. The temperature rise on the surface causes material dry-out at and near the surface zone. When the surface cools down again both temperature and moisture changes cause tensile stresses which may result in surface cracking. Facades facing different directions show differences in condition due to being subjected to degradation agents on different levels. Even material surface characteristics like colour, reflectivity, texture and roughness have effects on the dose of agents and contribute to the heat and moisture transfer mechanisms. The degradation rate is dependent on the condition, which is determined by the microclimate, and the microstructure of the surface material. In the long-term performance evaluation of external walls, particularly those made of porous mineral building materials, it is important to consider temperature effects as much as the moisture effects.

Limited research is carried out, particularly on the degradation of inorganic mineral building materials induced by temperature conditions while most studies are based on initial drying shrinkage properties of cement-based materials. This paper focuses, in general, on the actual temperature conditions in rendered AAC walls, and in particular, on the risk for degradation by thermal cracking. Panels are exposed to large temperature variations, mainly due to solar radiation, with relatively low impact of driving rain on the one side of a test cabin. Panels exposed on the opposite side, however, are hardly ever hit by sunlight and receive significantly higher amounts of driving rain. Our hypothesis is that if the moisture is the principal degradation agent, more signs of degradation of the renderings would be seen on the wet side of the test cabin. If, on the other hand, the varying temperatures are more important, the sunny side of the test cabin would display more damage.

This study is a part of the EUREKA project DurAAC investigating the durability, service life and the maintenance intervals for external walls made of rendered AAC. The exposure and test programme started in May 1999, and has been running since then. The industrial partners of the project are Yxhult AB Sweden, Maxit (Optiroc) AB Sweden and Wacker Chemie GmbH Germany. The programme and some earlier results have been presented by Kus *et al.* [2004].

2 BACKGROUND

Cementitious materials have initial stress and strain conditions as a result of creep during hardening and drying of material. This creep is often assumed to be in the order of 0,03-0,06 % (from water saturation to equilibrium at 50 % RH), which is higher than the value of max. tensile strain of concrete 0,02 % [Bergström *et al.* 1970]. In unfavourable conditions even small changes of material strain may thus result in crack formation. In a material that is not restrained in anyway regarding movements (e.g. a precast building element) the initial stress and strain conditions may be minimized by proper production techniques. In most cases, particularly when mechanical loading is considered, there are restraints against movements to some degree. Movements as a result of uneven temperature and moisture changes in the component are more or less restrained. Stresses and strains in the material usually change during normal use under alternating mechanical loads of different kinds but even changes in temperature and moisture may induce changes in stress and strain.

Thermal and moisture stresses occur when the exterior surface of external walls is exposed to daily and/or seasonal temperature and moisture fluctuations while the internal surface is subjected to a controlled indoor air temperature and RH. The external surface of the wall material endures large variations in temperature due to ambient air temperature and solar radiation. The moisture content of the surface layer also changes rapidly due to these temperature fluctuations and possible wetting of the surface due to driving rain. Damage may occur instantaneously if the stresses are higher than the ultimate strength of the material or at a later stage if repeated stresses result in fatigue of the material. Fatigue is resulted by repeated or alternating load cycling and the effect is dependent on both the magnitude of load and the number of load cycles. Assuming that fatigue in AAC is generated in a similar way as in concrete, the fatigue strength at 2×10^6 cycles will be 57-67 % of the static ultimate strength [Ljungkrantz *et al*. 1997]. Creep under sustained stress depends on the loading as a ratio of the yield strength, the time the load is applied, and the temperature [Neville 1995]. Nielsen [1972] quotes values of ε_c ranging from 1 to 185 for concrete and AAC in compression, depending on different load ratios and time (for a temperature of about 20 °C). Knowledge regarding the material properties which affect the risk for temperature and/or moisture induced cracking of AAC is rather limited, but a simplified estimation of the risk is done based on the information available.

Briefly, the rate, magnitude and frequency of the thermal differentials, the capability of the material to accommodate strain variations and the specific boundary conditions should be considered altogether when assessing cracking potentials. Once the micro cracks arise and further propagate, the material becomes more vulnerable to the other degradation agents accelerating the degradation rate. Excessive water ingress from these cracks increases the moisture content locally which may lead to failures such as loss of adhesion of rendering to the substrate and disintegration of the material. Frost problems may arise for the saturated areas when the temperature falls below zero. Consequently, the durability is impaired and the service life shortened. Material properties determine if the material can withstand the effects of external actions.

3 EXPERIMENTAL PROCEDURES

3.1 Location

The city of Gävle is situated on the east coast of Sweden, by the Baltic Sea, at latitude 60,67° N and longitude 17,10° E, and approximately 5 m above sea level. The climate in Gävle is typical of the middle part of Sweden; the average temperature in summer is about 15 °C with peak temperatures of around 27 °C, and in winter -5 °C with absolute min. about -30 °C. Daylight hours are long (up to 19 hours) during summertime and short in wintertime (down to 4 hours). The summers are relatively sunny with more than half of the days in each month having more than 12 hours sunshine. Average yearly precipitation is around 600 mm and average monthly wind speed all year is about 2 m/s. The exposure station is located on the roof of a building at a height of approximately 7 m. According to the microclimate data monitored at the test site for the time period in question, the yearly average air temperature in winter is 0,8 °C and in summer 13,6 °C, the min. being -22,5 °C and the max. 34,5 °C.

3.2 Test Cabin and Test Panels

Within the framework of the DurAAC project the long-term performance of different rendering systems on AAC is investigated. A test cabin is installed at the exposure site of the Centre for Built Environment (BMG) building in Gävle, Sweden. It is oriented with the long sides facing south-west and north-east, respectively. The indoor temperature is regulated by electrical heaters in order to keep a min. temperature of 15 °C over the year. On each long side, there are 12 test panels including unrendered and untreated AAC as reference panel. Unreinforced AAC panels measuring 600 mm × 1200 mm × 150 mm were used as substrates for the rendering systems tested.

3.3 Materials

AAC having a dry density 423 kg/m^3 is employed as the substrate for the rendering systems tested. The general physical and mechanical properties of AAC are given in Table 1. The general coatings characteristics, of which the measurement results are reported in this paper, are given in Table 2. Panel 1 is untreated AAC (reference panel) but panel 9 is rendered with a coloured surface coating.

Table 1. Properties of AAC [Lättbetonghandboken 1993, Aroni *et al.* 1993]

Thermal conductivity (λ) W/mK	0,110
Coefficient of thermal expansion (α)mm/mm K	8×10^{-6}
Thermal capacity (C) kJ/kgK	1,0 - 1,1
Equilibirium moisture at 40 – 80 % RH (%)	2 - 7
Shrinkage at moisture change from 7 to 2 % µS	180
Young's modulus (E) Mpa	1200
Poisson's ratio (ν)	0,15 - 0, 20
Compression strength MPa (at 10 % moisture content by weight)	2,3
Tensile strength MPa	1/6 of compression strength
Bending strength MPa	1/5 of compression strength

Table 2. Test systems

Panel no	*AAC surface impregnation*	*Primer*	*Undercoat*	*Final coat*	*Colour*	*Thickness (mm)*
1	-	-	-	-	Grayish	-
9	Silane–siloxane emulsion	Acrylic/dolomite–calcite with silicon additive	-	Pure acrylic copolymer /dolomite–calcite	Red	< 1

3.4 Measurements

The exposure and the continuous microenvironment monitoring programme at the test cabin started in May 1999 and have been running since then. Microclimate parameters such as air temperature, air relative humidity, driving rain and radiation (UVA+UVB) are monitored continuously. Surface and bulk temperatures and moisture contents of the test panels are also measured continuously. Wetcorr sensors are used to measure the surface moisture and the surface temperature. Copper-constantan thermocouples are used to measure temperatures at different depths in the material. The moisture contents are also measured at the same depths with resistance type nail electrode pairs. The sensors are scanned at 5-minute intervals and the averages of temperature and resistance are stored every hour by means of a data logger/multiplexer device. For the purpose of the present study data obtained from the measurements carried out at 5-minute intervals are collected during specific time periods in order to capture the most realistic conditions.

3.5 Surface inspection

During the test period samples have been drilled out of the panels on a yearly basis for microstructural investigations by means of light optical and scanning electron microscopy. The changes in surface properties of the test panels during weathering have also been inspected visually. During the summer 2004 a special effort was made for a visual inspection of the test panels at both the north-east and south-west facades by means of a stereo microscope with magnification up to ×40.

4 MEASUREMENT RESULTS

4.1 Material temperature

Measurement data collected through May 1999 to August 2004 indicate that the test panels facing south-west are, as might be expected, subjected to higher solar radiation during longer periods than the test panels facing north-east. Temperature variations are thus both faster and greater on the first mentioned panels. Even during winter, the difference between daily minimum and maximum surface temperatures reaches to about 55 °C. Moreover, the material surface undergoes freeze-thaw cycles beside the extreme temperatures.

The diagram in Fig. 1 demonstrates measured surface temperatures for a selected day. Panel surfaces develop much higher temperatures than ambient air temperature when they are exposed to intense direct solar radiation. It is clearly seen that there is a co-variation between the surface temperature and the UV radiation. Rapid variations in surface temperatures occur specifically during which the sky conditions suddenly change between cloudy, rainy and sunny and can be numerous during one day. The absolute max. surface temperatures recorded are 60,5 °C and 53,2 °C for Panels 9 and 1 at the south-west facade, respectively, when the ambient air temperature is 33,1 °C. The absolute min. surface temperature recorded is as low as -22,9 °C when the ambient air temperature is -21,7 °C. In Fig. 2 differences between surface temperature and the temperatures at 5 mm and 25 mm depth from the surface of the material are exemplified for Panel 9SW. At very short intervals, in less than half an hour, the temperature difference can shift from negative to positive or vice versa. Such a repeated rapid and extreme fall (or rise) in temperature, which is generally faster than experienced under normal circumstances, might cause fatigue between the surface zone and the innermost parts of the material which is in a relatively steady state condition. After a sufficient number of temperature fluctuations, the cumulative synergistic effects of creep and fatigue stresses and strains would lead to ageing which may in turn result in failure as cracking or fracture.

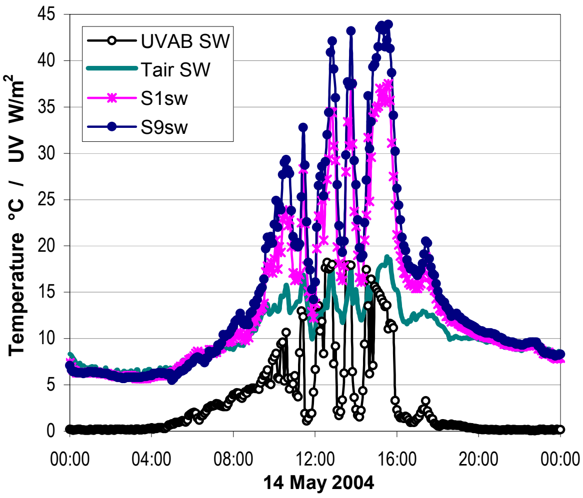

Figure 1. Rapid changes at south-west facade.

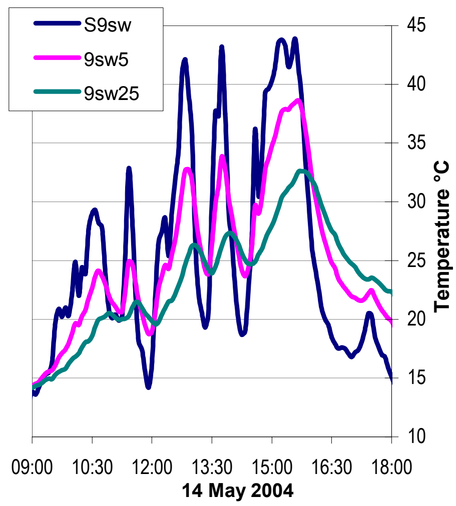

Figure 2. Temperatures of surface and at depths of 5 and 25 mm (Panel 9SW).

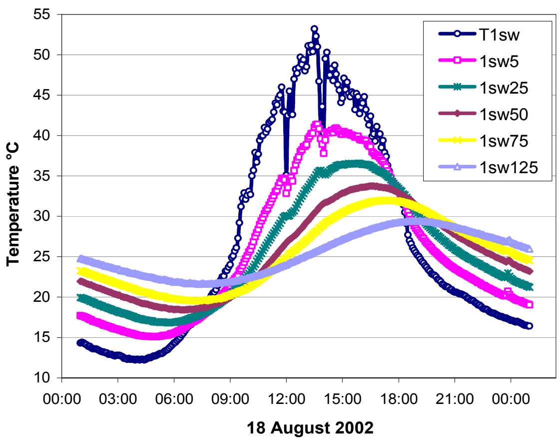

Figure 3. Daily temperature profiles in AAC: 5-min data (Panel 1SW).

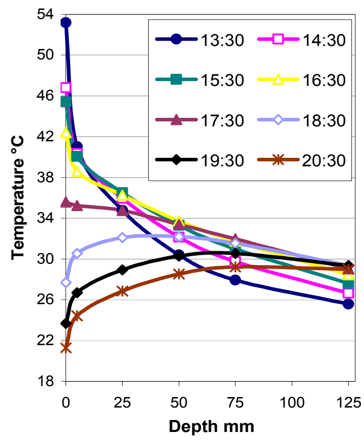

Figure 4. Temperature profiles in AAC measured at specific hours (Panel 1SW).

The greatest diurnal temperature differences and consequently the highest cooling rates occur throughout late spring and early summer. The difference between the daily minimum and maximum temperatures on the panel surfaces of south-west facade reach up to 45,2 °C when the ambient air temperature is min. -4,8 °C and max. 8,6 °C and the surface temperature -7,0 °C and 38,2 °C, respectively. On the panel surfaces of the north-east facade the highest recorded surface temperature is 39,8 °C when the ambient air temperature is 20,3 °C and the min. surface temperature 9,8 °C when the ambient air temperature is 10,4 °C the same day. The highest temperature decrease within 5 minutes for Systems 1 and 9 is 12,1 °C and 11,8 °C, respectively, whereas the highest increase is 12,2 °C and 13,5 °C, respectively. Temperature variations result in large temperature gradients in panel sections. Figures 3 and 4 demonstrate the temperature gradients in AAC (Panel 1SW) measured during 18th August 2002.

4.2 Material moisture

The difference in exposure to driving rain and solar radiation between the facades results in differences in moisture contents. The long-term microenvironment monitoring results indicate clearly that the south-west facade experiences significantly lower amounts of driving rain and thus exhibites relatively low moisture contents. Direct moisture effects are thus of primary concern for the test panels at the north-east facade, particularly for the inorganic test systems without surface treatments, as these panels experience significantly higher amounts of driving rain [Kus *et al.* 2004]. Due to low moisture content freeze-thaw degradation is not considered to be a problem for the rendered panels at the south-west facade. However, stress and strain due to changes in material moisture initiated by the humidity fluctuations in the environment have to be considered for all the panels at both sides.

4.3 Surface inspections

After more than five years of exposure all test panel surfaces, facing either north-east or south-west, are still in quite satisfactory aesthetical condition despite the micro cracks and small discoloration. Very few micro cracks are detected on the test panels of the north-east wall. In general, the coloured surfaces fade more at the south-west facade compared to the ones at the north-east. The micro cracks generally develop around the mineral grains of top rendering coatings, i.e. at the aggregate-paste interphase, and possibly cause the grains to loose and drop after sometime. Randomly distributed individual horizontal and vertical long fine cracks are observed with naked eye on the south-west facade, particularly on the test panels having a smooth plain surface. These cracks probably arise due to thermal stresses. Even though we have not been able to strictly quantify the extent of cracking on the panels, the qualitative evaluation indicates that the panels exposed to temperature variations rather than moisture, display more evidence of deterioration, mainly micro cracking.

5 EVALUATION OF THERMAL- AND MOISTURE INITIATED DEGRADATION

5.1 Risk for thermal cracking

The risk for degradation, particularly thermal cracking, is assessed based on the material properties and the temperature measurements. To simplify the discussion, the material is assumed to be homogeneous, isotropic and linearly elastic (to obey Hook's law). For an infinite plate (in X- and Y-directions, see Fig. 5) with a linear temperature change from one surface to another there is an analytical solution for determining temperature induced normal stress in the heated surface [Timoshenko and Goodier 1970]. According to the analytical solution in Eq. (1), a linear temperature difference of 26 °C results in a compressive stress of about 0,29 MPa on the hotter surface of an AAC component.

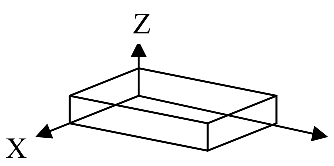

Figure 5. Directions of an infinite plate

$$\sigma = \frac{E \cdot \alpha \cdot \Delta T}{(1-\upsilon)} \qquad (1)$$

where; σ is normal stress in X or Y direction,
E is Young's modulus elasticity,
α is coefficient for thermal expansion,
ΔT is temperature difference between surfaces,
υ is Poisson's ratio of the material.

For a surface, in equilibrium with ambient air temperature and affected by the solar radiation, the temperature profile through the building element is far from linear as shown by the monitoring results

(Figs 3 and 4). To evaluate the temperature-induced stresses and strains, a Finite Element Model (FEM) is used. Silva *et al.* [1999] discuss a similar methodology for estimating temperature induced stresses in walls with ceramic tiles in order to study their performance of adhesion to the substrate. Following simplifications are made in the model;

- Due to symmetry a 2-D model is considered to be satisfactory (a strip through the component is modelled).
- The model uses constant strain elements (linear interpolation of strain).

In the case studied, the moisture changes in the south-west wall are so small that they only result in small variations in stress and strain conditions. Therefore only the stress and strain induced by temperature changes are considered. Calculations are first done for the ideal fully elastic case without relaxation or creep, and then the effects of relaxation and creep are considered. Since the knowledge on creep and relaxation in AAC is limited the discussion is mainly based on qualitative estimations.

5.1.1 Ideal case; neither creep nor relaxation in material

Rise in the surface temperature (by radiation) above the inner temperature of the material results in compressive stress and strain on the surface. When the total temperature difference between surface and the equilibrium temperature inside the element is about 26 °C, the max. stress on the surface is estimated to 0,34 MPa. This may be compared with the analytical solution based on a linear gradient as calculated above; the unlinear gradient gives a somewhat higher stress on the surface. The instantaneous elastic strain (ε) needed, to reach a stress of 0,34 MPa, is about 285 µS. The calculated compressive stress is only 15 % of the yield strength and there should therefore be no risk of cracking neither due to the stress level nor to fatigue. When the surface temperature decreases the stress in the material decreases as well, but the lower surface temperature compared with the inner temperature in the material does not result in tensile stresses on the surface for the idealised case calculated.

During a cold spell the temperature changes in the material, on both the south-west and north-east sides, are not as sudden as those shown in Figs 1 and 2. The temperature gradient is more linear through the material than in the highly dynamic case of radiation effects and the stress may be calculated according to the analytical case. The temperature difference may get as high as -40 °C a few times each winter, which results in a tensile stress of about 0,47 MPa at the surface. This calculated tensile stress is higher than the yield strength of 0,38 MPa, and the surface cracks.

5.1.2 Effects of creep and relaxation

In reality there is creep (and stress relaxation) under constant strain, and it is of great interest to evaluate how this affects the risk for cracking. Creep (ε_c) is often given as a fraction of instantaneous strain (ε) and denoted as fractional creep ($\Phi = \varepsilon_c/\varepsilon$). Due to creep (and relaxation) the stress does not reach the calculated value for the idealized case, but the amount of this effect is not known numerically at present. Tensile stress due to cold spell is somewhat lower than the calculated stress, but it is not known either if it is sufficient to avoid from cracking. During compressive loading due to increased surface temperature the material creeps and the effect of this is pronounced even for short-time loading. The creep results in tensile strain building up when the surface temperature falls again. The fluctuations in surface temperature due to the radiation effect of the sun give repeated tensile stresses during cooling of the surface. The rate of creep alone should be known in order to be able to explain the surface cracking on the sunny side by thermal induced effects. This requires that the tensile stress, caused by creep strain, is higher than the tensile yield strength of 0,38 MPa. To obtain this stress level the tensile strain has to be about 320 µS. The temperature variations of the surface can be as many as 500 per year and the risk for fatigue may be actual when a period of many years is considered. Risk for fatigue requires that the stress level in repeated loading reaches about 65 % of ultimate load, or a tensile stress of about 0,25 MPa. This stress is equivalent to tensile strain of about 210 µS. In both cases mentioned it is apparent that if the tensile stress that cracks the surface depends on creep alone, then fractional creep Φ has to be greater than 0,65. It is therefore most likely that creep is not the sole explanation to the crackings.

5.2 Combined effect of temperature and moisture changes on risk for cracking

The surface dries out during warming up and the shrinkage and creep result in strain. It is well known that shrinkage of AAC is substantial in response to moisture changes at low moisture content (Table 1). The effect of warming up the surface may result in the moisture content changing from an equilibrium at approximately 80 % RH (7 % moisture content, Table 1), and down to approximately 25-30 % RH (2 % moisture content). The resulting shrinkage is then approximately 180 µS (Table 1). The rate of creep is furthermore very much dependent on temperature; at 70 °C the rate is approximately 3,5 times higher than at 21 °C and at –10 °C the rate is about one-half of that at 21 °C [Neville 1995]. The simultaneous effect of changing moisture and temperature increases the rate of creep, but exactly how much is not clear. The effects of shrinkage and creep ratio added require a fractional creep Φ=0,45 to explain the cracking when fatigue is not considered, and Φ=0,15 when expecting a full fatigue effect. The actual reason for cracking is probably somewhere in between. To be able to better evaluate the reasons why the panels are cracking more on the sunny side than the north-eastern side, and to evaluate the risk for such cracks, there is a need for a more detailed study including measurements of material surface conditions.

6 CONCLUSIONS

Measurements show that the temperature changes in AAC panels with surface coatings of different colours are both very fast and can easily be in the order of ± 40 ºC many times each year, but the moisture changes are small due to effective surface treatments. The tensile force built up during cold spells may be sufficient to crack the surface of AAC panels but the risk for thermal fatigue cracking due to repeated loading seems to be small. For a complete study of the risk for thermal induced cracking, which is considered to be real as the panels oriented to south-west show more cracks than those oriented to north-east, a better knowledge regarding creep and relaxation of the material is needed. Future work would include measurements of thermally-induced stresses and strains in the panels and in this way the measurement results could be compared with the calculated values, and in turn, the effect of shrinkage and creep could be estimated.

7 REFERENCES

Aroni, S., de Groot, G.J., Robinson, M.J., Svanholm, G. & Wittman, F.H. (Eds.). 1993, *Autoclaved Aerated Concrete: Properties, Testing and Design.* RILEM, E&FN Spon, London.

Bergström, S.G., Nielsen, A., Ahlgren, L. & Fagerlund, G. 1970, *Allmän kurs i Byggnadsmateriallära, del I*, Byggnadsmateriallära, Tekniska Högskolan i Lund [in Swedish].

Kus, H., Nygren, K., & Norberg, P. 2004, 'In-use performance assessment of rendered autoclaved aerated concrete walls by long-term moisture monitoring', in *Building & Environment* 39 pp 677-687.

Ljungkrantz, Ch., Möller, G. & Peterson, N. (Eds), 1997, *Betonghandbok – Material*, Svensk byggtjänst, Stockholm, [in Swedish].

Lättbetonghandboken 1993, Siporex AB and Yxhult AB, Tryckoffset Göran Lindman AB. Sundbyberg [in Swedish].

Neville, A.M. 1995, *Properties of concrete*, Fourth edition, Longman Group Limited, Essex, England.

Nielsen, A. 1972, *Rheology of building materials*, Document D6:1972, National Swedish Building Research, Stockholm.

Silva, D.A., Roman, L.M.F., Fredel, M.C. & Roman, H.R. 1999, 'Theoretical analysis on the thermal stresses of ceramic tile coating systems', 8DBMC, Eds. M.A. Lacasse & D.J. Vanier, Vol. 1, pp 603-612.

Timoshenko, S.P. & Goodier, J.N. 1970, *Theory of Elasticity*, McGraw-Hill International Ed., NY.

DURABILITY OF FLY ASH CONCRETE AFFECTED WITH PARTICLE SIZES OF FLY ASH AND REPLACEMENT RATIO TO PORTLAND CEMENT

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ABSTRACT

The annual production of coal ash in Japan is increasing rapidly, and is estimated to be over 10 million tonnes in 2007. Although approximately 80% of coal ashes are utilized, most of which are low value applications such as land fill or raw material for cement replacing clay. The use of fly ash as cementitious materials in concrete is still limitted.

This paper describes the test results of a series of experimental studies conducted to make clear of the effects of particle size of fly ash and replacement ratio to portland cement on durability of fly ash concrete.

Specimens made from 28 mixes of fly ash concrete with water binder ratio of 38% to 60% and with replacement ratio of fly ash of 25% to 70% and 5 mixes of portland cement concrete with water cement ratio of 38% to 75% were tested for compressive strengths, drying shrinkage, carbonation and resistance to freezing and thawing. An ordinary portland cement, three types of fly ashes with specific surface area of 5070, 3760 and 1970 cm^2/g, which meet the requirement of Type 1, Type 2 and Type 4 in Japanese Industrial Standard for fly ash respectively, river sand, crushed sandstone, a water reducer and an air entraining agent were used. Target slump was 18 cm and target air conten was 4.5%.

Drying shrinkage of concrete reduced by cement replacement with fly ash. Reduction in drying shrinkage was greater for concrete containing type 2 fly ash and for those with higher replacement ratio of fly ash. Carbonation of fly ash concrete, by acceralating test at 20ºC and 5% of carbon di-oxicide concentration, increased with the increase of replacement ratio of fly ash. Type 1 fly ash indicated higher carbonation than Type 2 and Type 4 fly ashes. The carbonation rate with time was found to be linear with water cement ratio regardless of replacement ratio of fly ash. The carbonation rate of fly ash concrete containing Type 2 and Type 4 fly ashes was found to be same as that of concrete without fly ash with the same water cement ratio, while fly ash concrete with Type 1 fly ash indicated 10% higher carbonation rate. Durability factor after 300 cycles of freezing and thawing decreased with replacement ratio of fly ash, and durability factor of fly ash concrete containing Type1 and Type2 fly ashes with replacement ratio of 25% to 55% were higher than 80%, while those with Type4 fly ash showed lower durability factor less than 80% after 300 cycles. Concrete with 70% replacement of fly ash to cement were not durable regardless of the type of fly ash or specific surface area.

KEYWORDS

Fly ash, Particle size, Replacement ratio, Durability, Carbonation

1. INTRODUCTION

Recently, production of coal ash in Japan is rapidly increasing with the increase of coal-fired power stations [1]. The annual production of coal ash was 9.24 million tonnes in 2002, and has been estimated to be over 10 million tonnes in 2007. Although approximately 80% of coal ashes are utilized mainly in cement industries, only 2.4% of coal ash is used as material for concrete, such as blended cement and fly ash for concrete. On the other hand, the usage of ground granulated blast-furnace slag is increasing for the purpose of portland cement replacement in order to reduce material cost, thermal cracking in mass concrete, and to suppress alkali-silica aggregate reaction, as well as to reduce CO_2 emissions. The obstacles preventing utilization of fly ash in Japan are that the permissible replacement ratio to portland cement with fly ash is limited to 30% in standard specifications for structural concrete, and that only the use of Type 1 and Type 2 fly ash are usually specified, while Japanese Industrial Standard for fly ash specifies Type 1, 2, 3, and 4 fly ashes. For instance, JIS R 5213 Japanese Industrial Standard for Fly Ash Cements limits the portion of fly ash in the cement to 30% by mass. Japanese Architectural Standard Specification JASS 5 for Reinforced Concrete Work [2] limits the permissible portland cement replacement with Type 1 and Type 2 fly ash to 30% by mass, and JASS 5N for Reinforced Concrete Work at Nuclear Power Plants [3] limits to 20% by mass. It is Therefore, necessary to make clarify strength and durability characteristics of high volume fly ash concrete affected with the type of fly ash and to determine the permissible replacement ratio to portland cement with fly ash, so as to expand the usage of fly ash concrete both in quantity and in the area of application.
This paper presents the results of series of experimental studies conducted to clarify the effects of particle size of fly ash and replacement ratio to portland cement with fly ash on strength development, drying shrinkage, carbonation and freezing-thawing resistance.

2. SCOPE OF EXPERIMENT

Five mixture of ordinary portland cement (OPC) concrete with the water-cement ratio of 38 to 75%, twenty-eight mixture of fly ash concrete with the water-binder ratio of 38% to 60%, and with 25 to 70% portland cement replacement ratio with three types of fly ashes were tested for compressive strength, drying shringkage, carbonation and freezing and thawing resistance.

3. MATERIALS

Type 1 fly ash (FA1), Type 2 fly ash (FA2) and Type 4 fly ash (FA4) , from a coal-fired power plant were used. Each fly ash satisfied the requirements specified in JIS A 6201 Japnanese Industrial Standard for Fly Ash for Use in Condrete. Chemical and physical characteristics of fly ashes used are shown in Table 1. Scanning electron micrograph of fly ashes are shown in Fig.1. Difference among three types of fly ashes is mainly the difference in particle size, and the specific surface area of FA1 is 5070 cm^2/g, FA2 is 3760 cm^2/g and FA4 is 1970 cm^2/g.
An ordinary portland cement with density of 3.16 g/cm^3, specific surface area of 3280 cm^2/g and 28 days compressive strength of 62.0 N/mm^2 is used. A crushed sandstone from Oume and a river sand from Oui-River were used. Physical properties of aggregates are shown in Table 2. A water reducer of lignosulphonic acid and an air-entraining agent of anionic sulfactant were used.

4. MIXTURE PROPORTIONS AND DOSAGE OF AE AGENT

Mixture proportions of concretes with and without fly ash are shown in Table 3. The target slump was 18cm, and the target air content was 4.5%.
Water content of FA1 and FA2 concrete decreased with the increase of replacement ratio of fly ash, while water content of FA4 concrete increaseed with the increase of fly ash replacement ratio. Dosage

of air-entraining agent increased linearly with the increase of replacement ratio of fly ash, and dosage of air-entraining agent was found to be higher for FA1 with higher specific surface area than for FA2 and FA4, which can be due to adsorption of air-entraining agent to particles of fly ash.

5. EXPERMENTAL PROCEDURES

5.1 Fablication and curing of test specimens

Cylinders, 100 mm in diameter, 200 mm in height for compression tests were cast in lightgauge metal moulds in three layers by rodding, and were stripped of moulds 24 hours after casting and were cured in water at 20ºC until testing ages.

Prisms 100 mm by 100 mm and 400 mm long were used for drying shrinkage tests, accelerating carbonation tests and freezing and thawing tests, and prisms were cast in two layers by rodding. Prisms were stripped of moulds 24 hours after casting.Then, prisms for drying shrinkage tests were cured in water at 20ºC for 6 days, prisms for accelerating carbonation tests were cured in water at 20ºC until 28 days old and stored at air dried condition at 20ºC for 28 days old, and prisms for freezing and thawing tests were cured in water at 20ºC until 28 days old.

5.2 Test method for hardned concrete

Compressive strength tests. Cylinders were tested at the age of 1, 4, 8, 13 and 26 weeks at wet condition.

Drying shrinkage tests. Prisms taken from water at 7 days old, and were stored at 20ºC and 60% R.H., and were messured for length change and weight loss for 26 weeks.

Accelerating carbonation tests. Prisms, moist cured for 28days and air dried for 28 days, were stored in accelerating carbonation chanber at 20ºC, 60% R.H. and 5% CO_2 concentration. Carbonation depth was determined by phenolephthalein test on splitted surface.

Freezing and thawing tests. Prisms were sujected to Freezing and thawing cycles at the age 28 days according to JIS A 1148-2001, similar to ASTM C 666-A, Durability factor.

6. TEST RESULTS AND DISCUSSIONS

6.1 Compressive strength

Table 3 shows test results of fresh concrete and compressive strength tested on specimens cured in water at 20ºC. Fig.2 shows the strength development of fly ash concrete containing FA1, FA2 and FA4 fly ash. Fig.3 shows the effects of particle size with fly ash and replacement ratio on 13 week compressive strength. Strength development of FA4 concrete is slower than those of FA1 and FA2 concrete. Reduction in strength of fly ash concrete with the increase in replacement ratio of fly ash, despite the type of fly ash, or particle size of fly ash. However, FA1 and FA2 concrete with 25% replacement ratio with fly ash showed approximately the same strength than OPC concrete at 13 weeks. FA4 concrete showed slower strength gain than FA1 and FA2. As shown in Fig.3, conpressive strength at 13 weeks was found to be increased with the increase in specific surface area of fly ash.

Fig.4 shows the effects of binder water ratio and replacement ratio on 13 week compressive strength of fly ash concrete containg FA2. Fig.5 shows the relationship between cement water ratio and 13 week compressive strength. Compressive strength were found to be increased with the increase of binder water ratio. Compressive strength at 13 weeks of fly ash concrete was almost in proportion to cement water ratio regardless of specific surface area with fly ash and replacement ratio.

6.2 Drying shrinkage

Fig.6 shows the effects of particle size with fly ash and replacement ratio on unit water conten, drying shrinkage and weight loss of fly ash concrete. Fig.7 shows the effects of specific surface area of fly

ash on drying shrinkage. Fly ash concrete indicated considerably lower drying shrinkage than OPC concrete, depending on specific surface area with fly ash and replacement ratio. FA2 concrete showed the smallest drying shrinkage and weight loss. Drying shrinkage of FA2 concrete decreased with the increase in replacement ratio of fly ash. Drying shrinkage of OPC concrete increased with the increase of water content, and higher drying shrinkage of FA4 concrete associated with higher water content.
Honda et al. 2001 [4] showed that drying shrinkage of fly ash concrete with specific surface area of 6000 cm^2/g increased by about 20% compared with concrete of specific surface area of 3000 cm^2/g, by the condition of water binder ratio 35%, 40%, 45% and replacement ratio of fly ash 50%, 60%, 70%. In this paper, drying shrinkage of FA1 concrete increased by about 20~30% compared with FA2 concrete. However, higher drying shrinkage of FA1 concrete can be due to carbonation shrinkage, as Neville discussed in 1981[5].

6.3 Carbonation

With conventional carbonation rate equations, the depth of carbonation belowed concrete's surface is expressed by the equation given below; that is, carbonation depth is roughly proportional to the square root of elapsed time, according to Hamada 1969 [6].

$C = A \sqrt{t}$ Where, C =carbonation depth , t =elapsed time, and A=carbonation rate.

Fig.8 shows the test results of accelerated carbonation of OPC concrete and fly ash concrete, and Fig.9 shows the effects of particle size and replacement ratio of fly ash on carbonation rate. When A is mixed, Carbonation of fly ash concrete increased with the decrease in unit cement content and and the consumption of $Ca(OH)_2$ by the pozzolanic activity. In this paper, carbonation rate was found to be increased rapidly with the increase in replacement ratio of fly ash, and to be increased slightly with the increase in specific surface area of fly ash.
Fig.10 shows the relationship between water binder ratio and carbonation rate of FA2 concrete. Carbonation rate increased linearly with the increase in water binder ratio.
Fig. 11 shows the relationship between water cement ratio and carbonation rate. Relation between water cement ratio and carbonation rate was found to be the same for OPC concrete and FA2 and FA4 fly ash concrete, while FA1 fly ash concrete showed slightly higher carbonation rate for the same water ratio than those of the other concretes. Carbonation rate can be considered roughly proportional to water cement ratio regardless of particle size and replacement ratio of fly ash, and the same result was provided as Meyer 1968 [7] and Wada et al. 1998 [8].
Fig.12 shows the relationship between 4 week compressive stength and carbonation rate. Carbonation rate of concrete with and without fly ash decreased exponentially with the increase in compressive strength, for instance Kokubo et al. discussed in 1989 [9]. In this paper, Fly ash concrete showed greater carbonation rate than OPC concrete with the same compressive strength. Increase in carbonation rate of fly ash concrete was found to be greater for fly ash with higher specific surface area.
Fig. 13 shows the relationship between unit fly ash content and carbonation rate. Carbonation rate increased exponentially with the increase in unit fly ash content. Increase in carbonation of fly ash concrete can be due to decrease in formation of $Ca(OH)_2$ caused by the decrease in unit portland cement content and due to the decrease in $Ca(OH)_2$ caused by the consumption in pozzolanic activity of fly ash. Higher increase in carbonation of FA1 concrete also can be explained by the higher pozzolanicity of FA1 with finer particle size and higher strength activity.

6.4 Resistance of freezing and thawing

Fig.14 shows the durability factor of air-entrained concrete containing OPC, FA1, FA2 and FA4 after 300 cycles of freezing and thawing. Durability factor of FA1 and FA2 concrete with 25 to 55% fly ash replacement was found to be higher than 80%, while FA4 concrete showed smaller durability factor less than 80% after 300 cycles. Fly ash concrete with 70% replacement ratio was not found to be durable and collapsed before 300 cycles regardless of the type of fly ash or specific surface area.

Table 1. Chemical analysis and physical characteristics of fly ash

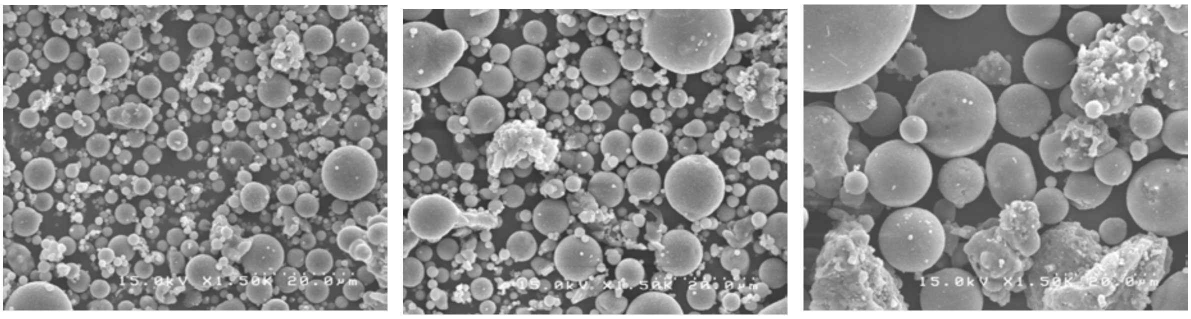
	Chemical analysis							Physical characteristics			Percent flow of mortar (%)	Strength activity of mortar (%)	
									Fineness				
	SiO_2 (%)	SO_3 (%)	CaO (%)	MgO (%)	Ka_2O (%)	Na_2O (%)	Loss on ignision (%)	Density (g/cm^3)	Retained on 45μm sieve (%)	Specific surface area (cm^2/g)		28 days	91 days
FA 1	60.0	0.2	1.2	1.8	0.7	0.0	0.9	2.39	0.1	5070	114	93	111
FA 2	59.9	0.2	1.3	0.6	0.7	0.0	0.9	2.29	7.1	3760	107	84	103
FA 4	57.4	0.2	1.6	0.5	0.7	0.0	1.7	2.15	12.5	1970	104	74	86

Table 2. Physical properties of aggregate

	Type of aggregate	Maximum size (mm)	Density in oven-dry condition (kg/m^3)	Absorption (%)	Bulk density (kg/m^3)	Fineness Modulus
Coase aggregate	Crushed Sandstone	20	2.70	0.51	1.58	6.69
Fine aggregate	River sand	2.5	2.61	1.08	1.81	2.87

Table 3. Mixture proportion and properties of fresh and hardned concrete

	FA/(C+FA) (%)	W/(C+FA) (%)	W/C (%)	Mixture proportion (kg/m^3)					Fresh concrete		Compressive strength (N/mm^2) (Moist cured)			
				W	C	FA	S	G	Slump (cm)	Air (%)	7days	28days	91days	182days
OPC	0	-	43	178	414	-	762	971	17.5	5.0	39.9	48.8	56.3	57.9
		-	50	174	348	-	827		18.5	4.9	35.1	38.9	49.0	52.0
		-	60	170	283	-	892		17.5	4.6	25.6	30.5	38.6	38.5
		-	75	175	233	-	936	955	18.5	4.7	16.1	21.9	27.4	27.2
FA 1	25	43	57	176	307	102	743	971	17.5	4.7	28.8	43.1	56.6	62.8
		50	67	165	248	82	844		18.0	3.2	22.6	34.8	46.3	51.4
	40	43	72	174	239	159	751		17.0	4.4	21.0	33.5	45.0	49.8
		50	83	163	196	130	840		16.5	4.3	16.9	28.9	38.9	44.4
	55	43	96	167	175	214	770	955	18.0	4.0	11.3	19.3	26.2	32.3
		50	111	166	149	183	828		16.5	5.4	9.6	17.8	23.6	32.2
	70	43	143	168	115	269	765		19.0	5.0	4.7	9.5	14.4	22.5
		50	167	164	98	230	824		16.5	4.6	3.5	6.9	11.6	16.8
FA 2	25	43	57	165	288	96	791	971	18.0	4.3	31.2	37.9	54.8	60.8
		50	67	165	248	82	840		18.5	4.5	23.0	32.8	44.5	49.9
		60	80	161	201	67	907		17.0	4.8	15.9	24.6	34.7	39.6
	40	38	63	170	268	179	698		19.0	4.7	28.0	39.2	53.9	58.2
		43	72	163	227	152	782		18.0	4.4	21.3	31.7	44.3	50.9
		50	83	165	198	132	824		18.0	4.4	15.4	26.0	36.4	41.0
	55	38	84	167	198	242	708	955	19.0	4.9	17.3	27.2	38.4	43.3
		43	96	163	171	208	780		19.0	5.1	12.6	21.3	31.6	42.8
		50	111	167	150	184	815		18.0	4.8	9.3	16.4	25.5	31.5
	70	38	127	177	140	326	633		19.0	4.7	9.2	15.0	22.7	31.8
		43	143	160	112	260	777		18.5	5.4	6.0	9.6	17.3	26.6
		50	167	158	95	221	842		19.0	4.9	3.2	5.6	13.0	20.8
FA 4	25	43	57	176	307	102	731	971	18.5	4.4	31.1	41.0	53.6	57.6
		50	67	172	258	86	802		18.0	3.9	21.9	31.0	40.1	44.2
	40	43	72	180	251	167	687		18.5	5.0	20.8	29.9	40.0	46.0
		50	83	174	209	139	773		18.0	4.6	15.1	22.6	32.6	39.0
	55	43	96	184	195	238	647	955	19.0	5.1	11.2	16.4	26.1	31.8
		50	111	175	158	193	763		18.0	5.3	8.7	14.0	21.3	27.0
	70	43	143	198	138	322	559		18.5	4.7	5.2	9.4	16.4	22.7
		50	167	176	106	246	738		16.5	5.3	3.0	5.4	10.9	18.4



FA 1 20µm **FA 2** 20µm **FA 4** 20µm

Fig.1 Scanning electron micrograph of fly ash (1,500 times)

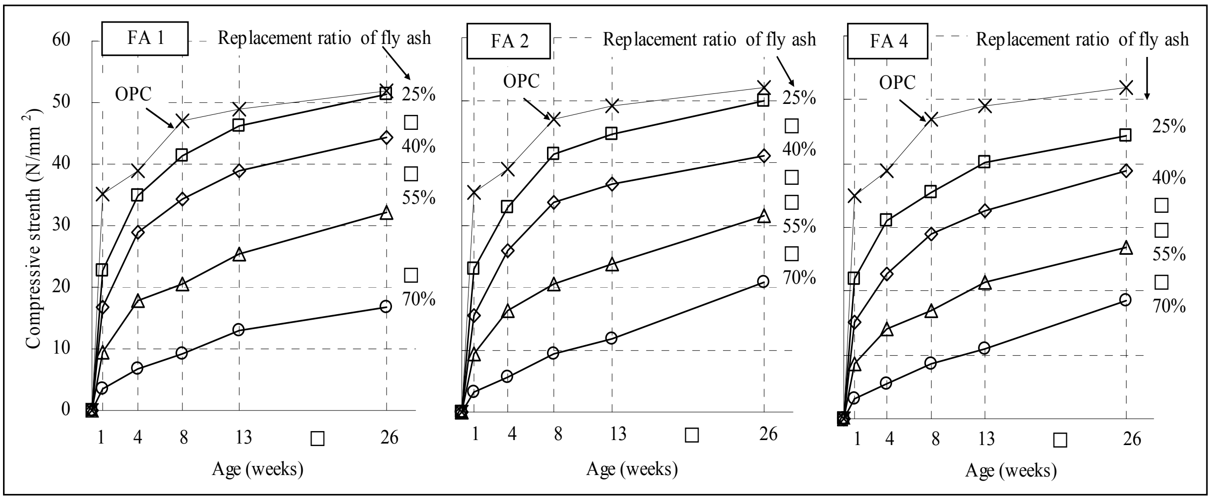
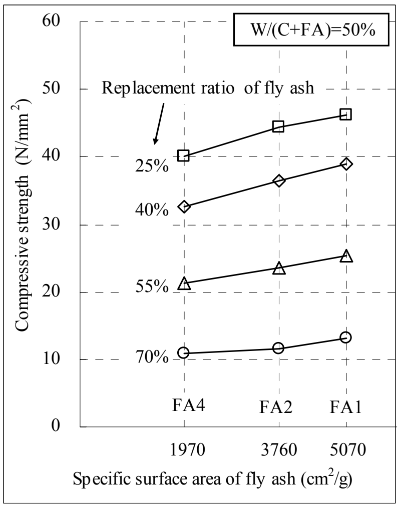
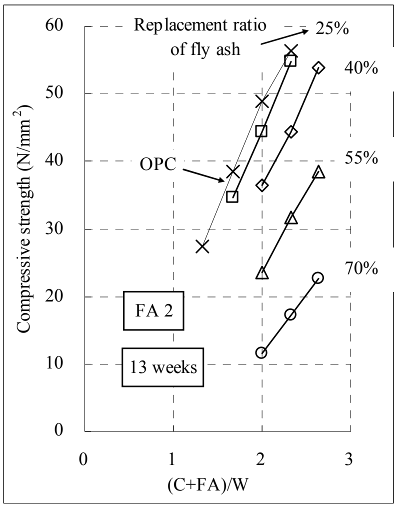


Fig.2 Strength development of fly ash concrete (W/(C+FA)=50%, cured in water at 20°C)





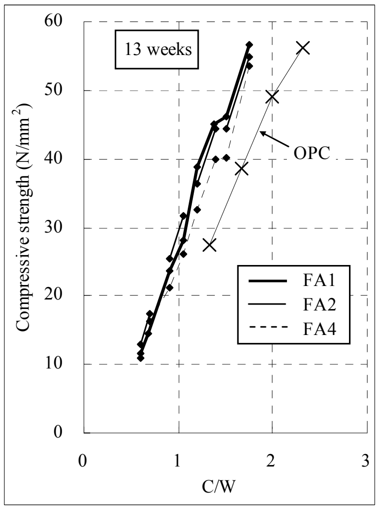


Fig.3 Effect of particle size with fly ash and replcement ratio on 13 week compressive strength

Fig.4 Effect of binder water ratio and replacement ratio on 13 week compressive strength

Fig.5 Relationship between cement water ratio and 13 week compressive strength

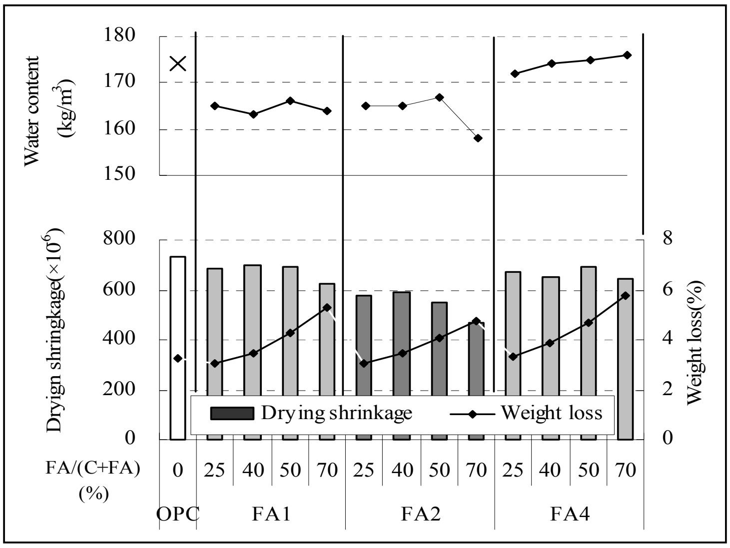


Fig.6 Relationship among water content, drying shrinkage and weight loss at 13 weeks (W/(C+FA)=50%)

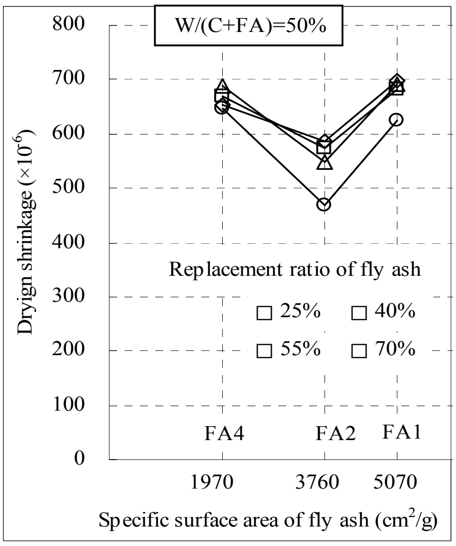


Fig.7 Effect of particle size with fly ash and replcement ratio on drying shrinkage at 13 weeks

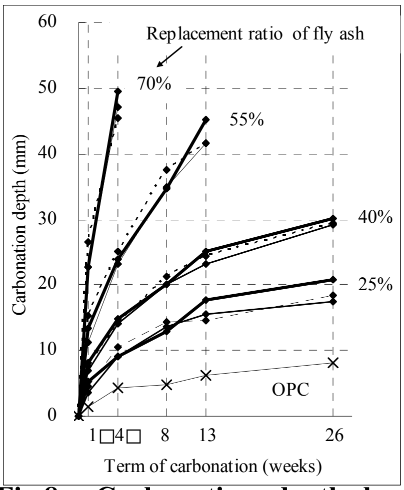


Fig.8 Carbonation depth by accelerating carbonation (W/(C+FA)=50%, at 5% CO_2)

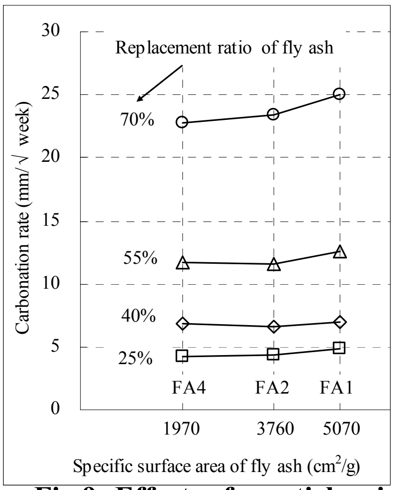


Fig.9 Effect of particle size with fly ash and replcement ratio on carbonation rate

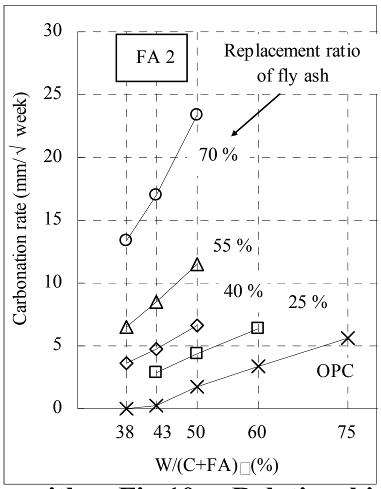
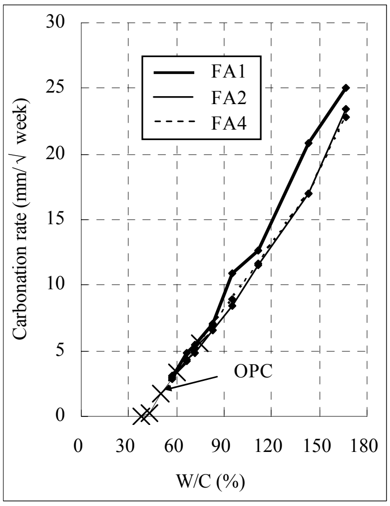
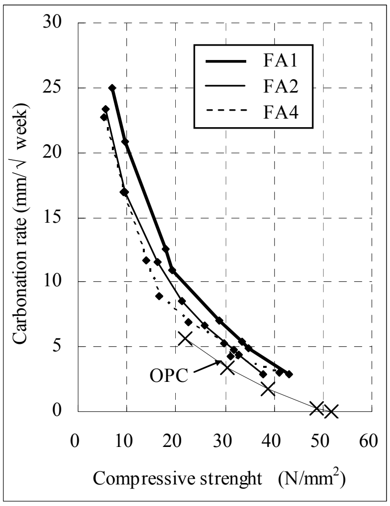


Fig.10 Relationship between water binder ratio and carbonation rate





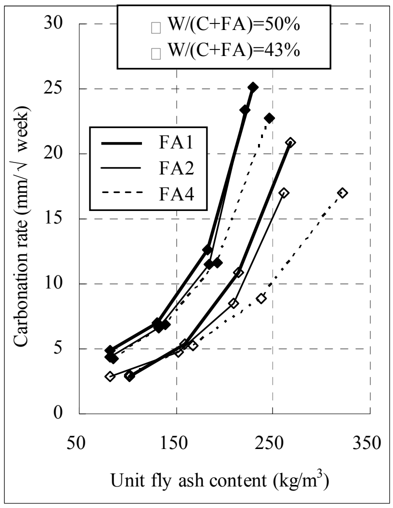


Fig.11 Relationship between water cement ratio and carbonation rate

Fig.12 Relationship between 4 week compressive strenght and carbonation rate

Fig.13 Relationship between unit fly ash content and carbonation rate of concrete

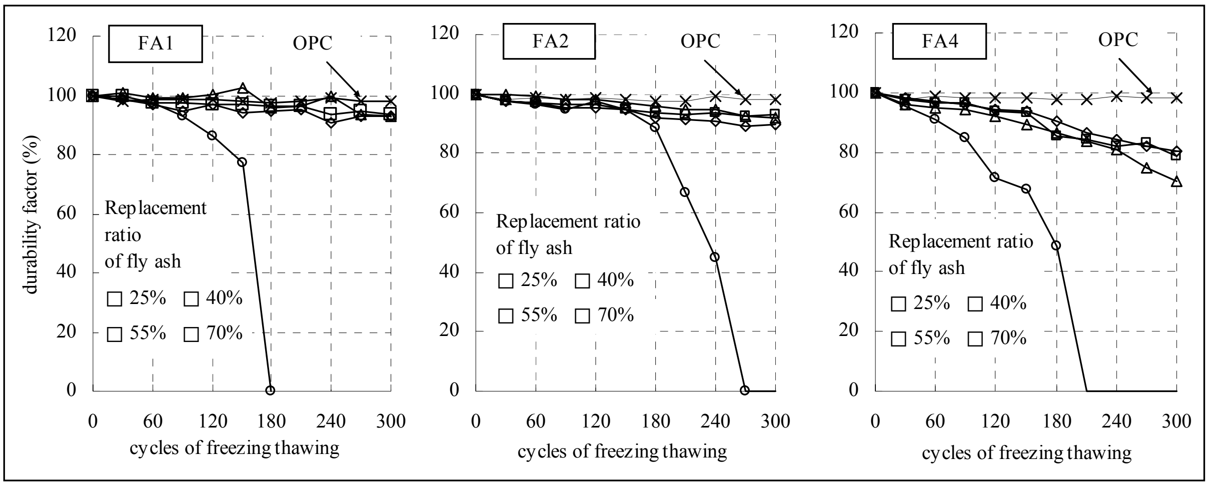


Fig.14 Relationship among particle size, replacement ratio with fly ash and durability factor (W/(C+FA)=43%)

7. CONCLUSIONS

Follwing conclusions may be drawn from the experimental study on the Durability of fly ash concrete was found to be effected of particle size of fly ash and portland cement replacement ratio.

(1) compressive strength was found to be increased with the increase in specific surface area of fly ash, strength development becomes slower with the increase in replacement ratio of fly ash.

(2) Drying shrinkage of FA1 and FA2 concrete was smaller than OPC concrete and FA4 concrete, and was reduced with the increase in replacement ratio of fly ash.

(3) Carbonation of fly ash concrete increases rapidly with the increase in replacement ratio or unit content of fly ash. Increase in carbonation with the increase in fly ash replacement ratio can be due to reduction in $Ca(OH)_2$ formation caused by reduction in portland cement and consumption of $Ca(OH)_2$ in pozzolanic activity of fly ash.

(4) Air-entraining concrete containing FA1 and FA2 fly ash with 25 to 55% replacement ratio was found to be durable to freezing and thawing. Fly ash concrete with 70% replacement reduces durability rapidly.

8. REFERENCES

1) Center for Coal Utilization, Japan, 2000, 'Survey Report of Coal Ashes in Japan '.

2) Japanese Architectual Institue. 2003, 'Japanese Architectural Standard Specification JASS 5 for Reinforced Concrete Work, '.

3) Japanese Architectual Institue. 2001, 'Japanese Architectural Standard Specification JASS 5N for Reinforced Concrete Work at Nuclear Power Plants, '.

4) Honda, S & Shiiba, 2001, 'Several properties of conrete using high-volume fly ash' Proc. of the Japan Concrete Institute, Vol.23, No.2, pp.127-132.

5) Neville, A., 1981, 'Properties of concrete', 3rd edition, pp.391-395, Longman Scientific & Technical

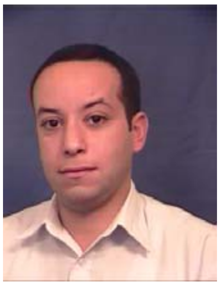
6) Hamada, M., 1969, 'Carbonation of Concrete and Corrosion of Reinforcing Steel', Proc. of the Cement Association of Japan, Cement & Concrete, No.272, pp.2-18, October.

7) Meyer, A., 1968, 'Investigation on the carbonation of concrete', Proc. of 5th Int.Symp. on Chemistry of Cement, Part 3, pp.394-401.

8) Wada, S., Abe, M., Yanagi, K. & Fujita, K., 1998, 'Experimental Study on Carbonation, Watertightness and Salt Interruption of Fly Ash Concrete', Proc. of the Japan Concrete Institute, Vol.20, No.2, pp.121-126.

9) Kokubo, M. & Nagataki, S., 1989, 'Carbonation of Concrete with Fly Ash and Corrosion of Reinforcements in 20-Years Tests', Proc. of 3rd International Conference on Fly Ash, Silica Fume, Slag and Natural Pozzolans in Concrete, Vol.1, ACI SP114-14, pp.315-329.

Experimental quantification of the products of carbonation of cement-based materials



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ABSTRACT

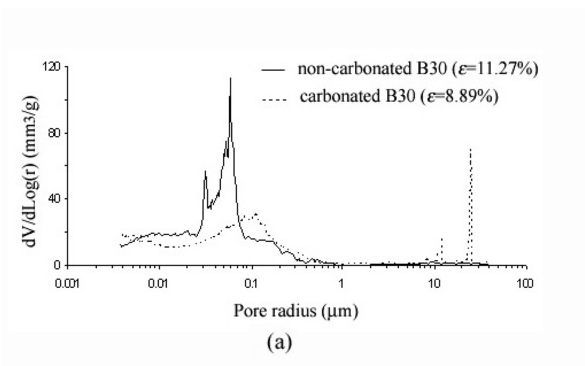
Carbonation process generates some physico-chemical effects such as modification of the porosity and the compressive strength. These two physical consequences are slightly depending on the quantity of solid products of carbonation.
Based on the results of accelerated carbonation test, this work suggests a method to quantify the water and solid products of carbonation. The tested materials, two types of mortars and concretes, were previously cured under water.
Results show that the quantity of water produced in tested materials varies between 0.3 and 0.5% and that of solid products between 0.9 and 2.9%, relatively to non-carbonated materials.
This study provides a complementary explanation to the reduction of porosity and the enhancement of compressive strength in cement-based materials due to carbonation.

KEYWORDS

Cementitious materials, carbonation process, carbonation products.

1 INTRODUCTION

Steel corrosion is recognized as the most important durability problem for reinforced concrete structures [Kashino 1984], [CEB 1989]. The steel reinforcing bars are protected from corrosion by the high alkalinity (pH>13) of the pore solution. This high alkalinity is due to the OH^- ions, which are mainly produced by the dissolution of portlandite, $Ca(OH)_2$. By penetration within cement-based materials, the atmospheric CO_2 reacts with the dissolved portlandite and initiates the carbonation process. This phenomenon leads to the fall of the pH of the pore solution and the production of calcium carbonate ($CaCO_3$) and water.
Generally, two traditional approaches are used to monitor the carbonation progress in cement-based materials. The first one refers to the quantification of the carbonation degree and the second concerns the measurement of the carbonation depth. This depth is usually called carbonation front. It is defined as the limit between carbonated and non-carbonated zones in the material. The most applied technique for measuring carbonation depth is the phenolphthalein method. However, this technique provides approximate and incomplete information about the carbonation process. For the quantification of the carbonation degree, Asano *et al.* [1971] and Fumiaki *et al.* [2000] used XRD technique. Nevertheless, this technique is too complex for use as a routine test method for simple identification of carbonation progress. Richardson *et al*. [1993] and Valls & Vàzquez [2001] had tested RMN method and Cole & Kroone [1960] and Rahman & Glasser [1989] had tested thermogravimetry analysis (TGA). These two techniques are suitable to quantify both the hydrated and the carbonated products, respectively. Their major inconvenient is the interpretation of results, which seems to be rather difficult and uncertain. Groves *et al*. [1990] and Curtil *et al*. [1993] used the polarizing microscope (with great magnification) to distinguish the mineralogy of calcium carbonate from hydrates. However, this technique over-estimates the carbonation degree when the materials contain calcareous aggregates. Except phenolphthalein indicator, all the cited techniques are very expensive and time-consuming for analyzing results. In comparison with these techniques, phenolphthalein indicator is more rapid and has a very low cost. Generally, the measurement of carbonation depth by phenolphthalein indicator does not include any quantitative evaluation of carbonation.
It is well known that carbonation changes the microstructure of cement-based materials and reduces their porosity ([Ngala & Page 1997] and [Miragliotta *et al*. 1999] (Fig. 1 a)).
The reduction of porosity and the volume of coarse pores lead to increase the compressive strength of the carbonated materials ([Matsusato *et al*. 1992] (Fig. 1 b)).
In this work, we propose a simple method to quantify the carbonation products: calcium carbonate and water. We discuss the relationship between the carbonation solid products and porosity reduction as well as compressive strength enhancement in carbonated cement-based materials.



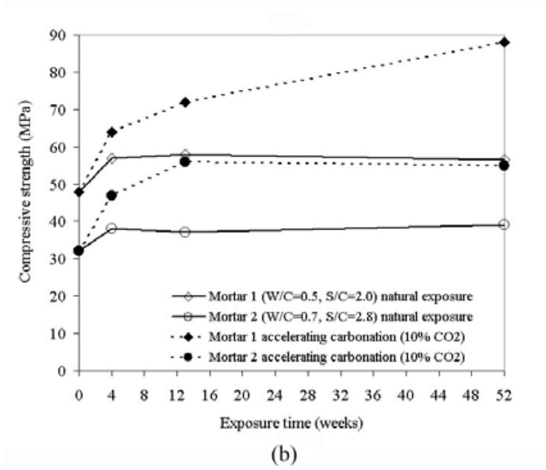


Figure 1. Carbonation effects: (a) microstructure and porosity evolution (Miragliotta *et al.* [1999], (b) compressive strength enhancement (Matsusato *et al.* [1992])

2 MATERIALS AND METHODS

2.1 Materials and conservation

Four cement-based materials were tested: two mortars, M1 and M2 and two concretes C1 and C2. They were manufactured following European standardized procedure: EN 196-1, with an OPC of type CEM I 52.5 (EN 197-1). The compositions of these materials are given in tables 1 and 2. Specimens have prismatic shape: 4×4×16 cm^3 for mortar and 7×7×28 cm^3 for concrete. After manufacturing, they were stored in a climatic chamber (20°C, 95% RH) during 24 hours. After being turned, they were sawn in identical thickness pieces: 4×4×2 cm^3 for mortar and 7×7×4 cm^3 for concrete, then cured in water until total hydration and saturation (90 days). They were removed from water and conserved at a constant temperature (20°C) in boxes containing a saline solution (NH_4NO_3) for ensuring a relative humidity of about 65%. Mortar and concrete samples are regularly weighed, until their masses stabilization, a proof that a moisture equilibrium between the materials and their conservation ambience is reached. Then, the samples were resin-impermeabilized on four of their faces in order to ensure unidirectional exchanges with their exposure ambience (through two parallel faces without resin). Half of the samples were dried at 105°C for determining their water content at 65% RH. The other were tested in accelerated carbonation test.

Material	*C/S*	*W/C*
M1	1/3	0.5
M2	1/3	0.8

Table 1. Composition of mortars

Material	*Cement* [kg.m^{-3}]	*Water* [l.m^{-3}]	*W/C* [-]	*Gravel* (5/12.5mm) [kg.m^{-3}]	*Sand* (0/4mm) [kg.m^{-3}]
C1	410.0	205.0	0.5	1180.0	689.4
C2	260.0	208.0	0.8	1129.1	844.2

Table 2. Composition of concretes

2.2 Accelerated carbonation test

The carbonation test is carried out in a container of 0.3 m^3 at a constant temperature and RH of 20°C and 65%, respectively. The ambience, which contains a gas mixture (50% CO_2 + 50% air), is continuously controlled by an infrared carbon dioxide analyzer. For each tested material, four samples were tested during 7, 28 and 90 days, where the carbonation front is measured on two samples. The two others carbonated samples were dried at 105°C and regularly weighed until their mass stabilization.

2.3 Assessment of carbonation products

The water content of the carbonated samples, lost during drying, includes the water contained in samples conditioned at 65% RH before carbonation and the water produced during carbonation. Obviously, if assuming that the water produced during carbonation remains within the material and doesn't diffuses to the ambience. The drying of the non-carbonated samples gives the quantity of water within the materials when they are moisture equilibrated in ambiance of 65% RH. Since these samples have the same shape and dimensions to those tested in carbonation, we suppose that the samples tested in accelerated carbonation have the same relative water content measured by drying the non-carbonated samples. Consequently, the evolution of relative weights of solid products of carbonation can be obtained by:

$$\Delta m_{\mathrm{sc}} = \frac{m_{cd} - m_d}{m} \times 100 \qquad (1)$$

$$\Delta m_{\text{wc}} = \frac{(m_c - m_{cd}) - (m - m_d)}{m} \times 100 \tag{2}$$

Where:

Δm_{sc} and Δm_{wc} (%) are the relative gain in weight due to the solid compounds and water produced during carbonation, respectively;

m (g): mass of sample before carbonation;

m_c (g): mass of sample after carbonation;

m_d (g): mass of sample dried at 105°C (without carbonation);

m_{cd} (g): mass of sample carbonated and dried at 105°C.

3 RESULTS AND DISCUSSIONS

3.1 Evolution of Carbonation front

Figure 2 shows the results of accelerated carbonation test obtained by phenolphthalein indicator, according to the RILEM [1984] recommendations. The concrete C2 is the more carbonated material. As early as 7 days, a thickness of 14 mm, from the exposed face, is carbonated. After 90 days, the carbonation reaches 28 mm.

The mortars seem to be more resistant to the carbonation compared with the concretes. After 90 days, the carbonation in mortars M1 and M2 reaches hardly 6 and 10 mm, respectively. These results are due to the fact that the volume fraction of cement paste in mortars is higher than in concrete (high cement content) and globally the mortar's porous network is more tortuous (high sand in mortars) and less connected [Maso 1980]. Consequently, the diffusion of CO_2 slows in this kind of material.

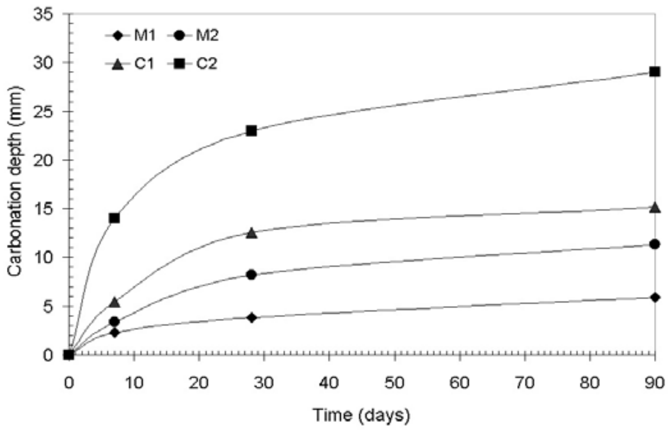


Figure 2. Carbonation front vs time

For the same *W/C* concretes are usually more denser than mortars, due the weak porosity of gravel. But on the other hand, during carbonation process, only the cement paste in concretes or mortars is concerned by the diffusion and the reaction of CO_2. Aggregates are circumvented during CO_2 diffusion. The compactness of the cement paste, and therefore the resistance to carbonation, increase with the quantity of cement used for manufacturing mortars or concretes.

It is also noticed that after 7 days of exposure, carbonation depths of mortars M1 and M2 and concrete C1 have approximately the same magnitude. According to Meyer [Meyer 1987] and Miragliotta [Miragliotta 2000], the skin layer of the materials may have the same composition: cement paste agglomeration in the moulded faces. Therefore the transfer properties of this zone are similar.

3.2 Drying kinetics

Drying operations were carried out at 105°C on carbonated and non-carbonated samples. Results show that the weights stabilize after 40 days of desiccation.
The comparison of the slopes of the drying curves shows that weights of non-carbonated materials decrease more quickly than that of the carbonated ones. This is noticed clearly on the curves of materials carbonated during 90 days. As the carbonation progress, the porosity decreases and the water evacuation of the carbonated samples becomes harder.
Relatively to its initial weight, mortar M2 contains more water than the other materials. After 60 days of drying, the relative lost in weight of the carbonated M2 exceeds 5% (Fig. 3b), while the carbonated concrete C1 loses about 2.5% (Fig. 3c).
The drying curves of materials with smaller *W/C* ratio (M1 and C1) are more spread out than those of M2 and C2 (high *W/C* ratio). Between 20 and 60 days of drying, the weights of carbonated M1 and C1 continue to decrease slowly, while those of M2 and C2 are constant (Figs 3b and 3d). Materials with high *W/C* ratio have a great porosity and their pores are more connected. Consequently, these materials dry more quickly than those with smaller *W/C* ratio.

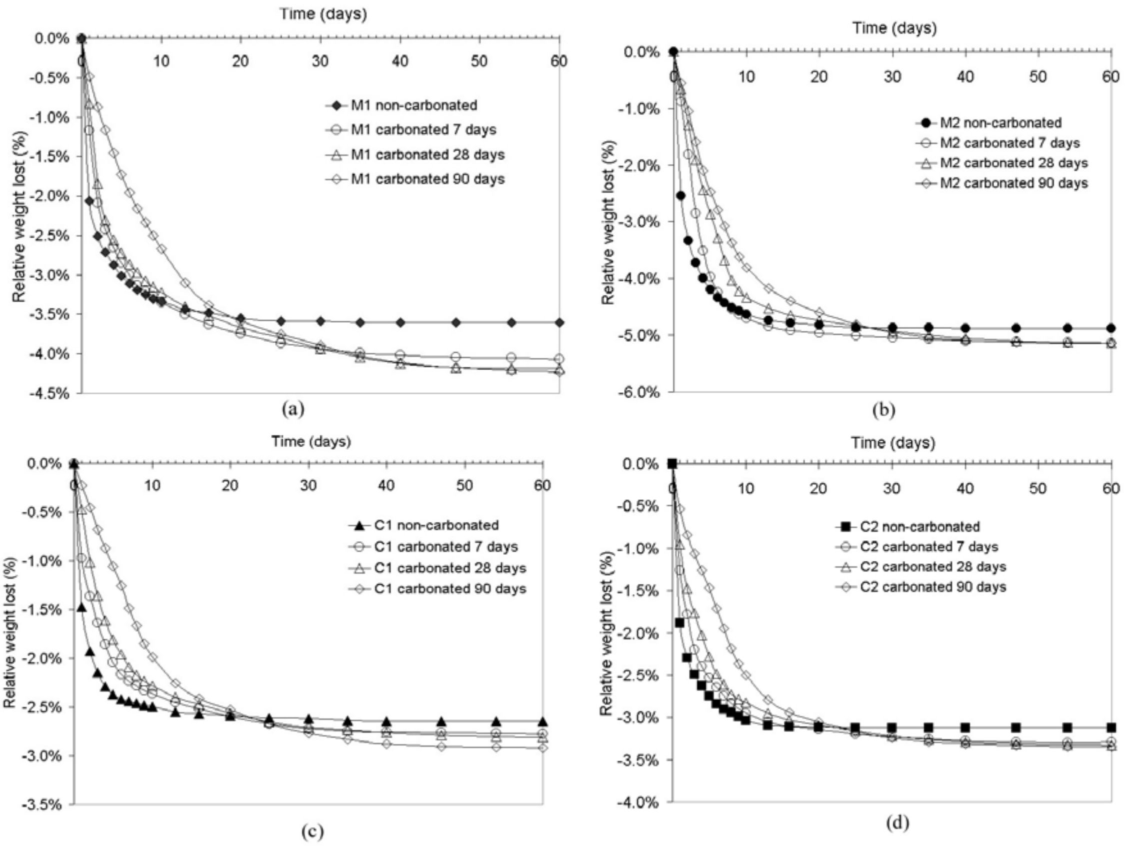


Figure 3. Relative weight loss during drying vs time

3.3 Quantification of the carbonation products

The relative gain in weight due to the solid products generated during carbonation can be deduced from equation (1). In fact, this equation quantifies the difference between the solid products generated and the solid compounds consumed (hydrates) during carbonation. Relative gain in weight can reflect the carbonation degree of the materials.
The volumes of the tested mortars and concretes are different (section 2.1). For comparing their degrees of carbonation, the gain in weight of these materials (m_{cd}-m_d) is divided by their respective surface of exposure (A=16 cm^2 for mortars and A=56 cm^2 for concretes).
In figure 4, we report the evolution of the mass of the solid products for a unit surface of exposure in (g/cm^2). The quantity of solid products of carbonation increase with time. Globally, it is showed that materials with high *W/C* ratio cumulate more quantity of these solid products. After 7 days of exposure

and for the same *W/C* ratio, mortars pile up more solid carbonation products than concretes. The quantity of solid carbonation products in concrete C2 increase rapidly. This is in correlation with the evolution of the carbonation front in this material (Fig. 2). The quantity of solid carbonation products in mortar M1 is more important than in the concrete C1, although the carbonation depth of C1 is more important than those of M1. Considering aggregates inert during carbonation, fraction volume of cement paste (reacting volume) in mortar M1 is more large than in concrete C1. After 90 days of carbonation, according to relation (1), mortars M1 and M2 pile up 2.4 and 2.9% of solid carbonation products, respectively, whereas concretes C1 and C2 accumulate 0.9 and 1.8%, respectively.

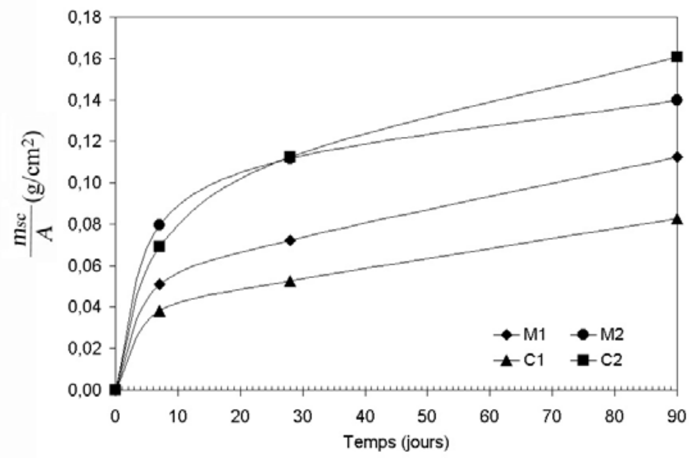


Figure 4. Gain in weight due to solid products generated by carbonation

Equation (2) quantifies the water produced during carbonation. Figure 5 shows that the water quantity produced during carbonation (for a unit surface of exposure) is largely lower than that of solid products. According to relation (2), the relative gain in weight due to water doesn't exceed 0.55%. At any stage of carbonation, the gains in weight due to water produced in the four materials during carbonation are approximately the same.
Carbonation reactions are exothermic and the heat released (74 kJ/mol) is proportional to the quantity of reactants (products). Moorehead [1986] supposes that this released heat may evaporates the water produced during the carbonation of portlandite. This assumption is not confirmed in this study because, it was find that water produced during carbonation increase with time.

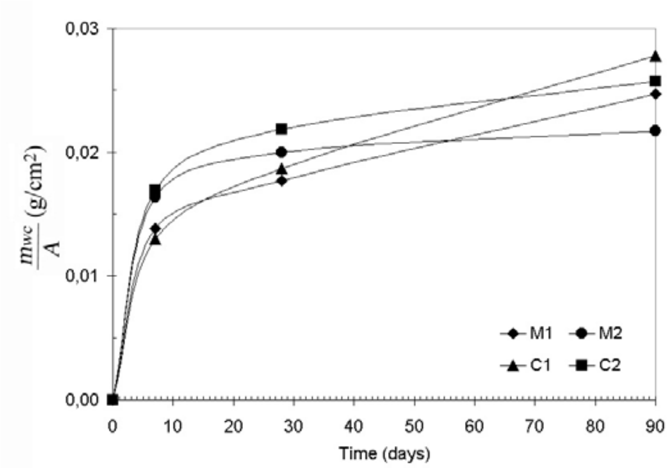


Figure 5. Gain in weight due to water produced during accelerated carbonation test

3.4 Porosity decrease during carbonation

The main reactions of concrete or mortar are those describing the conversion of $Ca(OH)_2$ (portlandite) into calcium carbonate and the decomposition of CSH:

$$Ca(OH)_2 + CO_2 \rightarrow CaCO_3 + H_2O \tag{3-a}$$

$$3CaO.2SiO_2.3H_2O + 3CO_2 \rightarrow 3CaCO_3.2SiO_2.3H_2O \tag{3-b}$$

Carbonation changes the microstructure of cement-based materials because solid products (calcium carbonate and silica gel) settle in pores with diameter between 20 nm and 400 nm (Fig. 1 a) and filling them. In other hand, a new variety of pores appears, corresponding to the porosity of solid products of carbonation and the voids generated after hydrates consumption according to reactions (3).

The gain in weight measured during carbonation includes the difference between the solid products generated and the hydrates consumed during carbonation. In order to exploit these measures it is necessary to distinguish between the masses of consumed hydrates and produced calcium carbonate and silica gel. This is will be possible when the initial concentrations of hydrates (before carbonation) and their kinetics of carbonation are known. Using respective densities of the solid species, we can obtain their volumes variations during carbonation. Relatively to the volume of the carbonated zone, the respective differences between the volumes of products and reactants in equations (3), represent the reduction in porosity due to the carbonation.

This reduction of porosity and the high densities of solid products of carbonation leads to increase the compressive strength of the carbonated materials.

4 CONCLUSIONS

We propose a simple procedure to quantify solid compounds and water produced during carbonation process. Results show that the quantity of water produced in mortars and concretes is largely smaller than that of solid products.

The evolution of the depths of carbonation in the cement-based materials depends on their compactness. This property is as greater as the *W/C* ratio is lower and the cement content is higher.

For the materials with *W/C*=0.5, the quantity of solid products of carbonation is more important in mortars than in concretes, despite the fact that carbonation fronts of concretes are more important. The reacting volume of mortars is more large than those of concretes (containing gravel). Conversely, for materials with *W/C*=0.8 the quantity of solid products of carbonation is more important in concretes than in mortars. In fact, the high depths of carbonation of concretes generate a large reacting volume, although the quantity of cement in concrete is less than in mortars.

In high porous materials (high W/C and low cement contain), the process that monitor carbonation rate is the CO_2 diffusion. In lower porous materials (low W/C and high cement content), reactions kinetics of carbonation control the rate of the process. Therefore, it seems that the two modes of investigation: carbonation depths and carbonation degree are complementary tools for monitoring the carbonation process.

It known that carbonation reactions are exothermic. The measurements of the water produced during carbonation show that the generated heat of this process doesn't evaporate all the quantity of water produced during carbonation. This result must be confirmed by others protocols.

5 REFERENCES

Kashino, N., 1984, 'Investigation into the limit of initial corrosion occurrence in existing reinforced concrete structures', Proc. 3rd Conf. Durability of Building Materials and Components, Espoo, Finland, August 1984, vol. 3, pp. 176-186.

Comité Euro-International du Béton 1989, Durable Concrete Structures: CEB Design guide, Bulletin d'information N° 182, Lausanne, 268 pp.

Asano, S., Kamatani Y. & Inoue Y. 1971, 'On the carbonation of calcium silicate compounds', Yogyo Kyokaishi, (Japan), **79** [9], 303-311.

Fumiaki, M., Uoshimichi, A. & Sumio, S., 2000, 'Carbonation degree of autoclaved aerated concrete', Cement and Concrete Research, **30** [11], 1741-1745.

Richardson, I. G., Groves, G. W., Brough, A. R. & Dobson, C. M., 1993, 'The carbonation of OPC and OPC/silica fume of hardened cement pastes in air under conditions of fixed humidity', Advances in Cement Research, **5** [18], 81-86.

Valls, S. & Vàzquez, E., 2001, 'Accelerated carbonation of sewage sludge-cement-sand mortars and its environmental impact', Cement and Concrete Research, **31** [9], 1271-1276.

Cole, W. F. & Kroone B., 1960, 'Carbon dioxide in hydrated Portland cement', Journal of American Concrete Institute, **31** [12], 1275-1295.

Rahman, A. A. & Glasser, F. P., 1989, 'Comparative studies of the carbonation of hydrated cement', Advances in Cement Research, **2** [6], 49-54.

Groves, G. W., Rodway, D. I. & Richardson, I. G., 1990, 'The carbonation of hardened cement pastes', Advances in Cement Research, **3** [11], 117-125.

Curtil, L., Gielly, J. & Murat, M., 1993, 'The polarizing microscope: A tool of interest for investigation on concrete; Application to carbonation', Cement and Concrete Research, **23** [2], 329-334.

Ngala, V. T. & Page, C. L., 1997, 'Effects of carbonation on pore structure and diffusional properties od hydrated cement pastes', Cement and Concrete Research, **27** [7], 995-1007.

Miragliotta, R., Rougeau, P., Aït-Mokhtar, A. & Amiri, O. 1999, 'A methodology for predicting the advance of carbonation in concrete:influence of the microstructure of the skin layer on concrete carbonation kinetics', 16th BIBM international congress, Venezia, Italy, vol. 2, pp. 73-78.

Matsusato, H., Ogawa, K., Funato, M. & Sato, T. 1992, 'Study of the carbonation of hydrated cement and effect on microstructure and strength', 9th International congress on the chemistry of cement', India, vol. 5, pp. 363-369.

RILEM, 1984, 'Measurement of hardened concrete carbonation depth. CPC-18', Materials and Structures, **17** [102], 435-440.

Meyer, A., 1987, 'The importance of the surface layer for the durability of concrete', in *Concrete Durability ACI SO-100*, ed. Katarine & Bryant Mather, vol 1, pp. 46-61.

Miragliotta, R., 2000, *Modélisation des processus physico-chimiques de la carbonatation des bétons préfabriqués: Prise en compte des effets de paroi*, PhD thesis (in french), University of La Rochelle.

Maso, J. C., 1980, 'La liaison entre les granulats et la pâte de ciment hydraté', 7ème Congrès International de la Chimie des Ciments, Paris, France, 1980, vol. 1, n° 7-1, pp. 3-14.

Moorehead, D. R. 1986, 'Cementation by the carbonation of hydrated lime', Cement and Concrete Research, **16** [5], 700-708.

Optimizing Mixing Parameters in Fly Ash Concrete With Respect To Compressive Strength and Chloride Permeability

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ABSTRACT

Durability is the key concern of reinforced concrete structure throughout its service life and permeability is the basic feature of concrete to maintain durability without deterioration. Corrosion of reinforcement is the most serious of all the durability problems and especially arises from the ingress of chloride ions.

In this study, the effect of fly ash addition on the chloride ion penetration into the concrete is investigated together with its effect on compressive strength. A factorial analysis approach is applied to find out the main influencing factors on these chosen properties. In the analysis, four input factors were used, such as total binder content, water/binder ratio, fly ash/binder ratio as numeric factors and curing condition (in water or in air) as categorical factor. The range of the numeric factors were selected between 300-400 kg/m^3 for binder content, 0.46-0.60 for water/binder ratio and 0.10-0.40 for fly ash/binder ratio. Water curing was applied over the half of the specimens while the remaining ones were stored in laboratory conditions. The compressive strength and rapid chloride permeability tests were conducted on the specimens at the age of 90 days.

It has been found that the main factors affecting both chloride ion penetration resistance and compressive strength are water/binder ratio and fly ash/binder ratio as numeric ones and also curing condition as categorical one. Suprisingly, the amount of total binder is not a significant factor on the observed properties within the range of investigation. Statistical method also shows that there are interactions between binder content and water/binder ratio, binder content and fly ash/binder ratio for compressive strength, and between water/binder and curing condition, fly ash/binder ratio and curing condition for rapid chloride permeability, respectively.

KEYWORDS

Durability, fly ash concrete,compressive strength, chloride permeability, factorial analysis.

1 INTRODUCTION

Pozzolanic materials, most of which are by products of industries, are widely used in concrete production, especially to increase the durability properties. Fly ash, granulated blast furnace slag (GBFS) and silica fume are the most common pozzolans together with natural pozzolans used in concrete technology. These pozzolans are also used in the production of pozzolanic cements (CEM II and IV, in accordance with EN 197-1 standard [EN 197-1 2000]), hence the utilization of pozzolans in both concrete and cement industry helps to reduce CO_2 emission in the world.

Pozzolanic materials react with $Ca(OH)_2$, generated during the hydration of C_3S and C_2S components of cement, to form calcium silicate hydrate (C-S-H), hence reduce a weak part of concrete against durability. Furthermore, pozzolans improve the impermeability of concrete which is the main parameter in durability of concrete.

Moist curing affects both strength and durability properties of concrete. The humidity of concrete should be such that the vapour pressure in the capillaries should remain over 80% of the saturation limit [Neville 1993], otherwise the hydration reaction between cement and water terminates. For type I cement concrete, 58% reduction in strength was reported [Price 1951] between moist-cured and air-cured specimens at the age of 6 months. A continuous strength increase was obtained [Wood 1991] for the concretes prepared with type I cement and stored in water for five years; while the strength of air-cured specimens remained at the level of 28-day strength for the 5 years old specimens. A strength reduction of about 17-22% was mentioned [Aitcin *et al.* 1994] between the water-cured and air-cured specimens at the age of 1 year.

The effect of poor curing on strength of concretes prepared with pozzolans and pozzolanic cements is worse than that for ordinary Portland cement (OPC); a strength loss of 38% was reported [Ramezanianpour & Malhotra 1995] for the concretes of 25% GBFS replacement, and up to 50% reduction in strength was obtained for the 25% fly ash or more, or 50% GBFS replacement between the two extreme curing conditions. Pozzolanic (natural) cement concrete, water-cured for 14 days and over, can reach the strength level of OPC concrete cured continuously in water, in periods shorter than 2 months [Ozer & Ozkul 2004 a]. However, the pozzolanic cement concretes when initially water-cured for 3 days or shorter, can never attain the strength of the latter concrete stored continuously in water.

The inadequate curing also affects the durability properties of concrete. Six times reduction in the coefficient of oxygen permeability was obtained [Thomas *et al.* 1989] for the concretes prepared with OPC when the water curing period is extended from 1 day to 28 days. Similarly, the oxygen permeability decreased 10 times for the concretes which contain 15% fly ash when the curing period increased from 1 day to 28 days. The resistance to chloride-ion penetration of concretes prepared with slag, fly ash or silica fume, increased in large extends for the water-cured specimens with respect to air-cured ones. For trass-cement concretes, the difference between the sorptivities of the air-cured and water-cured specimens is 5.3 times while it is only 1.7 times for OPC concretes [Ozer & Ozkul 2004 b]

2 EXPERIMENTAL

2.1 Materials

An ordinary portland cement, PC 42.5 (CEM I, in accordance to TS EN 197-1 standard) and a fly ash, maintained from Tuncbilek are used. Tuncbilek fly ash meets the requirements of Turkish Standart TS EN 197-1 class V (silicious fly ash) with a reactive CaO content of less than 10% and a reactive SiO_2

content of more than 25% (40.1%). Furthermore, Tuncbilek fly ash also meets the requirements of ASTM C 618 Class F fly ash with a total amount of $SiO_2 + Al_2O_3 + Fe_2O_3$ more than 70%. Physical and chemical properties of cement and fly ash are presented in Table 1.

			Cement	*Fly Ash*
Physical properties	Specific gravity		3.17	2.24
	Fineness	Passing 45μ(%)	91.6	93.4
		Blaine cm^2/g	3560	5350
	Compressive Strength, MPa	2-day	28.6	-
		7-day	44.0	-
		28-day	55.2	-
	Setting time (h:min)	Initial	2:37	-
		Final	3:08	-
	Strength activity index (%)	7-day	-	101.3
		28-day	-	113.8
Chemical analysis	Silicon dioxide (SiO_2)		20.41	59.29
	Aluminum oxide (Al_2O_3)		4.72	16.76
	Ferric oxide (Fe_2O_3)		3.70	10.97
	Calcium oxide (CaO)		65.08	3.06
	Magnesium oxide (MgO)		0.92	5.43
	Sulfur trioxide (SO_3)		2.56	0.60
	Sodium oxide (Na_2O)		0.34	0.74
	Potassium oxide (K_2O)		0.81	1.80
	Free lime		1.20	0.00
	Loss on ignition (%)		1.39	0.02
	Bogue potential compound composition	C_3S	60.57	-
		C_2S	12.91	-
		C_3A	6.26	-
		C_4AF	11.26	-

Table 1. Physical and chemical compositions of cement and fly ash

Natural and crushed stone sand were used as fine aggregates with specific gravities of 2.60 kg/dm^3 and 2.70 kg/dm^3, respectively. Crushed limestone with a maximum size of 25 mm and specific gravity of 2.72 kg/dm^3 was used as coarse aggregate. A polycarboxylate based HRWR admixture was employed in all the mixtures to obtain a similar workability.

2.2 Mixture design and methodology

In this study central composite design (CCD) was utilized. Four input factors were used in the test program: A (total binder amount), B (water/binder ratio), C (fly ash/binder ratio) as numeric factors and D (water and air curing) as categorical factor. The ranges of the numeric factors were selected between 300kg/m^3-400kg/m^3 for A, 0.46-0.60 for B and 0.10-0.40 for C. Each of the three selected mixture components is investigated at five levels. According to CCD, k=3 independent variables compose 2^k=8 factorial (or cube) points representing all combinations of codified values of ±1, 2k=6 axial (or star) points where each variable adjusted at a distance ±α from the origin with the other variables kept at the center point and six replicate cenral points with coded values of zero in order to estimate the degree of experimental error. The α value was 1.68 so as to make the design rotatable which fixes the uncertainty of determined response surface symmetry [Caulcut 1991]. Ranges and coded value of numeric factors are presented in Table 2. Total number of mixtures was 20 and the order was randomized. Mixture proportions and test results are given in Table 3. The modeled responses were 90-day compressive strength and rapid chloride permeation.

Factors	*Unit*	*Coded value*				
		-1.68	*-1*	*0*	*1*	*1.68*
Binder (A)	Kg/m^3	266	300	350	400	434
Water/Binder (B)	-	0.41	0.46	0.53	0.60	0.65
Fly Ash/Binder (C)	-	0	0.10	0.25	0.40	0.50

Table 2. Numeric factors, their ranges and codified values

All the concrete mixtures were prepared in a pan mixer with 0.04 m^3 volume and the admixture dosage adjusted to have a workability with a slump value of about 15±2 cm. Thus, admixture dosages were in the range of 0 to 1.4% by weight of total binder content. From each batch, 100 x 200 mm cylinders for rapid chloride permeability test and 150 x 150 x 150 mm cubes for compressive strength test were cast. Vibration table was utilized for compacting the specimens. After demoulding, half of the specimens were immersed in lime saturated water at 23±2^{O}C while rest were stored in air in laboratory conditions (R.H. 65±5% and 22±2^{O}C temperature) for 90 days. Rapid chloride permeability test was carried out in accordance with ASTM C 1202.

3 TEST RESULTS AND DISCUSSION

A summary of the slump values, 90-day compressive strengths and rapid chloride permeations (coulomb) for two curing conditions concerning mixture proportions is presented in Table 3.

3.1 Statistical analysis

A commercially available software for design and analysis was used. The objective was to determine a mixture replacing the maximum possible amount of fly ash with cement which would maintain a less chloride permeability and similar compressive strength. Analysis of variance (ANOVA) was carried out to calculate the significance of factors and interactions among them in the two response models. Table 4 shows the coefficients of response models and results of ANOVA.

Coefficients show the significance of the factors on response. Mean square of a term is the varience associated with that term. It is the sum of squares divided by the degrees of freedom. Probability>F values indicates the probability that the contribution of a given parameter to the tested response exceeds the value of the specified coefficient. Thus, provided this value is <0.05, factor is significant.

Two mathematical models derived for each response because of the categorical factor, curing. Consequently, variation of influence of the factors with respect to curing is determined.

$$\text{Strength}_{AIR}=+215.185 - 0.386 \times A - 343.048 \times B + 121.773 \times C + 3.8 \times 10^{-4} \times A^2 + 139.784 \times B^2 - 77.196 \times C^2 + 0.348 \times A \times B - 0.243 \times A \times C - 26.786 \times B \times C$$

$$\text{Strength}_{WATER}=+213.885 - 0.378 \times A - 340.519 \times B + 122.035 \times C + 3.8 \times 10^{-4} \times A^2 + 139.784 \times B^2 - 77.196 \times C^2 + 0.348 \times A \times B - 0.243 \times A \times C - 26.786 \times B \times C$$

Table 4 shows that the most significant terms in strength response are B (water/binder ratio), C (fly ash/binder ratio) and D (curing). There exist also interactions between A (binder content) and B (water/binder ratio) as well as A and C.

Run	*Mixture proportions*			*Curing*	*Test results*		
	Binder (kg/m^3)	*W/B*	*FA/B*		*Slump (cm)*	*90-d CS (Mpa)*	*RCP (Coulomb)*
1	350	0.53	0.25	Air	16	46.9	2874
				Water		50.2	527
2	266	0.53	0.25	Air	13	51.8	2448
				Water		53.5	554
3	400	0.60	0.40	Air	17	39.6	4869
				Water		42.8	622
4	350	0.53	0.25	Air	14	47.7	2637
				Water		51.2	456
5	350	0.53	0.25	Air	15	46.7	2281
				Water		53.4	527
6	350	0.53	0.25	Air	16	50.1	2327
				Water		53.7	480
7	400	0.46	0.10	Air	16	54.2	2719
				Water		63.3	985
8	300	0.46	0.40	Air	13	56.2	1133
				Water		58.0	207
9	350	0.65	0.25	Air	18	43.0	5122
				Water		47.7	709
10	434	0.53	0.25	Air	14	55.5	1612
				Water		58.6	460
11	350	0.41	0.25	Air	14	62.2	693
				Water		63.5	260
12	350	0.53	0.00	Air	16	50.1	4271
				Water		51.8	3443
13	300	0.60	0.40	Air	17	40.9	4251
				Water		45.6	406
14	300	0.46	0.10	Air	17	57.3	959
				Water		59.5	509
15	350	0.53	0.25	Air	16	51.0	2829
				Water		57.2	500
16	400	0.46	0.40	Air	17	50.2	1421
				Water		52.3	229
17	350	0.53	0.50	Air	17	41.5	3877
				Water		45.6	258
18	400	0.60	0.10	Air	18	50.2	4663
				Water		51.6	1316
19	300	0.60	0.10	Air	16	43.3	3567
				Water		45.9	1103
20	350	0.53	0.25	Air	15	51.1	2651
				Water		56.5	507

Table 3. Mixture proportions and test results

ANOVA results given in Table 4 for rapid chloride permeation show that W/B, FA/B and curing (D) are sigificant parameters, and there also exist interactions between W/B and curing, as well as FA/B and curing. Model equations for air and water curing are given below.

$$RCP_{AIR} = -15637.77 + 51.93 \text{ x } A + 17387.31 \text{ x } B - 8897.28 \text{ x } C - 0.05666 \text{ x } A^2 + 1935.43 \text{ x } B^2 + 20318.69 \text{ x } C^2 - 7.20 \text{ x } A \text{ x } B - 20.01 \text{ x } A \text{ x } C - 10005.95 \text{ x } B \text{ x } C$$

$$RCP_{WATER} = -7107.90 + 49.61 \text{ x } A + 342.46 \text{ x } B - 12269.89 \text{ x } C - 0.05666 \text{ x } A^2 + 1935.43 \text{ x } B^2 + 20318.69 \text{ x } C^2 - 7.20 \text{ x } A \text{ x } B - 20.01 \text{ x } A \text{ x } C - 10005.95 \text{ x } B \text{ x } C$$

Factor	*Compressive strength (R^2=0.92)*				*RCP (R^2=0.95)*			
	Coefficient	*Mean square*	*F Value*	*Prob>F*	*Coefficient*	*Mean square*	*F Value*	*Prob>F*
Model	-	102.54	21.84	<0.0001	-	$6.5x10^6$	34.78	<0.0001
A	0.45	5.54	1.18	0.2873	114.41	$3.6x10^5$	1.90	0.1793
B	-5.49	823.35	175.41	<0.0001	762.94	$1.6x10^7$	84.70	<0.0001
C	-2.36	152.74	32.54	<0.0001	-318.60	$2.8x10^6$	14.77	0.0007
D	1.81	131.04	27.92	<0.0001	-1080.28	$4.7x10^7$	248.69	<0.0001
A^2	0.95	26.02	5.54	0.0264	-141.66	$5.8x10^5$	3.08	0.0910
B^2	0.68	13.52	2.88	0.1016	9.48	2592.3	0.014	0.9074
C^2	-1.74	86.95	18.52	0.0002	457.17	$6.0x10^6$	32.09	<0.0001
AB	1.22	23.77	5.06	0.0331	-25.19	10150.6	0.054	0.8179
AC	-1.82	52.93	11.28	0.0024	-150.06	$3.6x10^5$	1.92	0.1777
AD	0.25	1.72	0.37	0.5502	-58.11	92221.7	0.49	0.4896
BC	-0.28	1.27	0.27	0.6080	105.06	$1.8x10^5$	0.94	0.3410
BD	0.089	0.21	0.046	0.8226	-596.57	$9.7x10^6$	51.79	<0.0001
CD	0.020	0.011	0.0022	0.9626	-252.95	$1.7x10^6$	9.31	0.0052
Intercept	51.35	-	-	-	1561.26	-	-	-

A: Binder content, B: water/binder ratio, C: fly ash/binder ratio, D: categorical factor (curing)

Table 4. Coefficients and ANOVA results of the models

Illustration of variance of real responses data plotted against predicted responses for (a) compressive strength and (b) rapid chloride permeability is presented in Fig. 1. The isostrength lines (contour lines) are illustrated in Fig. 2 for two W/B levels.

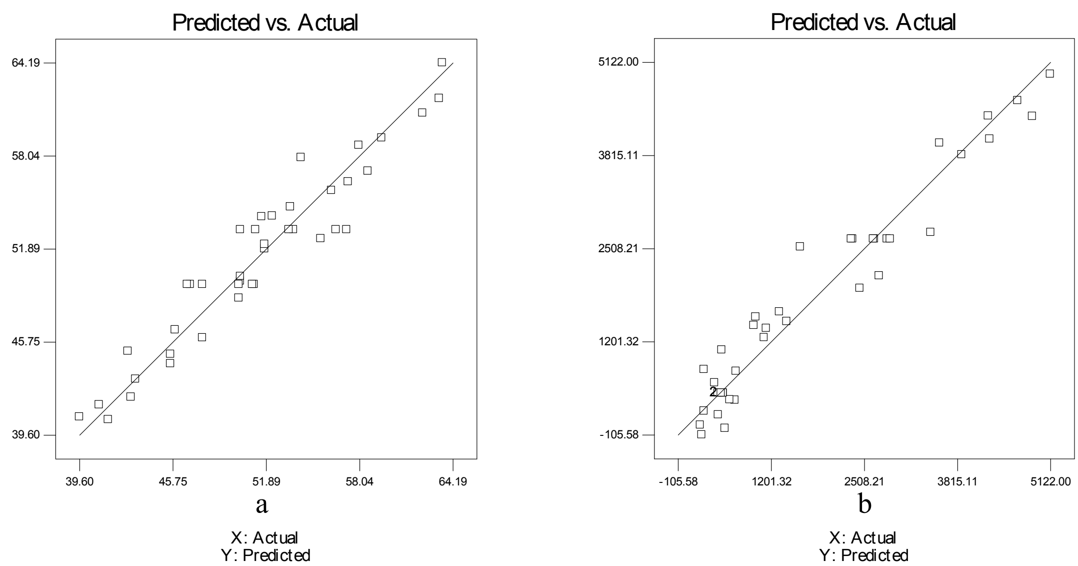

Figure 1. Predicted and actual values of (a) compressive strength and (b) RCP

Comparison of Fig. 2a and Fig 2b exhibits that there are interactions with respect to FA/B and binder content, and also W/B and binder content.Figure 2a shows that for low W/B ratios, when FA/B ratio is also low, the binder content has no effect on compressive strength of concretes stored in air. However, for the same case, but when FA/B ratios is high, which means the fly ash content is also high, increasing binder content decreases the strength. This behaviour can be due to the high relative amount of fly ash in the binder and the $Ca(OH)_2$ released during the hydration reaction of cement and also water content of concrete (low W/B ratio and air curing) are not enough for a complete pozzolanic activity. On the other hand, for high W/B ratios and low fly ash contents [Fig. 2b], fly ash shows sufficient pozzolanic activity and also due to the higher fineness of fly ash than cement, strength increases with increasing binder content (hence relative fly ash content increases).

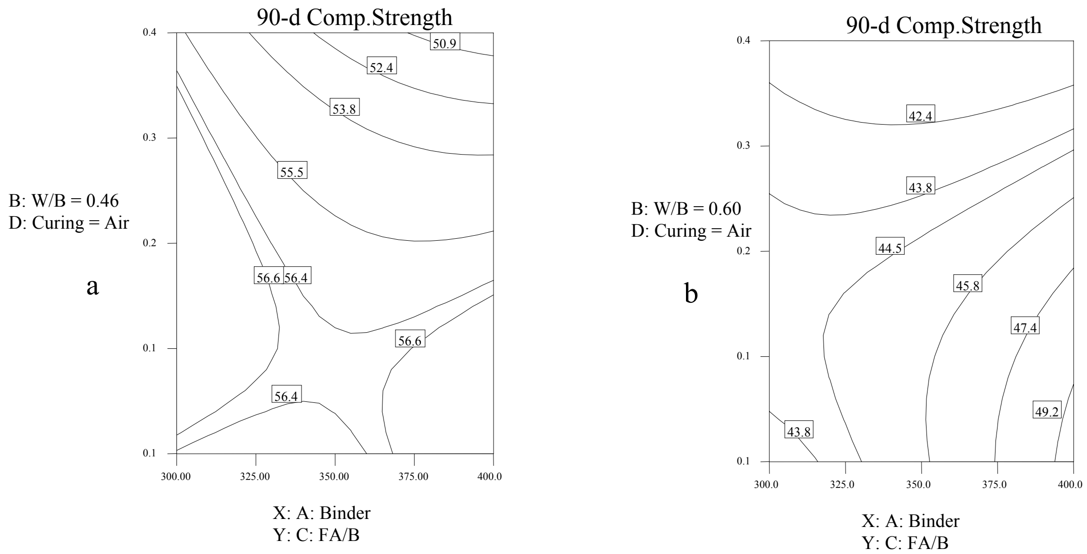

Figure 2. Isostrength lines for (a) 0.46 and (b) 0.60 W/B ratios of air stored concretes

The effect of curing can be seen from the Fig. 3, there is a shift to the higher strength levels for water-cured specimens for the same W/B ratios, however the trend remains the same for both curing conditions.

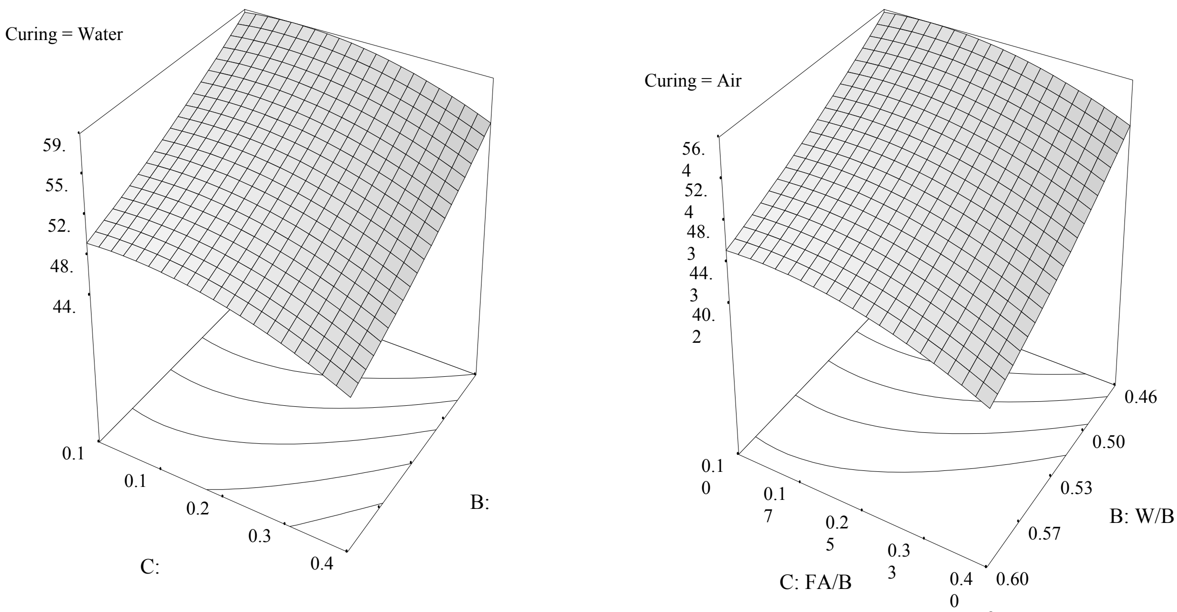

Figure 3. Response surface of compressive strength of concretes with 350 kg/m^3 binder content for both of the curing conditions

For a moderate binder content of 350 kg/m^3, the response surfaces of RCP are drawn in Fig. 4 for water and air curing. The levels of RCP in both surfaces show the positive effect of water curing on rapid chloride permeation resistance. Figure 4 also shows the interaction between curing and FA/B. For water cured concretes, when FA/B ratio decreases (hence, relative amount of fly ash increases), RCP resistance also decreases (RCP current increases) for the same W/B ratio which shows the positive effect of pozzolan (fly ash) on the chloride impermeability. However, for air stored specimens, the effect of FA/B ratio on RCP is slight, the effective parameter is W/B ratio, because pozzolans are curing sensitive and they can not exhibit their positive effect on impermeability in poor curing conditions.

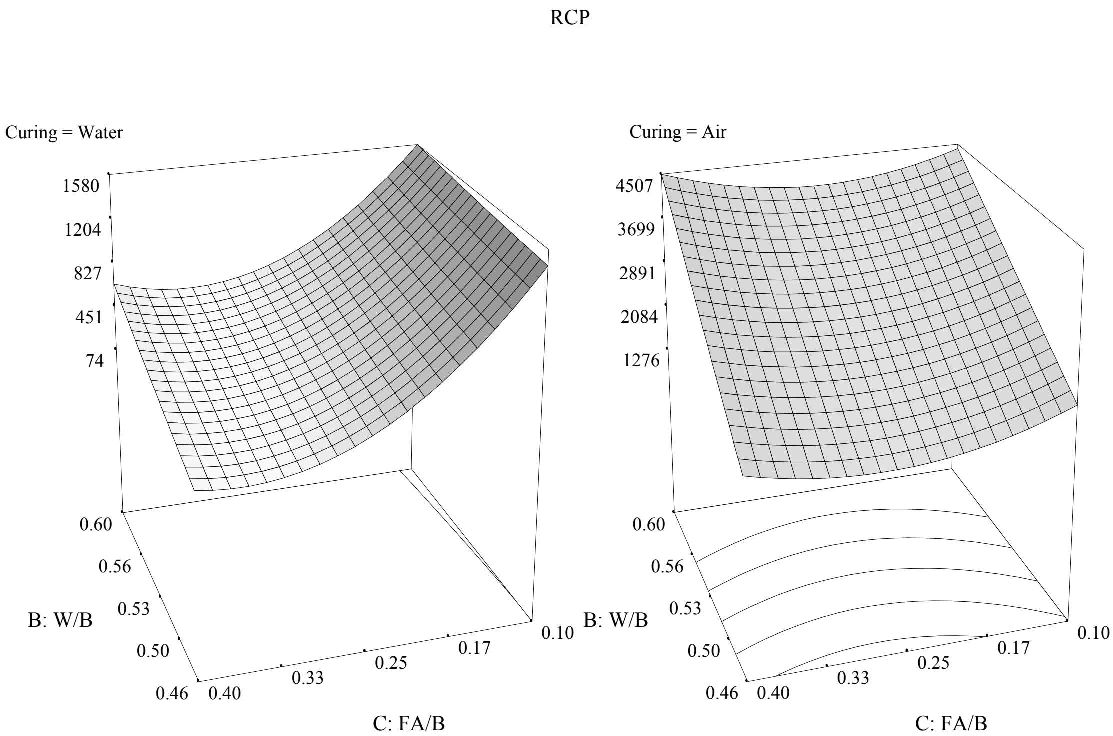

Figure 4. Response surface of RCP of concretes with 350 kg/m^3 binder content for both of the curing conditions

4 CONCLUSION

Based on the results of this study, the following conclusions can be drawn:

1. Central composite design method is a promising approach for designing and optimizing the most convenient mixture proportion of fly ash concretes with respect to compressive strength and chloride permeability.
2. Water/binder ratio, fly ash/binder ratio and curing condition are significant on compressive strength. Although binder amount is not effective itself, it has a significant interaction with each of the water/binder and fly ash/binder ratios in affecting compressive strength.
3. Interaction of curing-water/binder ratio and curing-fly ash/binder ratio are the most influential parameters in yielding rapid chloride permeability of fly ash concretes in addition to water/binder ratio, fly ash/binder ratio and curing.
4. For high level of fly ash/binder and water/binder ratios, chloride penetration resistance increases more than those of low fly ash/binder and water/binder ratios when the curing changes from air to water.
5. Fly ash/binder ratio becomes the most significant factor on Rapid Chloride Permeation when concretes immersed in water, whereas water/binder ratio is the most effective one for air stored concretes.

5 ACKNOWLEDGMENTS

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6 REFERENCES

Aitcin, P.C., Miao, B., Cook, W.D. & Mitchell, D. 1994, 'Effect of size and curing on cylinder compressive strength of normal and high strength concretes', *ACI Materials Journal,* **91**[4], 349-354.

Caulcut, 1991, *Statistics in Research and Development,* Chapman & Hall, London.

European Standard, EN 197-1, 2000, 'Composition, specification and conformity criteria for common cements', European Committee for Standardization.

Neville, A.M. 1993, *Properties of Concrete,* Longman Scientific & Technical, Harlow, Essex.

Ozer, B. & Ozkul, M.H., 2004, 'The influence of initial curing on the strength development of ordinary portland and pozzolanic cement concretes', *Cement & Concrete Research,* **34**[1], 13-18.

Ozer, B. & Ozkul, M.H., 2004, 'The effect of moist curing on the properties of concretes made by pozzolanic cements', Proc. CD of 9th Int. Symp. on Concrete Roads, Istanbul, Turkey, 4-7 April 2004.

Price, W.H., 1951, 'Factors influencing concrete strength', *Journal of American Concrete Institute,* **22**[6], 417-432.

Ramezanianpour, A.A. & Malhotra, V.M., 1995, 'Effect of curing on the compressive strength, resistance to chloride-ion penetration and porosity of concretes incorporating slag, fly ash or silica fume', *Cement & Concrete Composites,* **17**[2], 125-133.

Thomas, M.D.A., Matthews, J.D. & Haynes, C.A., 1989, 'The effect of curing on the strength and permeability of PFA concrete', Proc. 3rd CANMET/ACI Int. Conf. On The Use of Fly Ash, Silica Fume, Slag & Natural Pozzolans in Concrete, ACI SP-114, pp.191-217.

Wood, S.I., 1991, 'Evaluation of the long-term properties of concrete', *ACI Materials Journal,* **88**[6], 630-643.

CEMENTITIOUS ADHESIVES PERFORMANCE DURING SERVICE LIFE

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ABSTRACT

Ceramic tiles have been extensively used on external walls in Portugal for many years. These tiles are often bedded into mortar or into a cement-based adhesive. In the last few years, however, there have been many mistakes, such as adhesion failure, in the use of ceramic wall tiles. A fundamental cause of these failures is an inadequate selection of the cementitious adhesive.

Adhesive performance is usually evaluated upon the bases of early age behaviour. The knowledge of this behaviour is essential for classification and labelling, however it gives no indication of long term service life performance. Objective test methods that can evaluate the building system and its components' performance throughout its service life are lacking.

At Faculdade de Engenharia da Universidade do Porto's (FEUP) Building Physics Laboratory - LFC, about 40 samples were submitted to more than one hundred accelerated aging cycles. This study's main goal was to evaluate the durability of the tile/cementitious adhesives system. This was done in terms of relating the decreasing performance (the influence of types and classes of cementitious adhesives and the tiles absorption coefficients) to the tensile adhesion strength and establish criterion of long-term performance; i.e. the selection of the most adequate adhesive for an external ceramic wall tileing systems.

KEYWORDS

Cementitious adhesives, Ceramic tile coating, Service life, Accelerated aging tests, Pathology.

1 INTRODUCTION

Since the 16th century Portugal has been Europe's leading user of ceramic tile coatings for building façades. Nowadays this type of coating is still greatly used due to its high durability, large functionality and aesthetic performance.

Despite significant developments in the ceramic and adhesives industries, there are still frequent and serious problems related to detachment. Materials' performance, especially as far as the cementitious adhesives are concerned, is usually evaluated at the early stages of the application. Knowing its initial characteristics is essential to its CE marking and classification. Although this classification is essential when selecting the material that best suits the desired application, it does not cover performance over long periods of time. Objective methods that can evaluate the tileing system throughout its service life are lacking.

Ceramic tile coating systems are basically made of substrate, rendering grout, adhesive mortar and ceramic tile. These materials are subjected to temperature and humidity variation, to sun radiation and rain, especially when applied on exterior walls. The materials' response to these degradation factors can be seen with the loss of its functionality. To evaluate the influence of the aging process in the cementitious adhesives' performance we performed tests in the accelerated ageing chamber at FEUP's LFC. The chamber simulated some of the most significant climatic conditions and actions (irradiation, rain, freezing and thawing), to which the chosen building elements are generally exposed. The threshold temperature and humidity values were established in order to slightly overcome the most adverse weather conditions that exterior walls in Portugal could possibly experience. Extreme conditions of hygrothermal movement were thus established.

The samples were subject to a cycle range that varied from 1 to 112 cycles. Material response to the degradation agents can be seen by the deterioration of certain performance-related characteristics. This study's main goal is to evaluate the durability of the cementitious adhesives in relation to their decreasing performance in tensile adhesion strength.

2 THE PROBLEM

Ceramic tile detachment [Fig.1] is a severe and common pathology. Moreover, ceramic tile detachment from façades also poses a serious threat to human safety.

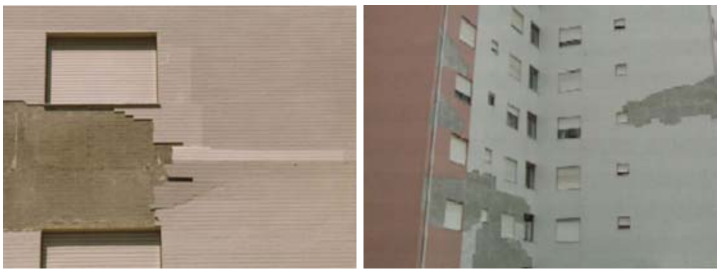


Figure 1. Pathologies on the ceramic tile coating of façades.

The main causes of ceramic tile coating detachment are: structural deformability, instability of the support, rendering grout characteristics, joint dimensions and inadequate selection of the cementitious adhesive. Although CE markings are very helpful, they don't cover material performance through time. There is an urgent need to develop service life evaluation testing methods.

3 SERVICE LIFE PREDICTION

Service life prediction of ceramic tile coatings can be estimated both by experimental or numerical analysis. On the development of a methodology for the prediction of ceramic tile coating working life and on the modelling of the degradation mechanisms it should be considered those that mostly feats its nature and its use in service.

Guidance Document 003 of the European Organization for Technical Approvals – GD003, EOTA, December 1999 – proposes a systematic methodology for assessing and/or predicting the working life of products. Based on that proposed methodology, we established the followed steps to evaluate the durability of ceramic tile coating:

1. Definition – User needs, building context, performance requirements and criteria and product characterisation are established.
2. Performance evaluation – After knowing the product characteristics we evaluate its initial performance by laboratory experimental tests.
3. Preparation – Possible degradation mechanisms, degradation factors, degradation indicators and suggest ageing tests.
4. Testing – Ageing tests both in short term and for longer periods are proposed.
5. Performance evaluation – Determine the performance of the aged product on the same characteristics evaluated before (3).
6. Predict service life – Verify that the degradation achieved is similar in both short term and for longer periods testing. If the behaviour under the effects of short term testing is similar to that observed in the longer term the results of the short term testing may be used to predict the service life.

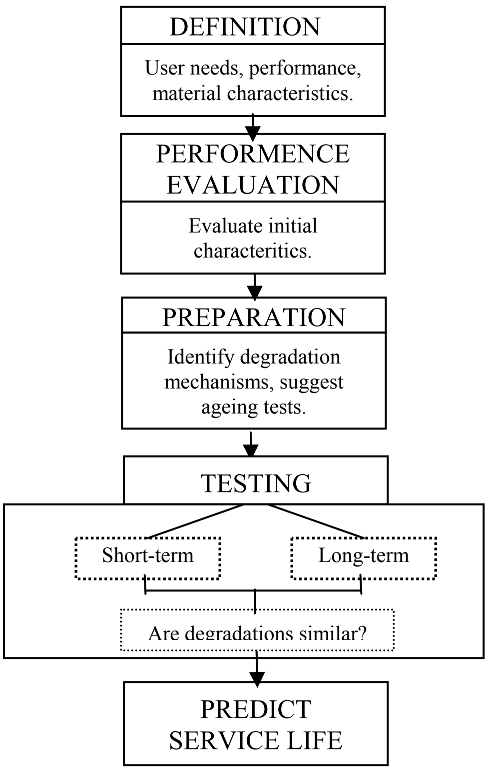


Figure 2. Systematic methodology for predicting the service life of ceramic tile coatings.

4 EXPERIMENTAL STUDY

4.1 Characteristics of the materials

Three different types of ceramic tiles were used in this experimental study. All three types are commonly used for external façade coatings in Portugal. With very distinct absorption coefficients, the ceramic tiles were called L0, L1 and L2. Table 1 and 2 summarises the main characteristics of the ceramic tiles in question.

Designation	*Group*[1]	*With x length* [mm^2]	*Thickness* [mm]
L0	BIa	50 x 50	5
L1	AI	50 x 50	5
L2	BIIa	50 x 50	10

Table 1. Dimensional characteristics of the ceramic tiles.

Designation	*Group*[1]	*Water absorption* [%]	*Flexural strength* [MPa]	*Abrasion resistance* [Mohs]	*Thermal expansion coefficient* [K^{-1}]
L0	BIa	0,02	≥ 27	≥ 6	$\leq 9x10^{-6}$
L1	AI	2,74	≥ 27	≥ 5	$\leq 9x10^{-6}$
L2	BIIa	5	≥ 18	≥ 6	$\leq 12x10^{-6}$

Table 2. Physical characteristics of the ceramic tiles.

The cementitious adhesives used in the study belong to the C2 and C2S classes, the only ones recommended for external application. Table 3 lists the main characteristics of cementitious adhesives.

Characteristics	*C2* [2] [MPa]	*C2S* [2] [MPa]
High initial tensile adhesion strength (after 3 days)	1	-
High initial tensile adhesion strength (after 28 days)	1,5	2
High tensile adhesion strength after heat ageing	1,0	1,5
High tensile adhesion strength after water immersion	0,5	1,0
High tensile adhesion strength after freeze-thaw cycles	-	1,0

Table 3. Characteristics of the cementitious adhesives used.

4.2 Samples Characterization

The tests were performed using 35 concrete slabs measuring 300x200x40mm. Three ceramic tiles were placed on each concrete slab according to the scheme in Figure 3. The 40mm-thick concrete slab, cementitious adhesive, class C2 or C2S, and three ceramic tiles, type L0, L1 or L2, comprise the sample [Fig.3].

[1] Ceramic tile' groups according to European standard EN 14441, *Ceramic tiles – Definitions, classification, characteristics and marketing*, 2003.
[2] Cementitious adhesives for tiles' classification – *Cahier du CSTB 3264*. October, 2000.

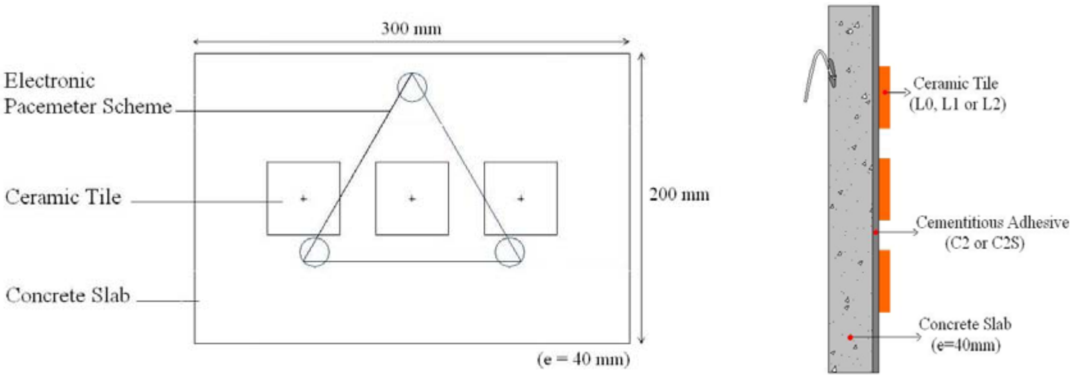


Figure 3. Ceramic tiles placement on the concrete slab.

Table 4 identifies the components comprising the samples prepared for this study.

Samples		*Cementitious adhesives*	
		C2	*C2S*
Tile	*L0*	PE0	-
	L1	PE1	PE2
	L2	PE3	PE4

Table 4. Characteristics of the cementitious adhesives used.

Seven samples were prepared for each type: PE0, PE1, PE2, PE3 and PE4. Each sample was subject to a different number of ageing cycles, varying from 1 to 112. Since one complete cycle lasts 12 hours, the conditioned time limit in the test chamber was of 2 months. Each sample consisted of three ceramic tiles. A total of 105 tiles were tested with a direct pull tensile force.

All samples were prepared under the supervision of a chemical engineer from the cimentitous adhesive manufacturer. We believe to have complied with the most adequate application process which, therefore, did not affect the service life.

4.3 Laboratory accelerated aging program

The accelerated aging program, conducted within an environmental test chamber, was established based on international standards: DS 1127, ASTM D 4798, ASTM E 632, ASTM G 26 and ASTM C481 and the following EOTA documents: Guidance Document GD 003 and Technical Report TR 010.

The established aging test has a total duration of 12 hours (720 minutes) and consists of the 9 steps summarised in Table 5.

Step	*Humidity* [%]	*Temperature* [ºC]	*Time* [min]	*Accumuleted Time* [min]	*Radiation/Rain*
1	95	20	1	1	
2	95	20	139	140	Rain – ON
3	95	20	30	170	Rain – OFF
4	60	-10	140	310	
5	60	-10	60	370	
6	95	50	180	550	
7	95	50	20	570	
8	40	30	140	710	Radiation – ON
9	40	30	10	720	Radiation – OFF

Table 5. Accelerated aging programme. Steps of a complete cycle.

4.4 Tensile adhesion strength tests

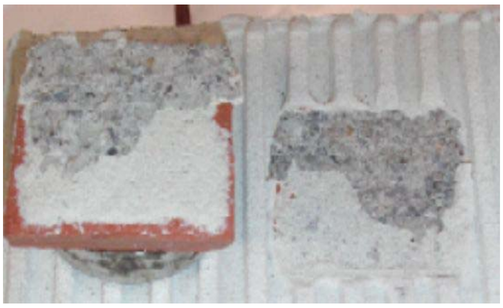
Tensile adhesion strength is the characteristic evaluated in this study that affects the performance of ceramic tile coating systems under investigation. To determine functional changes, each specimen type (PE0, PE1, PE2, PE3 e PE4) was analyzed before entering the environmental test chamber and at the end of the planned exposure times inside the chamber. The tensile adhesion strength tests were performed according to the test method described in European Standard EN 1348 (CEN, 1997).

5 RESULTS

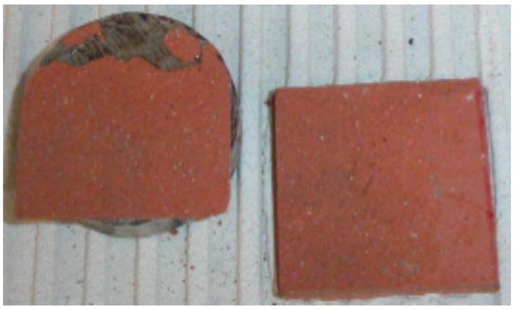
The tensile adhesion strength tests performed for type PE0 samples determined an adhesive failure in the interface between the ceramic tile and the cementitious adhesive [Fig. 4].

Figure 4 – Adhesive failure in the interface between the ceramic tile and the cementitious adhesive Sample PE0.

Cohesive failure was detected in the other tests, especially within the cementitious adhesive. However, in the tests of samples with cementitious adhesive type C2S and after 1 aging cycle, a cohesive failure was detected within the substrate [Fig.5 - PE4.1], and there was only one case where cohesive failure took place in the ceramic tile [Fig.5 - PE4.2].



Sample PE4 .1



Sample PE4.2

Figure 5 – Picture showing two different cohesive failure. Sample PE4.1 - Cohesive failure within the substrate; Sample PE4.2 - Cohesive failure within the ceramic tile.

The graphics in the next two figures show tensile adhesion strength according to the number of aging cycles. The results obtained so far show significant deterioration in tensile adhesion strength, although the critical level of tensile adhesion strength indicated as 0.3 MPa was not reached. Based on the average results, we can compute a tendency indicating the critical value under which the service life can be considered to have terminated.

The graphic in Figure 6 shows the tensile adhesion strength variation under the aging tests according to the number of cycles on specimen types PE0, PE1 and PE3. The tests results from these samples reveal three essential aspects:

1. Regardless of the cementitious adhesive type, about 140 cycles will lead to the end of ceramic tile coating's service life.

2. The failure type was greatly influenced by the type of ceramic tile used. Thus, the failure in the PE0 sample, consisting of ceramic tile with a low absorption coefficient (0.02%), belonged to the adhesive type. Consequently, the indicated values represent the cementitious adhesive adhesion. Nevertheless, the failure in the PE1 and PE3 samples (absorption coefficient > 0.5%) belonged to the cohesive type within the cementitious adhesive, and thus the attained values represent their tensile adhesion strength.
3. After 112 aging cycles we measured about 30% of the tensile adhesion strength's initial value.

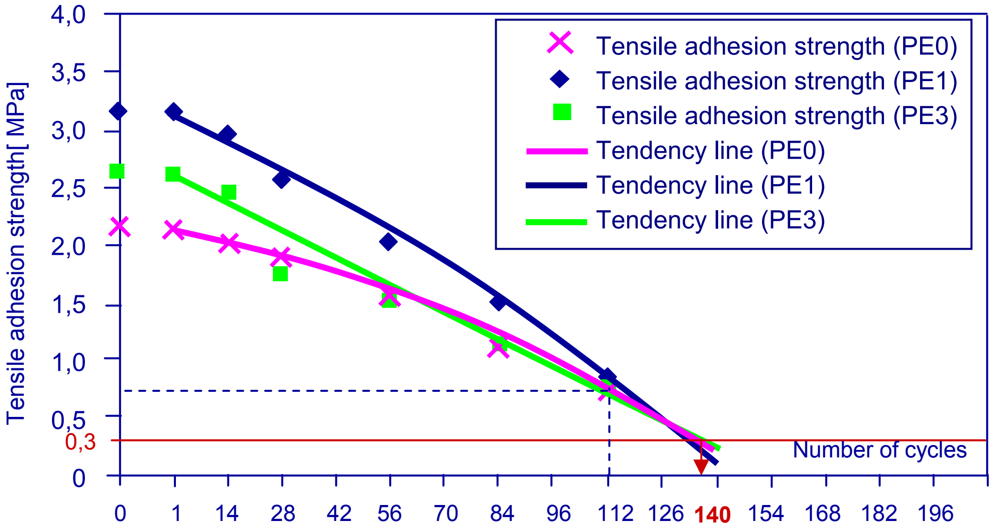


Figure 6 – Change in tensile adhesion strength during aging cycles – Samples PE0, PE1 and PE3 – Cementitious adhesive C2.

The graphic in Figure 7 shows the tensile adhesion strength variation caused by aging cycles according to the number of cycles imposed on specimen types PE2 and PE4. The graphic data reveals three different aspects about the samples consisting of cementitious adhesive type C2S that are worth noting:

1. The critical level was reached at the end of 210 aging cycles, thereby determining the end of the system's service life.
2. The tensile adhesion values of aging cycles 0 and 1 are very similar to the values of aging cycle 14. This can be explained by the observation, in the first case, of a cohesive-type failure within the substrate, whereas in the other situations the failure belonged to the cohesive type within the cementitious adhesive. Whereas cohesive failure within the substrate represents its tensile adhesion strength, the cohesive failure within the substrate indicates only that the cementitious adhesive's tensile adhesion strength is superior to the obtained value but does not indicate its exact value, which may be much higher.
3. After 112 aging cycles, measurements revealed about 50% of the initial tensile adhesion strength.

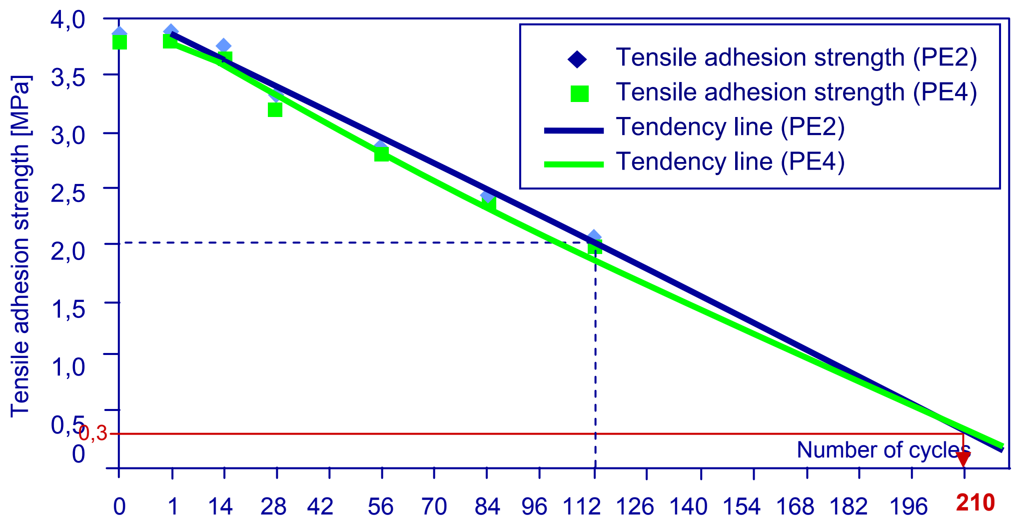


Figure 7 – Change in tensile adhesion strength during aging cycles – Samples PE2 and PE4 – Cementitious adhesive C2S.

6 CONCLUSIONS

The main conclusions are as follows:
1. The results show a substantial loss in tensile adhesion strength.
2. For the established critical value of 0.3 MPa, cementitious adhesives types C2 and C2S terminated their service life at 140 and 210 aging cycles, respectively.
3. The type of failure observed is strongly influenced by the type of ceramic tile used.
4. After 112 aging cycles, measurements indicated about 30% and 50% of the initial tensile adhesion strength for cementitious adhesive types C2 and C2S, respectively.
5. The accelerated aging test values will be compared, in the future, with values obtained from natural outdoor aging tests. The correlation between the results obtained from the two different methods will be necessary to determine the time re-scaling applied to determine the real service life of the ceramic tile coating.
6. The experimental data obtained so far will be very useful to establish an evaluation method to predict durability of cementitious adhesives in relation to the decreasing performance of the tensile adhesion strength and service life of all the coating systems.

7 REFERENCES

Freitas, Vasco Peixoto de; Sousa, Augusto Vaz Serra & Silva, J. A. Raimundo Mendes da; 2003, *Manual de Aplicação de Revestimentos Cerâmicos*, Coimbra.

Maggi, P.N.; Rejna, M.G.; Daniotti, B.; Cecconi, F. Re; Poli, T.; Rigamonti, G.; Jornet, A. & Teruzzi, T. 1999, 'Experimental program to evaluate building components service life: first results on brickwork', Eighth International Conference on Durability of Building Materials and Components 8 dbmc, Vancouver, Canada, May 30 – June 3, 1999, vol. 1, pp. 571-580.

Silva, D.A.; Roman, L.M.F.; Fredel, M.C. & Roman, H.R. 1999, 'Theoretical analysis of thermal stresses of ceramic tile coating systems', Eighth International Conference on Durability of Building Materials and Components 8 dbmc, Vancouver, Canada, May 30 – June 3, 1999, vol. 1, pp. 603-612.

Lucas, José A. Carvalho; 2001, *Anomalias em revestimentos cerâmicos colados*, ICT – ITMC 28. LNEC, Lisboa.

Sá, Ana Vaz; 2005, *Durabilidade de cimentos-cola em revestimentos cerâmicos aderentes a fachadas*, Dissertação de Mestrado em Construção de Edifícios. FEUP, Porto.

Verification of the suitability of shotcrete on low strength concrete surfaces

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ABSTRACT

In Germany shotcretes are approved repair materials for concrete structures requiring a compressive strength higher than 35 MPa. The assessment of hydraulic engineering structures has shown that 50-125 years old concrete sometimes has extremely low compressive strength. Shotcretes for repair are normally used for concrete showing moderate strength, moderate modulus of elasticity and low porosity. However, this approach usually does not achieve a durable repair in hydraulic engineering structures made of low strength concrete.
Manufacturers of repair systems are now developing new products for the repair of old structures made of low strength concrete. These shotcrete for repair and protection are currently being tested at the German Federal Institute for Materials Research and Testing (BAM). Since the compatibility between the shotcrete and the concrete structure is the primary problem these tests have to be carried out on composite specimens. Accordingly, special formulations for low strength concrete that emulated old hydraulic engineering structures were designed at BAM. The two different series of test specimens reached a compressive strength of 15 and 25 MPa, respectively. The repair systems were applied in a layer with a thickness of up to 6 cm on the low strength concrete slabs. In order to be able to observe significant effects that might occur at the interface between the repair system and concrete, the size of the slabs was quite large compared to standard test specimens. Important criteria for the performance and durability of the repair systems are the freeze-thaw resistance, the temperature shock resistance and the performance in changing moisture conditions (under water and under dry conditions). The outcome of the study may lead to modifications of standard test methods.

KEYWORDS

hydraulic engineering structures, low strength concrete, repair systems, performance tests

1 INTRODUCTION

In addition to the international and national standards the Federal Ministry of Transport, Building and Housing drafted guidelines for the construction and the repair of hydraulic engineering constructions. With inception of the new concrete standards EN 206 and DIN 1045, a revision of these guidelines was necessary [Westendarp 2001]. The analysis of 140 hydraulic engineering structures, which were built in the first half of the 20th century, has shown that nearly 60 % of them have weak mechanical properties [Hohmann *et al.* 1994]. According to the new guideline they have to be classified into the concrete classes A2 and A3 [BAW 2005]. This corresponds to a compressive strength of 15 MPa (A2) and 25 MPa (A3) and an adhesive strength of 0.8 MPa (A2) and 1.2 MPa. (A3).
Such structures often show damages close to the surfaces. The dismantling, of these constructions is not necessary in each case. The repair with shotcrete can be a reasonable alternative. Shotcretes with and without additives for the repair of concrete were developed already with normal strength for the application on bridges. Usually they exhibit a high cement content combined with a appropriate grading curve of the aggregate, which causes a dense microstructure with high durability and strength and a high elastic modulus (35 GPa). Such systems are not useful for a durable repair of low strength concrete (elastic modulus 15 – 25 GPa; tensile-strength 0.8 – 2.0 MPa).
With implementation of the new guideline (ZTV-W 219 section 5) [BAW 2005] the repair with shotcrete without reinforcement is regulated. The leaflet describing the test procedures was established on a preliminary basis because no sufficient experience existed. Main objective of the study is the verification of test methods for the evaluation of the suitability and durability of shotcretes repair systems on low strength concrete surfaces.
In the first part of this paper the requirements and stresses of repair systems are described. The second part is focused on the description of the test methods and materials.

2 REQUIREMENTS AND STRESSES

The durable bond between shotcrete-layer and concrete surface is the pursuit objective of the repair. A negative impact by stress from a temperature or moisture gradient is not desirable. The proper adjustment of the chemical and mechanical properties of the repair material is a practical way to acquire compatibility to the old concrete. This entails low strength, low elastic modulus and adjusted thermal displacement. At the same time a high resistance against mechanical, chemical and physical stresses from the environment is required.
At sub-aqueous areas stress is restricted to the influence of fresh- and seawater with small variations in temperature above the freezing point. In the dry zone, above the water surface, strong variations of temperature and humidity occur. The most intense stress induced by a temperature change is found during summertime when thundershowers can cause a rapid cooling of the concrete surface from 40 to 60°C down to below 20°C [Westendarp & Schultz 2000]. Frost damages, however, are only possible if the shotcrete-layer is saturated with water.
The more important stresses take effect at the wet-dry zone. During summertime often a continuous change of slow heating in conjunction with drying of the concrete surface region and rapid cooling during water saturation takes place. In winter time freezing in air and thawing under water are the main risks for damage. For seawater constructions additionally the chloride content has to be considered.

3 VERIFICATION AND SUITABILITY OF TEST PROCEDURES

Fundamental studies at the BAM, funded and supported by the Federal Waterways Engineering and Research Institute BAW and assisted by producers of repair-systems are being implemented. At first concrete formulations with properties of the "old" concrete had to be developed. The next steps were the definition of the suitable specimen geometry and the types of stress applied to the compound specimens. These considerations and first results were used to establish the leaflet mentioned above.

3.1 Concrete formulation for compound specimens

At BAM the concrete formulation for the specimens for the recreation of the classified concretes A2 and A3 found in the old constructions were developed. In the original concretes very large aggregates were used, which are unfavorable in mechanical tests (e.g. tensile strength). As a compromise the maximum grain size is limited to 32 mm. For technological aspects a grading curve C 32 was chosen.
In the past very coarse grained cements were used for such constructions. Nowadays these cements are not available anymore. An ordinary CEM I 32,5 is the most practical substitution. In table 1 the formulations derived from the results of an extensive study of mixed proportions of historical concretes are shown.

mixture	**A2**	**A3**
CEM I 32,5 [kg/m³]	170	210
aggregates* [kg/m³]	1980	1974
water [kg/m³]	150	149
w/c-ratio	0,88	0,71
compaction	approx. 1,25	approx. 1,22
air content [%]	approx. 5	approx. 4

*The aggregates have to be composed from at least 5 grain size classes

Table 1. Formulations and fresh concrete properties for the concrete classes A2 and A3 [BAW 2005]

After demoulding the specimen were submerged in water for 7 days followed by exposure in air. The objective of this curing was an early strength development with constant properties after 56 days.

3.2 Properties of the recreated “old” concrete

In table 2 the 28 days compressive strength of the concretes (cubes) is shown. Additional tests were carried out with three cements from three different producers.

mixture	**A2**	**A3**
required strength	**15 ± 3**	**25 ± 3**
cement 1	15,8	25,5
cement 2	16,0	22,5
cement 3	16,1	22,8

Table 2. Resulting compressive strength at the age of 28 days using CEM I 32,5 R of different producers

The static elastic modulus measured on cylinders with a diameter of 15 cm and a height of 30 cm was 18 GPa (A2) and 21 GPa (A3). Both concretes are frost-resistant according to the CIF-test known as **C**apillary suction, **I**nternal damage and **F**reeze thaw test [BAW 2004].
The mixing procedure and the compaction are very important factors with regard to the concrete quality. Therefore recommendations were developed. A special test of suitability of the bond is necessary in any case. The knowledge about the quality of the correlation between adhesive and compressive strength is needed for the test procedure because of the estimation criterion the bond between old concrete and shotcrete. The tensile strength or the adhesive strength, depending on the layer thickness, are the values used later on for characterizing the quality of the bond between concrete and shotcrete layer. At first the tensile strength and the adhesive strength of the untreated concrete specimens and untreated concrete surfaces were quantified and correlated with the compressive strength. The scatter decreased with decreasing strength. Figure 1 shows the results of the investigations carried out at BAM on concretes for repair systems.

The concrete type A2 had an adhesive strength of about 1 MPa, the concrete type A3 of about 1.5 MPa. Consequently these concretes were suitable for testing the required adhesive strength of 0.8 MPa and 1.2 MPa at the age of 28 days for the repair mortars, which were classified in the guideline as SA2 and SA3.
The compressive strength of drilled cylinders taken from the concrete slabs (10 cm diameter) is significantly lower (25 %) than the compressive strength of cubes produced with the same material. This effect is probably due to the better compaction of the cubes and the coarse aggregates of 32 mm with regard to the slab-thickness of 10 cm.

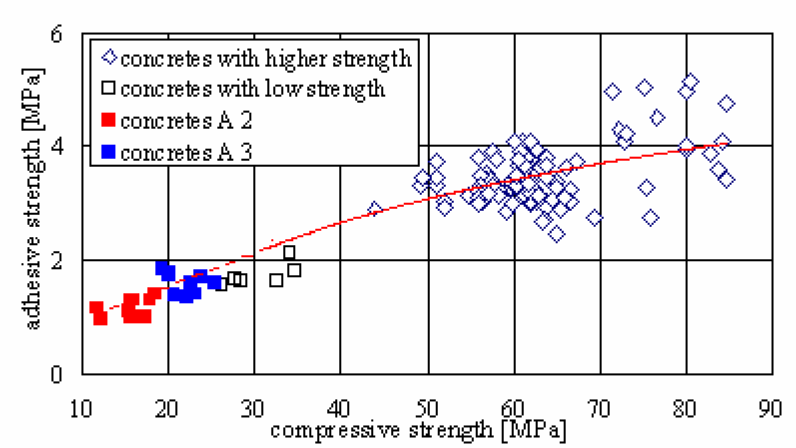

Figure 1. Adhesive strength of the untreated concrete surfaces correlated with the compressive strength.

3.3 Specimens

Specimens for the test of the thermal compatibility comprised of shotcrete with a maximum layer thickness of 6 cm on weak concrete, which had to be as thick as possible in order to avoid influences from the reverse side. The bondsurface has to be very extended in order to deal with possible occurring tensions. Considering handling and performance “old”-concrete specimens with a thickness of 10 cm, a width of 30 cm and a length of 50 cm with a weight of about 60 kg proved to be the best choice. These concrete slabs were roughened by sandblasting at the surface and were mounted vertically into a rack (Fig 2).

Figure 2. Vertical fixed specimens with formwork for a layer thickness of 6 cm.

3.4 Repair systems

Two manufactures were involved in the validation of the new guideline. They developed special repair systems for the concrete classes A 2 and A 3. Because of cost reasons the manufacturers decided to develop only one system for the two concrete classes. Fig. 3 shows the application of a wet spray mortar.

The manufacturers sprayed their systems for the production of the test specimens on the slabs produced at BAM. For the test program one suitable wet-spray mortar, one suitable dry-spray mortar and one dry-spray mortar, which proved to be incompatible, are worked up. The incompatible dry-spray shotcrete was used unsuccessfully in the past on a dam wall and was chosen for comparing the performance of the other two. This system showed extensive crack formation and large areas of spalling. The concrete slabs sprayed with this incompatible repair system are supposed to fail in the performance tests.

Figure 3. Application of the wet sprayed mortar (thickness 6 cm) takes place at the manufacturer.

For the examinations about 7 tons of the mortars were sprayed at 5 processing dates. The systems are based on portland cement. The approximate composition of the compatible systems is given in table 3.

shotcrete	wet system	dry system
cement	CEM I 42.5 R	CEM I 32.5 R
mineral addition	fly ash, micro silica	limestone
admixture	present	present
fibers	PP (4 vol.%)	---
max. grain [mm]	4	8

Table 3. Composition of the shotcrete

For shotcrete with a normal and a high quality (SA4 elastic modulus > 25 GPa) the requirements of the guideline RL SIB [DafStb 2001] are used. For the weaker systems called SA2 an elastic modulus ≤ 15 GPa is required and for the SA3 the ZTV-W LB 219 demands an elastic modulus ≤ 25 GPa. These values are corresponding with the elastic modulus of the concrete classes A2 and A3. For a suitable and durable repair the elastic modulus of a shotcrete have to be similar to the elastic modulus of the concrete. In addition to that the thermal extension, swelling and shrinkage are important factors. The results of the test program ought to proof the appropriateness of the leaflet. The thermal expansion of the mortars was measured within the framework of the basic investigation.

3.5 Types of stress

The combined assessment of thermal expansion and elastic modulus should be sufficient for the evaluation of the thermal compatibility between sprayed systems and concrete. The thermal compatibility of the high-strength systems SA4 is tested by a thermal shock treatment in saturated saline solution. This test provides sufficient results in a reasonable period. The weaker systems will be tested more realistically close to practice. For the development of such a test method extensive investigations are necessary. The adhesion strength (layer thickness 2 cm) and the tensile strength (layer thickness 6 cm) of the composites are the decisive test criteria.

Data of existing structures are necessary for the choice of realistic test conditions. Field tests of the BAW at hydraulic engineering structures (sluice walls) yielded a maximum thermal gradient of 6 to 8 K/h for cooling when exposed to air. Furthermore the thermal profile showed that in massive structures the maximum penetration depth of frost is about 10 cm.

The natural conditions at the construction were taken as a model with regard to the expected thermal stress. The experimental conditions included freezing in cold air reaching the temperature gradient mentioned above and thawing under water. According to the present state of knowledge stress in the wet-dry zone of water buildings represents the maximum thermal influence. At the same time a similar temperature gradient as in structures from the outside to the inside of approx. 20 K on a stretch of 10 cm had to be simulated, which is obtained by freezing the test slabs from one side.

Therefore a heat-flow-calculation of the heat insulated concrete slab stored in a top-loading freezer was carried out. The high heat conductivity of the concrete and the low heat transmission from the surrounding air to the surface was considered. The results showed that it was not possible to generate a sufficient temperature gradient with the used experimental setup without heating. By implementing a heater at the backside of the concrete slab the setup for the calculation was changed. The results of the new calculations have shown useful effects. A suitable weathering chamber where the slabs could be submerged and cooled from one side and heated from the other was not available. Figure 4 shows the experimental setup with the heat insulated and sealed concrete slab. First investigations were carried out on test bodies in a top-loading freezer.

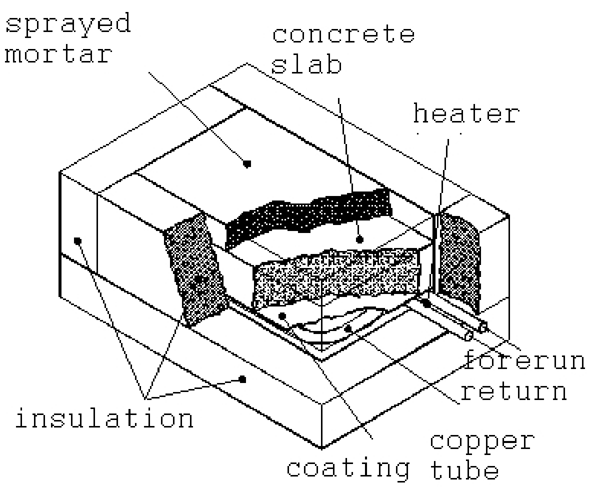

Figure 4. Schematic presentation of the experimental setup - 10 cm thick concrete slab with applied 6 cm shotcrete, coating and heat-insulation and backside heater.

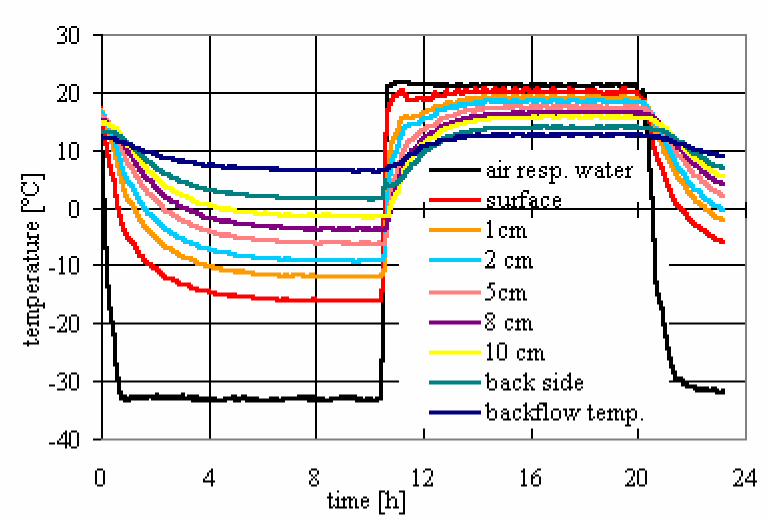

Figure 5. Resulting temperature sequences for one freezing and defrosting cycle in different measuring depth. - 10 cm concrete slab with applied 6 cm spray mortar, coating and heat-insulation and backside heater.

The experimental setup was changed, according to the calculated effects by the implementation of a heating system at the backside of the slab.
In order to obtain a temperature profile thermocouples were installed in different depth in the test body. By using a suitable feedback control in the top-loading freezer, the evaluation of the temperature yielded the required temperature distribution shown in figure 5.
The temperature at the surface changed between 20 °C and -15 °C while the backside of the slab remained at an almost constant temperature of approx. 5 °C.
Therefore a simulation of the temperature distribution at the real building and the resulting thermal tensions is possible for the composite specimen examination.

For the test procedure implemented in the new guideline 25 freeze-thaw-cycles as shown in fig. 5 are required. The tensile strength of drilled cores of the test slabs and the adhesive strength of the sprayed mortar are the assessment criteria. The requirements for sufficient systems repair systems with sufficient performance are shown in table 4.

shotcrete	S-A2	S-A3
tensile strength [MPa]	mean value ≥ 0.8	mean value ≥ 1.2
layer ≥ 4 cm	lowest value ≥ 0.5	lowest value ≥ 0.8
adhesive strength [MPa]	mean value ≥ 0.8	mean value ≥ 1.2
layer ≤ 4 cm	lowest value ≥ 0.5	lowest value ≥ 0.8
crack width [mm]	≤ 0.1	≤ 0.1
visual examination	no erosion	no erosion

Table 4. Requirements on the properties of shotcrete after thermal stress for the concrete classes A2 and A3 [ZTV-W LB 219 2005]

The complete suitability test on repair systems contains the determination of many additional characteristic properties of the raw materials, of the fresh concrete and of the hardened shotcrete (moulded and sprayed). In addition further tests on composite specimens are required. The tests ascertain: flexural strength, compressive strength, swelling (28 d), shrinkage (28 d), apparent density, static elastic modulus, resistance in $Ca(OH)_2$, capillary water absorption, durability against stress induced by wetting-drying, carbonation depth, chloride penetration resistance, CDF and CIF tests, wear resistance, adhesion strength or tensile strength by centric tensile tests after different strains.

Nevertheless the aims of a repair with shotcrete are a suitable and durable bond between old concrete and shotcrete and a surface without or with only a few cracks. The investigations of the full-scale tests are still in progress and results are expected within a short time period.

4 CONCLUSIONS

In the past repair tests with shotcretes on old dam walls or flood barriers have failed due to the incompatibility of concrete and shotcrete.
At present no certified shotcretes for repair of concrete with low compressive and tensile strength are commercially available. The new German guideline ZTV-W LB 219 shows in its section 5 for the first time possibilities to fill this gap. The additional leaflet describes the test procedures on a preliminary basis.
The objective of the study was the development of suitable test conditions that simulate actual stresses at hydraulic engineering constructions. Special formulations for low strength concrete that emulated the characteristic properties of material found in old hydraulic engineering structures were designed at BAM.
The newly developed repair systems are applied on the low strength concrete slabs. The approach for testing the adhesion strength of shotcrete on weak concrete with low compressive strength considers all significant types of stress. It is attempted to calibrate the examinations with an incompatible repair system. The test results may lead to a modification of the test methods in the leaflet.

The introduction to the market of such new repair systems may represent an economical alternative to the usual cost-intensive repair method with reinforced concrete shells.

5 ACKNOWLEDGMENTS

We should like to thank 'The Federal Waterways Engineering and Research Institute' (BAW) for financing and supporting the project.

6 REFERENCES

BAW 2005, ZTV-W LB 219 'Protection and repair of concrete structures' *Federal Waterway Engineering and Research Institute,* Karlsruhe.
BAW 2004,'Instructions Frost test of concrete'*, Federal Waterway Engineering and Research Institute,* Karlsruhe.
Westendarp, A. 2001, 'Concrete repair for hydraulic engineering - new requirements and developments', *Mitteilungsblatt der Bundesanstalt für Wasserbau* Nr. 83, Karlsruhe, pp. 51–57.
Westendarp, A.& Schulze, M. 2000, '*Frostbeanspruchung von Verkehrswasserbauwerken*'*,* Beton, Verlag Bau +Technik, Düsseldorf, 5/2000 pp. 260-266.
Hohmann, W & Lehmann, H. & Lehmann K. 1994, '*Beurteilung des Betons von Schleusen des westdeutschen Kanalnetzes*', Betoninformation Verlag Bau +Technik, Düsseldorf , vol. 1.
DAfSTb 2001, 'DAfStb-Instandsetzungs-Richtlinie: Schutz und Instandsetzung von Betonbauteilen. Teil 1: Allgemeine Regelungen und Planungsgrundsätze. Teil 2: Bauprodukte und Anwendung. Teil 3: Anforderungen an die Betriebe und Überwachung der Ausführung. Teil 4: Prüfverfahren'. *DAfSTB* Ausgabe Oktober, Berlin.

Concrete Deterioration Caused by Sulfuric Acid Attack

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ABSTRACT

Recently, biological deterioration of concrete in sewage and wastewater treatment plants has been reported. This deterioration is caused by sulfuric acid attack and is dependent on the concentration of sulfuric acid, this in turn being a function of both the specific location within the plant and also the time over which the concrete is exposed to elevated concentrations of acid. Given that concrete may often be exposed to very strong acid solutions, resin coatings are applied to the concrete to protect them. However these coating are only effective if there are neither pinholes nor defects due to coating operations. Otherwise, the deterioration of the concrete may proceed rapidly by sulfuric acid and sulfate. The method of predicting deterioration of coated concrete when subjected to sulfuric acid and sulfate attack has not yet been established. It is understood that the depth of deterioration of concrete due to sulfuric acid attack is proportional to the square root of the exposure time. This relationship is based on results of laboratory tests where specimens are continuously immersed in acid solutions over a specfied period of time. But in actual structures, concrete is often exposed to flowing water that contains an acid solution such as sewage. In other words, concrete is subjected to the shearing force of fluid that erodes the surface of areas of deteriorated concrete. In these instances, it is predicted that concrete deterioration caused by sulfuric acid attack will proceed at a greater rate than that predicted from the square root relationship. Hence in this study concrete deterioration caused by sulfuric acid attack was investigated considering the effects of the flow of acid solution over the surface of concrete with the intent of proposing a prediction method for the deterioration of concrete due to sulfuric acid. Cylindrical concrete specimens and mortar prisms were immersed in various concentrations of sulfuric acid. In certain tests the sulfuric acid solution was circulated onto concrete specimens. In both instances, the depths of zones eroded and neutralized by acids were measured. As well, the zones of deteriorated concrete were analysed with an XRD and an ion chromatoanalyzer. It was found that the rate of concrete deterioration caused by sulfuric acid attack depended on the pH value of acid solutions and that the depth of erosion of concrete was nearly proportional to the exposure time of flowing acid solution to which concrete was exposed.

KEYWORDS

Concrete, Chemical Deterioration, Acid Attack, Sulfuric Acid, Flowing Water

1 INTRODUCTION

Sulfuric acid solution in sewage, wastewater treatment plants and hot spring places deteriorates concrete structures hard by reacting with cement hydrates. Concrete is not a chemically stable material under the condition of acidic environment. Although acid attack is one of the primary chemical deterioration of concrete for many years, the prediction method for this kind of deterioration has not been established yet. In Japan, the JSCE (Japan Society of Civil Engineers) Standard Specification [2001] mentions about countermeasures against chemical deterioration and mentenances, but does not descirbe the prediction method.

As a study of the prediction method for deterioration by sulfuric acid attack, many kinds of immersion tests using mortar specimens have been carried out. But actual structures are made from concrete and concrete deterioration due to acid attack should be predicted. Those studies are not so much so far.

It is generally said that the deterioration depth of concrete due to acid attack including sulfuric acid attack is proportional to the square root of the exposure time, according to Sakamoto [1972], Pavlik [1994], and so on, as follows:

$$y = b\sqrt{t}$$

where, y : the deterioration depth (mm)
t : the exposure time in acid solution (year)
b : constant (mm/$\sqrt{year}$).

In this equation, the rate of deterioration is governed by the diffusion rate of acid in deteriorated zones from concrete surfaces to non-deteriorated concrete under the assumption that corrosion products keep to remain on concrete surfaces. In actual deteriorated structures of sewage and wastewater treatment plants, however, it is found that deterioration products have been removed from concrete surfaces and coarse aggregates are exposed on surfaces. This should be because the deterioration products of mainly gypsum are subjected to the shearing force of flowing water and/or splashing water.

Usually experimental studies on concrete deterioration due to sulfuric acid attack are performed using specimens statically immersed in acid solutions. In this case, because concrete surfaces are not subjected to the shearing force, deterioration products will remain on the surfaces. It is concerned that the results obtained from these experiments could evaluate lesser deterioration than actual deterioration of sewage and wastewater treatment plants subjected to the shearing force of folwing water.

In this study, in order to clarify the effects of the shearing force of flowing water on the concrete deterioration due to sulfuric acid attack, concrete and mortar specimens are immersed in sulfuric acid solutions that are circulated by pumps. This circulation generates the flow of solutions and the shearing force of the flow is applied to the surfaces of the specimens.

2 EXPERIMENTAL PROCEDURES

2.1 Materials and specimens

Ordinary Portland cement was used as cement. Blast furnace slag and fly ash were used as mineral admixtures. Concrete cylinders of ϕ150×300 mm in size and mortar prisms of 40×40×160 mm in size were made with water binder ratios of 0.35, 0.50 and 0.65. The replacement ratio of cement with mineral admixtures was 0.30. The propoerties of materials used in this study and the mix proportions are shown in **Table 1** and **Table 2**, respectively. Comparing the chemical resistivity at a certain age, it

Table 1. Physical properties of materials used in this study

Cement	Ordinary Portland cement Density : 3.16 (g/cm^3), Blaine fineness : 3080 (cm^2/g)
Mineral admixtures	Fly ash Density : 2.30 (g/cm^3), Blaine fineness : 4160 (cm^2/g)
	Blast furnace slag Density : 2.91 (g/cm^3), Blaine fineness : 6220 (cm^2/g)
Coarse aggregate	Iwase crushed stone Maximum Size : 20 (mm), Density (SSD) : 2.65 (g/cm^3) F.M. : 6.52, Water absorption : 0.74%, Solid content : 61.2%
Fine aggregate	Ogasa crushed sand Density (SSD) : 2.61 (g/cm^3), F.M. : 2.82, Water absorption : 1.44%

Table 2. Mix proportions and compressive strengths of specimens used in this study

	Mark	*Binder*	*Percentage of Replacement (%) (Replaced by Mass)*	*W/□*	*Compressive Strength [N/mm^2] (Age: days)*
Concrete	NC35	Ordinary Portland cement	—	0.35	65.5 (28)
	NC50		—	0.50	43.5 (28)
	NC65		—	0.65	32.6 (28)
	BS30-65	Ordinary Portland cement + Blast furnace slag	30	0.65	36.3 (42)
	FA30-50	Ordinary Portland cement + Fly ash	30	0.50	43.1 (91)
	FA30-65		30	0.65	33.7 (417)
Mortar	NC35	Ordinary Portland cement	—	0.35	47.2 (28)
	NC50		—	0.50	41.3 (28)

is supposed that specimens containing mineral admixtures should be weaker than those containing no mineral admixtures because of slow hydration. In this study, therefore, specimens containing mineral admixtures were cured until those compressive strengths became almost equal to specimens containing no mineral admixtures with the same water cement ratio cured for 28 days. It is assumed here that the pore structures in specimens are almost same if compressive strengths are same with a certain water cement ratio whether mineral admixtures are contained or not. The test results of compressive strength of specimens are also shown in **Table 2**.

2.2 Experimental method

The concentrations of sulfuric acid solutions in immersion tests were 2.0 mol/L for concrete specimens and about 0.09 mol/L (adjusted to pH=1.0) and about 0.06 mol/L (adjusted to pH=2.0) for mortar specimens. The immersion tests contain two types of Method 1 and Method 2. In the Method 1, specimens were immersed statically in sulfuric acid solutions and in the Method 2 they were immersed in sulfuric acid solutions that were circulated with a pump and flowed in a tank (**Table 3**). To keep the concentration of sulfuric acid solution constant, sulfuric acid was added suitably to the solution. At this time, a pH meter was used to measure the value of pH and the concentration of sulfuric acid solution was adjusted to the initial value of pH.

After the immersion tests were started, the erosion depth was measured with a vernier micrometer every seven days. The erosion depth is defined as a distance between the initial surface and current

Table 3. Methods for immersing specimens in sulfuric acid solutions

Abbreviation	*Detail*
Method 1	Specimens are immersed statically in sulfuric acid solution in a tank.
Method 2	Specimens are immersed in a tank where sulfuric acid solution is circulated by a pump. (Specimens are subjected to the shearing force of the flow of the solution.)

surface. Before every measurement, intentional removal of deteriorated zones on the surfaces was not carried out.

After specimens were immersed for certain periods, mineralogical changes in the specimens were analyzed with an XRD and the distribution of sulfate ions in the specimens was measured with an ion chromatoanalyzer.

3 RESULTS AND DISCUSSIONS

3.1 Effects of the flow of fluid on erosion

The erosion depth of concrete specimens immersed in 2.0 mol/L of sulfuric acid solution with the Method 1 and Method 2 is shown in **Fig. 1**. For specimens with every water cement ratio, the erosion depth with the Method 2 was larger than with the Method 1. Especially it is remarkable for specimens of NC50 and NC65 after 60 day immersion. As shown in **Fig. 2**, reaction products of gypsum remained on the surfaces of concrete specimens immersed with the Method 1, while any products could not be seen on the surfaces of concrete specimens immersed with the Method 2. The reaction

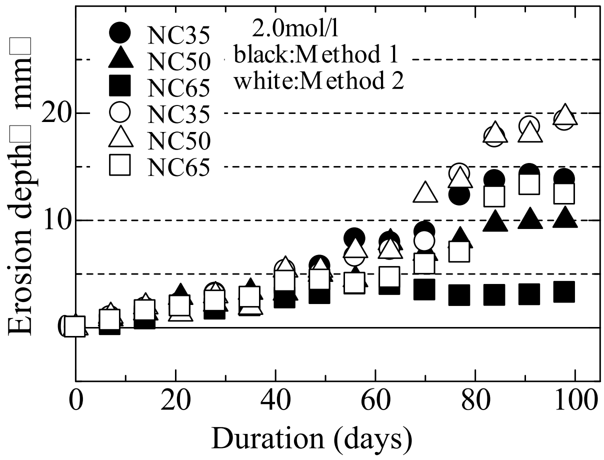

Figure 1. Erosion depth of concrete immersed in 2.0 mol/L of sulfuric acid solution

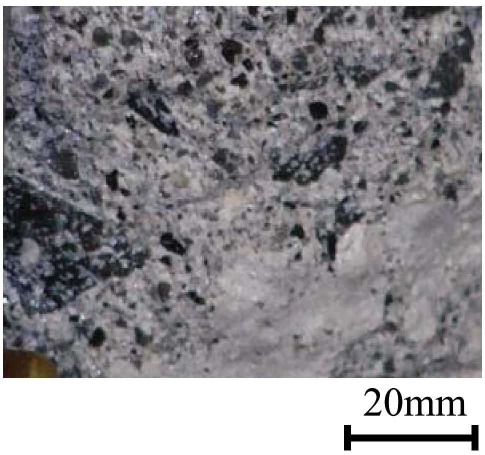

Figure 2. Reaction products on the surface of concrete specimen NC50 immersed in 2.0 mol/L of sulfuric acid solution

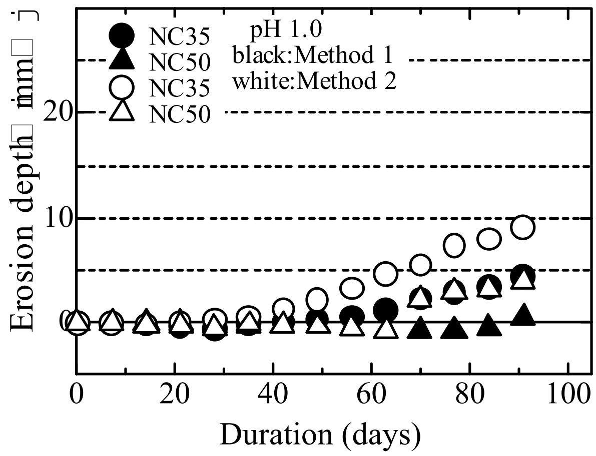

Figure 3. Erosion depth of mortar immersed in sulfuric acid solution of pH=1.0

products on the surfaces of concrete specimens immersed with the Method 2 should be driven out by the flow of fluid without being precipitated on the surfaces. As a result, sulfuric acid always attacks concrete surfaces without the necessity of diffusing into deteriorated zones. Reaction products could not be seen on the surfaces of concrete specimens NC35 immersed with the Method 1. Because of dense microstructures, it is thought that reaction products of gypsum could not be precipitated in microstructures and was driven out by the flow of fluid.

The erosion depth of mortar specimens immersed in ca. 0.09 mol/L (pH=1.0) of sulfuric acid solution with the Method 1 and Method 2 is shown in **Fig. 3**. For NC35, erosion of specimens can be seen after 49 day immersion with the Method 1 and after 28 day immersion with the Method 2. For NC50, erosion of specimens immersed with the Method 2 can be seen after 63 day immersion. The XRD patterns of surface areas for mortar specimens of NC50 immersed for 63 days are shown in **Fig. 4**. Gypsum was clearly identified for specimens immersed with both the Method 1 and Method 2. These results show that reaction products of sulfuric acid attack are mainly gypsum. On the other hand, as shown in **Fig. 5**, for mortar specimens of NC35 immersed in ca. 0.09 mol/L (pH=1.0) of sulfuric acid solution for 63 days no reaction products can be seen on the surface of the mortar specimen immersed with Method 2, while a layer of reaction products can be seen on the surface of the mortar specimen immersed with the Method 1. Since reaction products were precipitated on the surface when the specimen was immersed statically, reaction products would have been seen on the surface also in the case of Method 2. But the flow of the solution swept reaction products on the surface.

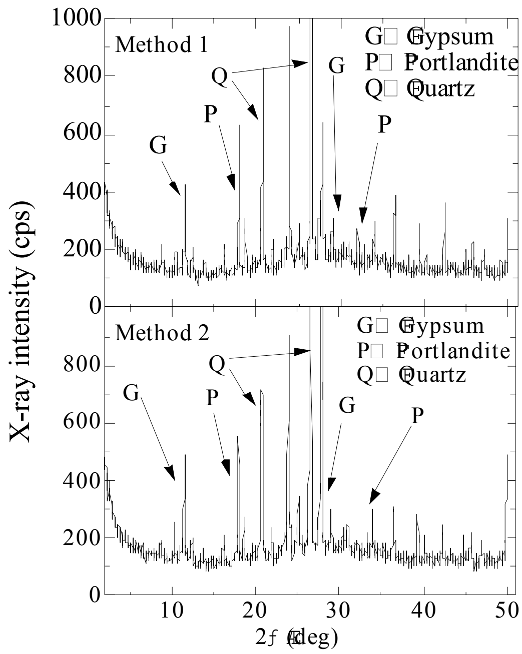

Figure 4. XRD patterns of surface areas for mortar specimens at the age of 28days (NC50)

The erosion depth of mortar specimens immersed in ca. 0.06 mol/L (pH=2.0) of sulfuric acid solution with the Method 1 and Method 2 is shown in **Fig. 6**. Every specimen immersed in sulfuric acid solution of pH=2.0 was hardly eroded but swelled a little. Reaction products were found on

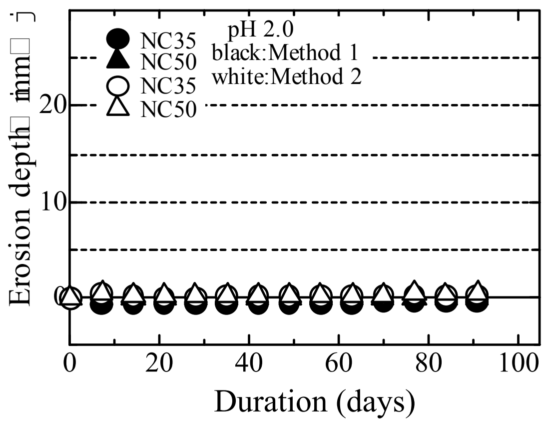

Figure 6. Erosion depth of mortars immersed in sulfuric acid solution of pH=2.0

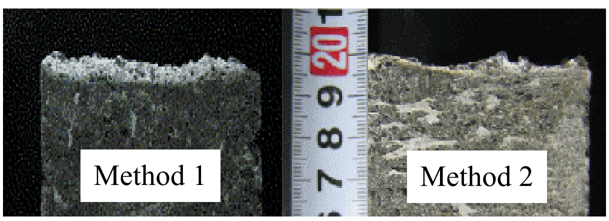

Figure 5. Deterioration of NC35 immersed in sulfuric acid solution of pH=1.0 for 63 days

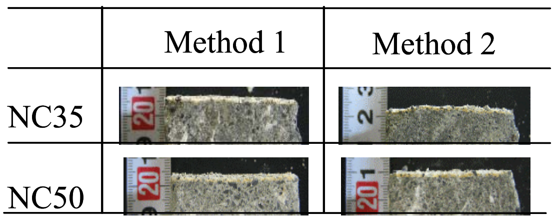

Figure 7. Deterioration of NC35 and NC50 immersed in sulfuric acid solution of pH=2.0 for 63 days

every surface of the specimens as shown in **Fig. 7**. But compared with specimens immersed in ca. 0.09 mol/L (pH=1.0) of sulfuric acid solution shown in **Fig. 5**, reaction products were not remarkable. The specimens immersed in ca. 0.06 mol/L (pH=2.0) of sulfuric acid solution were not reacted heavier than those immersed in ca. 0.09 mol/L (pH=1.0) of sulfuric acid solution.

Regarding the mechanism of concrete deterioration caused by sulfuric acid, Kurashige [2002] described that sulfuric acid penetrating into mortar or concrete reacts with calcium hydroxide of cement hydrates to produce gypsum and at this time the volume of solid substances increases largely, which causes expansion of reaction products resulting in erosion. This phenomenon is schematically illustrated in **Fig. 8**. Concrete with a high water cement ratio has larger and more pores than that with a low water cement ratio. These pores play the role of a capacity to absorb expansion caused by the production of gypsum. Therefore concrete with a high water cement ratio has a higher capacity to absorb the expansion of production reaction of gypsum than that with a low water cement ratio, that is to say, concrete with a low water cement ratio erodes earlier than that with a high water cement ratio and its erosion depth is nearly proportional to the exposure time instead of the square root of the exposure time. The results of this study also agreed with this mechanism. It is shown that specimens with a lower water cement ratio eroded heavier regardless of the Method 1 and Method 2 and the erosion depth was nearly proportional to the exposure time.

Ueda et al. [1996] pointed out that sulfuric acid is hard to penetrate into hardened cement. As shown in **Fig. 9**, sulfate ions did not penetrate very much into mortar or concrete also in this study. The reaction of cement hydrates and sulfuric acid should occur only in the surface portion of specimens. This would be because the reaction of production of gypsum in the surface portion is faster than the penetration of sulfate ions into the specimen. The surface portion, therefore, is a main field of the reaction of sulfuric acid. That is why specimens immersed with the Method 2 eroded much larger than those with the Method 1 since the flow of solution removed the reaction product of gypsum.

3.2 Effects of the addition of mineral admixtures on erosion

The erosion depth of concrete specimens with mineral admixtures immersed in 2.0 mol/L of sulfuric acid solution with the Method 1 is shown in **Fig. 10**. The erosion depth of specimens containing blast furnace slag and fly ash was smaller than that of specimens containing no mineral admixtures. This is because the production content of calcium hydroxide in concrete

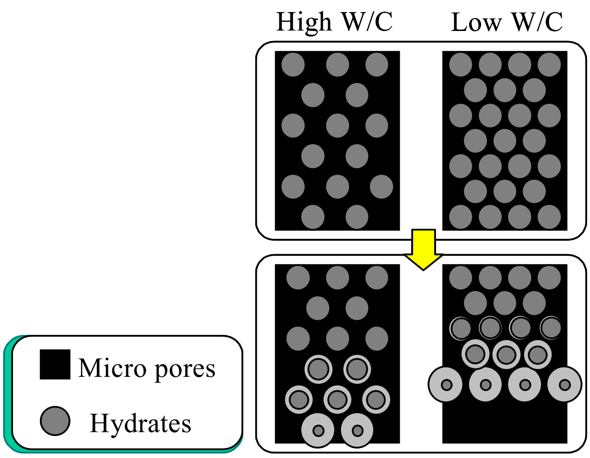

Figure 8. Mechanism of concrete deterioration due to sulfuric acid attack

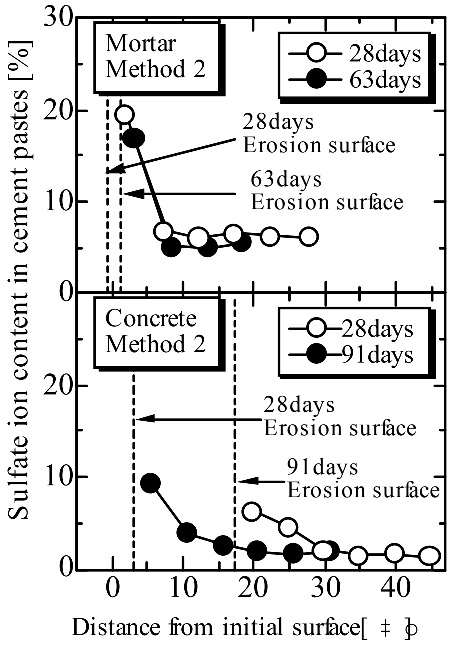

Figure 9. Sulfate ion content in cement paste portion of concrete and mortar (NC35)

containing mineral admixtures was lower than that in concrete containing no mineral admixtures since a part of cement was replaced with the mineral admixtures. As for concrete containing fly ash, since calcium hydroxide produced by cement hydration was consumed by the pozzolanic reaction, the erosion depth was smallest among concrete specimens with the same water cement ratio. Consequently the calcium hydroxide content in concrete is closely related to the volume increase due to the production of gypsum which causes the deterioration and is one of the factors governing the degree of concrete deterioration caused by sulfuric acid.

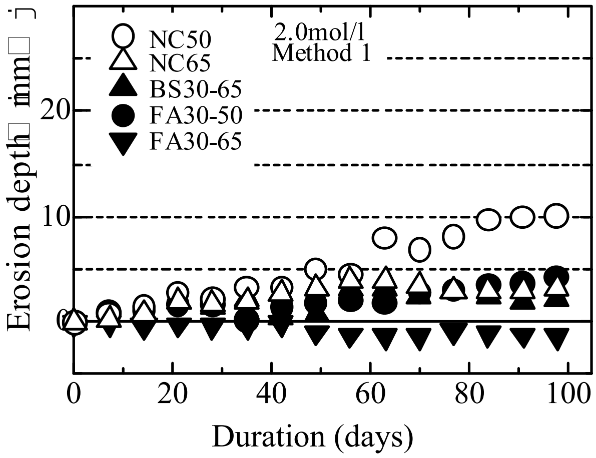

Figure 10. Erosion depth of concrete containing blast furnace slag and fly ash immersed in 2.0 mol/L of sulfuric acid solution

4 CONCLUSIONS

This study was perfomed to understand the mechanism of concrete deterioration caused by sulfuric acid. The effects of the flow of fluid, the concentration of sulfuric acid solution, the use of mineral admixtures and the difference of water cement ratio on the deterioration of concrete were investigated. As a result, the following conclusions were obtained.

1. Regarding concrete deterioration caused by sulfuric acid, the flow of fuid accelerates the deterioration and the rate of deterioration of concrete caused by sulfuric acid strongly depnds upon the concentration of sulfuric acid solution.. In an elevated concentration of sulfuric acid solution, the erosion depth of concrete is nearly proportional to the exposure time instead of the square root of the exposure time. Sulfate ions does not penetrate into concrete very much and the reaction of cement hydrates and sulfuric acid occurrs in the surface portion of concrete.

2. When a part of cement is replaced with blast furnace slag or fly ash and the strength of concrete containing the mineral admixture is almost equal to the strength of plain concrete with the same water binder ratio, the erosion depth of concrete containing mineral admixture due to sulfuric acid attack is smaller than that of plain concrete since the content of calcium hydroxide is small.

5. REFERENCES

JSCE, 2001, JSCE Standard Specifications for Concrete Structures [Mentenance], Japan Society of Civil Engineers, Japan. (in Japanese)

Sakamoto, H. 1972, 'Studies on resistance of cement mortar to acids', *Civil Engineering journal*, Vol.14, No.8, 38-44. (in Japanese)

V. Pavlik. 1994, 'Corrosion of cement paste by acetic and nitric acids - Part 1: Calculation of corrosion depth', Cement and Concrete Research, Vol.24, No.3, 551-562.

Kurashige, I. 2002, *Mechanism and Prediction Method for Deterioration of Concrete Due to Sulfuric Acid*, a doctoral dissertation thesis in Tokyo University, Japan. (in Japanese)

Ueda, H., Takata, J. & Tatematsu, H. 1996, 'Deterioration mechanism of hardened cement paste by various acids', Proceedings of the Japan Concrete Institute, Vol.18, No.1, 879-884. (in Japanese)

The evaluation of damage due to salt crystallisation of different re-pointing mortars studied in laboratory

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ABSTRACT

Efflorescence and crypto-efflorescence due to the crystallisation of soluble salts are among the most frequent causes of damage to facing brickwalls. Mortar joints are the mostly affected in the case of historic masonry and re-pointing is one of the techniques more frequently used for repairs.

Under the frame of an EC Contract, a series of crystallisation tests was carried out on brick masonry specimens to control the effectiveness of the different re-pointing.. The salt used was sodium sulphate in a saturated solution. The mortars used for re-pointing were based on: hydrated lime, hydraulic lime, white cement and mixed hydraulic lime + white cement. Two different particle size distributions of the aggregate were used in order to evaluate the influence of this parameter on the damage, taking into account that also mixing of the mortar constituents, compaction and the quality of the hardening process, of course, could influence the results. The long term damage caused by the salts was detected by visual inspection, photographic survey and measured by a laser profilometer. The laser device allows for measurements along chosen profiles on the surface masonry, quantifying with the variation of the profile depth over time, the loss of surface material.

The effectiveness of the four different re-pointing types was evaluated and is here presented.

KEYWORDS

Mortar joint re-pointing, salt crystallisation, masonry durability

1 INTRODUCTION

In a masonry wall, moisture due to capillary rise and/or rain penetration, is the vehicle through which soluble salts can migrate inside the material and the process of evaporation takes the salts toward the exposed surfaces of the walls; salts crystallising behind the surface cause delamination and crumbling of the masonry components and particularly to the weakest one i.e. the mortar. Re-pointing is a rear technique which is applied to masonry when the mortar joints are affected by damage. When re-pointing is carried out without any knowledge about the original materials (mortar and bricks) or when the choice of the material for re-pointing is not appropriated then very soon partial detachment or push out of the re-pointing can occur. The inappropriate or inaccurate use of re-pointing can lead to serious damage not only of the pointing itself but also of the original masonry components. Recurrent damages have been observed as: partial detachment of the re-pointing when poor workmanship is used, push out in case a very dense cement pointing mortar had been applied on materials with high porosity as old bricks and lime mortar [Lubelli *et al.*2001]. Taking into account that re-pointing is usually carried out on historic buildings, further damages should be avoided also in name of preservation of the integrity of the building. When choosing a mortar for re-pointing of existing masonry the first step must be an on site investigation on the causes of the damage and on their effects on the masonry. A research carried out by some of the Authors within the frame of a previous EU Contract lead to the production of a Damage Atlas [European Commission 1998] and of an Expert System to assist the professional people [Van Balen *et al.* 1997]. The second step has to be the characterisation in laboratory of the most representative materials (mortars, bricks, stones) present in the masonry to be re-pointed. It is known in fact that a large part of masonry of historic buildings is not homogeneous, but the results of several repair and modifications in the course of the centuries. Therefore [Baronio & Binda 1991] at the end of the investigation the results will help in the choice of the most appropriate intervention and of the materials for re-pointing. Within the frame of a European Contract (EC Contract ENV4-CT98-0706) the partners pointed out all the types of damage occurring after re-pointing due to different causes. Furthermore as the aim of the contract suitable types of pointing mortars were proposed and their compatibility was verified by accelerated laboratory tests [Cardani G. *et al.* 2001]. The extent of damage has been measured in different ways, from the visual one to the use of a laser profilometer. On the basis of the recorded experimental data, a suitable damage parameter describing the material deterioration process was chosen when using the profilometer. The parameter assumed is the loss of material from the surface at each measurement. The measurements have been made through the laser device along chosen profiles on the masonry surface. Therefore, the loss of material is quantified as the variation of the profile depth over time.

2. DAMAGING PROCESS

The effect of salt damage to the mortar joints appear frequently as cracking or loss of material in the joints ‘Fig.1a’. When pointing or re-pointing is carried out on existing masonry where capillary rise of water and salt solution are present, if the feeding of salts is not stopped or reduced, repointing can be soon damaged by cryptoefflorescence underneath the re-pointing. There might be also the possibility that non-compatible mortars are chosen; in those cases a poor bond is realised between pointing/repointing and bricks/stones or pointing/repointing and mortar joints. When the re-pointing is too strong compared to the original mortar and bricks, then either the new joint is pushed out or the brick can be damaged or both ‘Fig.1b’. In some cases the workmanship is very poor and the consequence is that when moisture and salts are not stopped, the re-pointing is very soon lost (even after few weeks) ‘Fig.1c’. The only way for carrying out friendly repair by re-pointing is to reach a deep knowledge of the characteristics of the damage, of its causes, of the original bricks and mortars characteristics and then to carry out tests on some available mortars for re-pointing in comparison with the original ones in order to choose the most appropriate material. Ageing crystallisation tests can help allowing for choosing the most durable combination brick-jointing mortar- re-pointing mortar.

Figure 1. Efflorescence on bricks and loss of mortar joints before re-pointing; b) effect of incompatible pointing mortars in case of good adhesion and c) in case of bad adhesion.

3. EXPERIMENTAL RESULTS

The aim of the research was to test the compatibility with the original bedding mortar and bricks of selected pointing mortars in presence of salt crystallisation processes. Some masonry prisms (wallettes) have been built using "hand made" bricks and hydrated lime mortar joints, simulating old masonries. These prisms were pointed with different types of mortars. It was stated within the above mentioned EC Contract that "the best re-pointing material is the one which does not cause further damage to masonry support and hence, in case of aggression from the environment, becomes a sacrificial part". So it is not so much important that the re-pointing material is more durable that the original ones but that it does not cause any further damage to the masonry. For example, an excessive bond strength and stiffness of this material can cause damage to the re-pointed joints and around them.

3.1 Preparation of the specimens and description of the test

Salt crystallization tests were carried out according to the Recommendation RILEM MS A.1 of RILEMTC127MS [RILEM MS-A1, 1998]. With respect to the RILEM test some changes were adopted concerning the salt solution concentration and re-wetting of the specimens. The wallettes (250x200x120 mm) were put in contact with their back side with a salt solution of Na_2SO_4 at the chosen concentration of 12.5% (w%), 'Fig.2a', and then stored over a layer of dry gravel in a plastic container (open at the top) with the upper face exposed to the environment (controlled laboratory environment of 20°C and 50% R.H.) 'Fig.2b'.

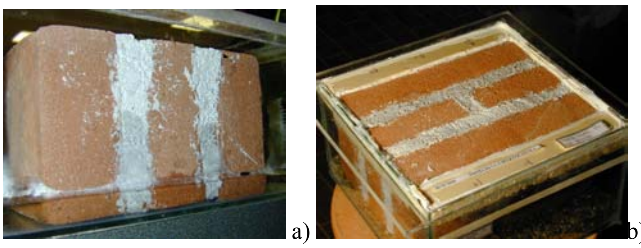
a) b)

Figure 2a,b. a) Different salt solution absorption in brick/mortar system and b) Evaporation upper surface of a wallette at the beginning of the test

It can be noticed from 'Fig. 2a' the different level of absorbed solution in the bricks and in the mortar. Each four weeks the wallettes were subjected to a) visual inspection, b) cleaning from efflorescence and detached materials with a soft brush and a vacuum cleaner, c) photographic survey, d) description of the observed damage, e) reading the surface profiles by means of a laser profilometer which allowed to quantify the damage. De-mineralised water was added every three months, when approximately the wallettes were approaching constant mass and the previous controlling steps were repeated. Each wallette consisted of three courses of bricks with two horizontal bed joints 'Fig.3a' and in some case a vertical one. A red soft-mud brick of high porosity (36 % in volume, 23% in weight), used for restoration was selected. The bedding mortar was a putty lime similar to the ones frequently used in the Lombardia Region in Italy and similar to the one sampled from an Italian case study in the EU project (Cascina Rossa in Milan). Two different types of pointing mortar were firstly used considered compatible with the substrate: hydrated lime mortar (L) and a hydrated lime+white cement mortar (LWC). Both were made with the same siliceous-calcareous aggregate but with a grain size finer (0.075-2.75 mm) than the aggregate used in the bedding mortar (0.075-4.75 mm) 'Fig.3a,b'.

Figure 3a,b. Preparation of the brick masonry wallettes: a) bedding and b) of the pointing

Some specimens were the joints were simply tooled and not pointed were used as reference. Some months later other wallettes were prepared with other re-pointing materials (white cement and hydraulic lime), each with different aggregate size: the one used for the first re-pointing cases and the one of the bed joint mortar in order to check the influence of the aggregate size. In Table 1 the composition of the mortars and the material combination are described.

Units	Bedding mortar	Pointing mortar	
Softmud red bricks	putty lime and siliceous + calcareous aggregate (binder/sand ratio 1:3) grain size 0.075-4.75mm	tooled bedding	TB
		hydrated lime and siliceous + calcareous aggregate (binder/sand ratio 1:3 ,grain size 0.075-2.75mm)	L
		hydrated lime + white cement (2:1) and siliceous + calcareous aggregate (binder/sand ratio 1:3, grain size 0.075-2.75mm)	LWC
		White cement and calcareous aggregate (binder/sand ratio 1:3, grain size 0.075-2.75mm)	WCs
		White cement and calcareous aggregate (binder/sand ratio 1:3, grain size 0.075-4.75mm)	WCl
		Hydraulic lime and calcareous aggregate (binder/sand ratio 1:3, grain size 0.075-2.75mm)	HLs
		Hydraulic lime and calcareous aggregate (binder/sand ratio 1:3, grain size 0.075-4.75mm)	HLl

Table 1. Material used for the laboratory masonry specimen

3.2 EXPERIMENTAL INVESTIGATION

Salt crystallisation started immediately after the salt was added to the specimens, due to the high quantity of salt solution used in the test. Due to the fact that the test is very severe, all the wallettes were damaged. Therefore only by comparison of the damages it was possible to sort the most appropriate re-pointing.

Visual inspection. In all the wallettes with tooled bedding mortar (TB) and re-pointing the damage to the bricks after two months was similar: detachment of thin layers 1 mm thick. Sanding was also evident in all joints. In the case of the re-pointed joints there were differences in the quantity of efflorescence at the top of the joints. Much higher for the joints re-pointed with hydrated lime mortar (L), lower for the joints re-pointed with lime-white cement mortar (LWC), hydraulic lime (HL) and white cement mortar (WC). In general the wallettes were completely covered by the salts with fluffy consistence, with a maximum at the brick/mortar interface, much evident in case of re-pointing. The damage on the bricks started from the edges with powdering proceeding to the centre with exfoliation. This was connected with the fact that that the mortar joint has a much higher absorption than the bricks 'Fig 2a'. From the results of this first crystallization cycle it can be observed that the bond of the pointing made with materials chosen as compatible with the bedding mortar, i.e. with the same ratio binder/aggregate, is good, also in case of the use of white cement combined with lime. In fact, no pushing out of the pointing was detected for all the wallettes; the accelerated process of salt crystallisation and the decay gives uniform damage for all brick/mortar systems for 6 months. After 6 months, the different decay due to crystallisation was more clear as the different behaviour of the pointing and tooling was more defined.

In the wallettes with tooled bedding mortar (TB), the decay increased regularly in time. It was evident that the tooled joints were more damaged than the bricks. At the end of the test (11 months) the thickness of the lost materials reached the value of approximately 8mm for the mortar and 5,5 mm for the brick. In the wallettes with L pointing, the decay became more regular over time, after a first period of bulging (1 month). The thickness of lost material in the joint is less than in the TB wallettes and LWC and WCs mortar: approximately 2.5mm for the mortar and approximately 2.8mm for the bricks, but after 4 months the decay is visibly higher in the pointing mortar than in the bricks, ending at 11 months with a thickness of lost material of 6 mm for the pointing mortar and 3,8mm for the brick. In the wallettes with LWC pointing, the decay is regular without preference of a specific component and compared to the damage observed for L pointing mortar, the presence of cement in the pointing mortar seems to increase the damage of the bricks in the same period of time. Usually, as mentioned before, the decay of the bricks started where the highest capillary flow takes place, i.e. at the brick/joints interface. In case of TB mortar, this process is very fast, so after one month all the bricks surface is equally damaged. The presence of pointing decreases this phenomenon, without avoiding it, so that after 5 months it is also visible in all the wallettes.

Observing the wallettes with a pointing made of only white cement (WC) or hydraulic lime (HL), the effect of these binders is evident: no damage of the pointing was observed and bricks in the first months presented just a very small powdering. Nevertheless at the end of the test irrregular damage was observed on bricks with WCs pointing. Finally they reached the highest damage even compared to the TB and LWC re-pointing.

__Decay measurements.__ Figure 4a,b shows the measurements realised with the laser profilometer device. The presence of the typical swelling phenomenon of the surface can compromise the damage measurements. Since bulging is the previous step before detachment, when salt crystallises under the surface, it is possible to consider it as the starting point of a damage. Therefore, through a simple model the experimental measurements with the laser profilometer have been converted in deterioration diagrams where the bulging has been eliminated, but considered as a starting point of decay [Garavaglia E. *et al*. 2002]. By calculating at each survey the percentage of the area loss of the specimen section, normal to the profile, area loss against time can be represented 'Fig.5'.

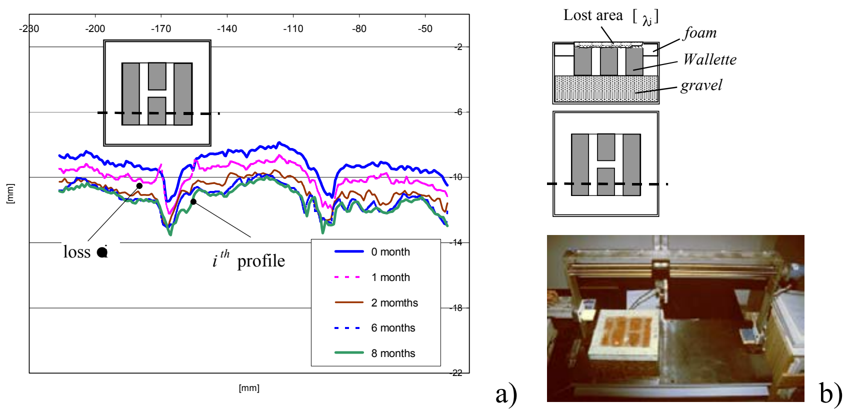

Figure 4. a) example of the first measurements realised with the laser profilometer on one wallette; b) Laser profilometer device during measurement and the box scheme

Figure 5 shows the elaboration of the results from the damage measurements using the profilometer on the whole wallette. In this quantitative evaluation it is clearly visible the comparison among the different pointing effects on wallettes and how the minor decay is attributed to the hydrated lime and hydraulic lime pointing mortar. After 11 months the crystallisation test on this first series of wallettes can be considered ended as the damage, although the insertion of new water, does not increase, because an equilibrium with the environment is reached (20°C and 50% R.H.)

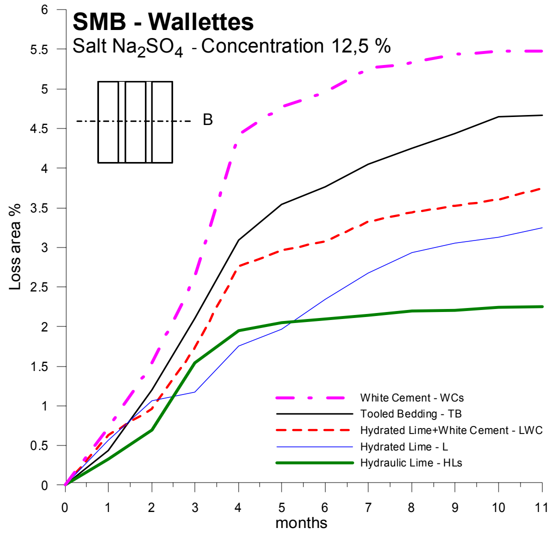

Figure 5. Average lost area in masonry wallettes for each type of pointing used

4. DISCUSSION OF THE RESULTS

Taking in to account that the severe conditions of the test always lead to a damage, the loss of material measurements on wallettes by laser profilometer could be analysed showing the effect of the salt crystallization on the masonry single components ‘Fig.6a,b’. When a large damage was reached as in tooled bedding wallette (B) but also in Lime (L), LWC, HL and WC pointed wallettes, large part of the decay is related to the pointing mortar, showing thus an attempt to preserve the original bricks and to be a sacrifice surface. On the contrary the bricks appear more damaged with a cement based pointing mortar (WC and LWC), showing that these are the two less compatible re-pointing materials since they are not respecting completely the requirements that the mortar re-pointing should be the sacrificial material after the intervention. ‘Fig. 7a’ in fact shows clearly the different behaviour of the two materials at the end of the test in case of White Cement pointing.

After some crystallization cycles, the weakest part of some masonry wallettes is clearly visible, as shown in ‘Fig. 7b’: the brick-mortar interface, where after the adhesion is lost, the evaporation can continuosly take place producing lack of adhesion and detachment. This effect is in laboratory evident due to the position of the wallettes, layed horizontally, but it can happen also in reality on the external surface of the wall.

The obtained results also show how the aggregate grain-size distribution in pointing mortar influences the brick masonry decay. In the WC pointing case, the highest decay ‘Fig.8a’ is due to a finer maximum aggregate (0.75-2.75mm) (WCS). On the contrary for the wallettes pointed with hydraulic lime (HL) better results were showed by finer aggregate ‘Fig.8b’.

The test was stopped after 11 months, then, after 33 months water was added again to the boxes and the profilometer measurements were started again. The results show that very little damage was caused that is the damage tends to an asymptotic value when the salt feeding is stopped and most of salts have practically all dried out by evaporation.

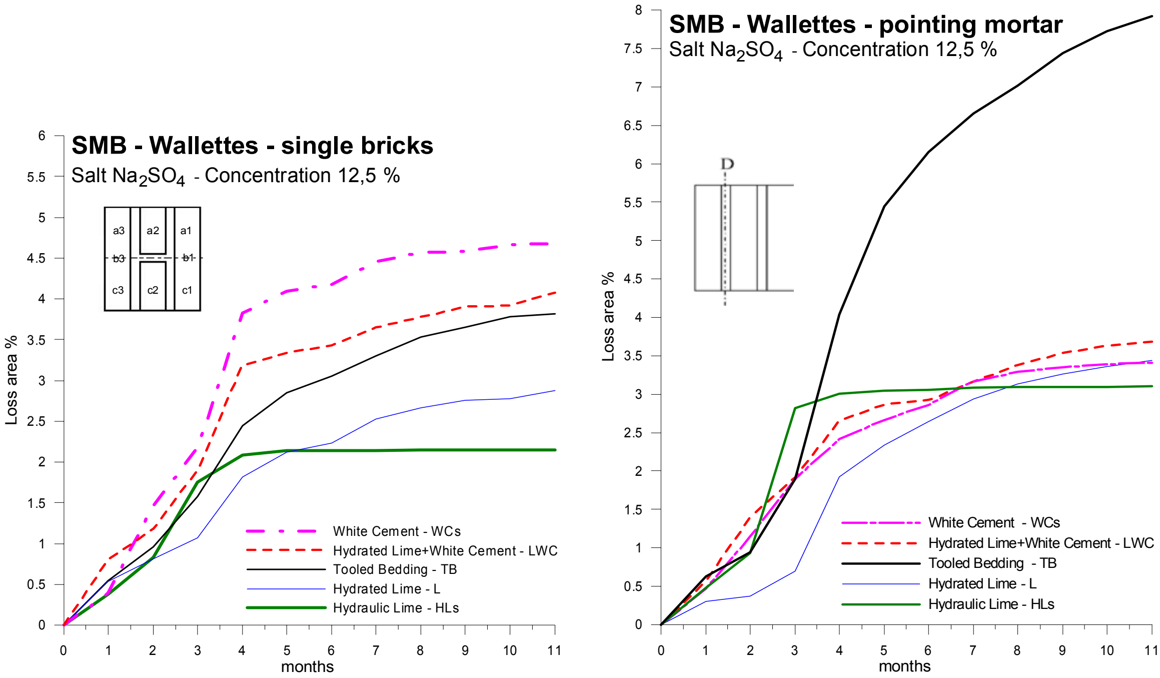

Figure 6. Average lost area measured only on the bricks (a) and only on the mortar (b) in the masonry wallettes

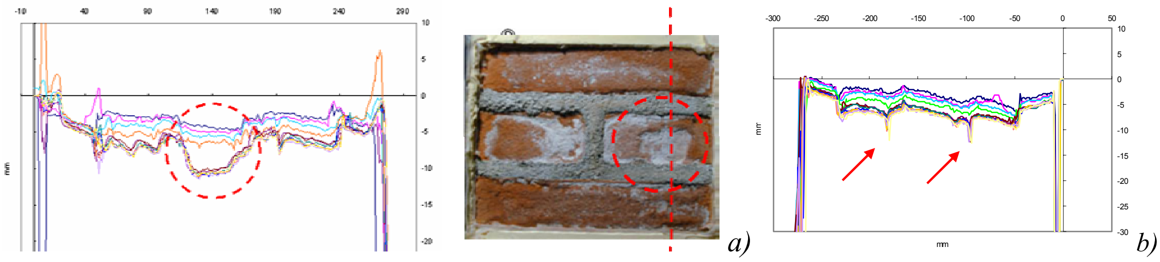

Figure 7. ***a)*** **The loss of the original brick substrate due to the salt crystallization process, in case of cement based pointing;** ***b)*** **Detachment of the interface revealed by the laser profilometer**

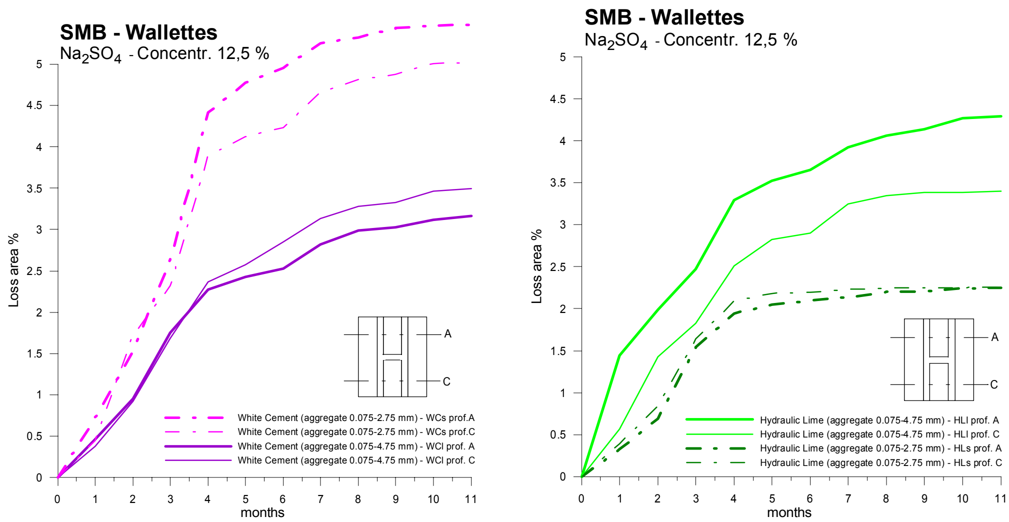

Figure 8. Avarage lost area in masonry wallettes for two types of pointing using different grain-size distribution: a) for White Cement mortar wallettes and b) Hydraulic Lime mortar.

CONCLUSION

The aim of the experimental research described in this paper was to determine the compatibility, with respect to salt crystallisation, of different types of pointing to the same bedding mortar/brick substrate. The pointing mortars were chosen in order to be suitable for the substrate, but with different chemical, physical and mechanical properties in order to study possible differences in the damaging process, with minor damage for the hydrated lime re-pointing.

It can also be remarked that the laser profilometer shows that the higher decay occurred to the tooled bedding joint, without repointing, and confirms the good performance of the pointing as a repair technique.

The following comments can be made:

- Salt crystallization starts immediately after salinization due to the severe testing condition and progressively decreases in the next three months when the most quantity of water evaporates. When new water is added the decay is increased until the phenomenon started.
- In all cases of pointing the damage on the bricks started from the edges, at the interface with the mortar joint, with powdering proceeding to the centre with thin exfoliation.
- The lowest average decay is reached by pointing with hydrated lime and with hydraulic lime.
- From the results it can be observed that at the beginning the bond of the different pointing made with different materials was good, also in case of use of white cement combined with lime or alone, but at the end of the test some detachments were observed. Usually good behaviour is found when the size of the aggregates is smaller than the one of the bedding, except in one case (WCs).
- The bricks of the wallettes pointed with lime-white cement suffered the greatest loss together with the ones pointed with white cement and finer aggregates

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REFERENCES

Baronio G. & Binda L. 1991, *Experimental approach to a procedure for the investigation of historic mortars,* 9th Int. Brick/Block Masonry Conf., Berlino, Vol.3, pp. 1397-1405.

Cardani G., Tedeschi C., Binda L. & Baronio G. 2001, *Laboratory Crystallisation Tests for Damage Evaluation, Maintenance of Pointing in Historic Building: Decay and Replacement*, N. Contract EV-CT98-0706, Final Report to EC, paper 5.3, CD-Rom, pp.99-107.

European Commission 1998, *Damage Atlas - Classification and Analyses of Damage Patterns Found in Brick Masonry, Protection and Conservation of European Cultural Heritage*, Project EV5V-CT92-0108 Research Report N° 8, Fraunhofer IRB Verlag, 2, ,

Garavaglia E., Cardani G., Binda L. 2002 *Application of a Probabilistic Model for the Prediction of the Decay Due to Salt Crystallisation of Masonry Building Materials*, Proc. Int. Conf. 9th Durability of Building Materials, Brisbane, 17-20/03/2002

Lubelli B. & Hees R.P.J Van 2001, 'The role of pointing mortar in the damage due to salt crystallisation', in Proceeding of the congress STREMAH VII, pp. 537-547,.

RILEM MS-A1 1998, *Determination of the resistance of wallettes against sulphates and chlorides*.

Van Balen K., Mateus J.M., Binda L., Baronio G., Van Hess R.P.J., Naldini S., Van Der Klugt L.J.A..R. & Franke L., 1997, *Scientific Background of the Damage Atlas and the Masonry Damage Diagnostic Sistem,* Final Report, Contract EV5V-CT92-0108.

Mass Transfer Resistivity over Time of Repair Materials for Concrete under Cyclic Accelerated Environmental Conditions

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ABSTRACT

Damaged reinforced concrete structures brought about by the corrosion of steel bars may be repaired by materials such as polymer modified cement mortars (PCM). To affect an optimally repaired system some basic properities of the repair material should be known, such as the in dimensional compatibility with existing concrete, overall dimensions of the repair, permeability, and chemical and electrochemical properties. However, there is little information about the performance of repairs and repair materials over time. In this paper, changes in such properties as water diffusivity, air permeability, and capillary water absorbtance of PCM repair materials were investigated under cyclic accelerated environmental conditions that included moisture and drying, and high and low temperature change. The results are discussed on the basis of change in porosity of the repair materials. The patterns of degradation of PCM could be classified in two categories: (i) the deterioration of the matrix, composed of cement mortar and polymer, and; (ii) damage of the surface polymer film.

KEYWORDS

Repair Materials, High-Low Temperature Change, Dry-Moisture Change, Resistance to Mass Transfer, Pore Distribution, Polymer Film

1 INTRODUCTION

The reinforced concrete structures which were constructed in the 20th century have now become very important social assets, and they are maintained to ensure their usability. Such structures, however, are changing due to various reasons as time goes by, and the maintenance work becomes more difficult.To evaluate the performance of repair methods, first of all, it is important to identify the changes of the physical properties of the repair materials with time accurately and it is essential to quantify the durability analysis parameters of repair materials to review the durability performance of the repaired buildings in an analytical way. However, it is hard to say that the research and information on the secular variation of the various performances of the repair materials is sufficient at the present day.
This study was conducted to identify the secular variation of mass transport resistance under the deterioration accelerating environment focusing on the patch repair materials containing polymer because it is important to select the appropriate repair method and materials after defining the durability of repair materials so as to optimize the maintenance plan of a building within an intended lifecycle allowing the long-term use of the building. The performance required from patch repair materials includes the sufficient bond to a substrate, similarity of the electrochemical property and movement property of deterioration factor to that of a substrate, and maintenance of the steel corrosion resistance around a repair area. Given the deterioration phenomena of the patch repair materials containing polymer under the deterioration accelerating environment, we will discuss the mass transport resistance, a micro behavior of patch repair materials containing polymer on the basis of the change of micro-physical properties, that is, the change of the fine pore size distribution or total porosity from the perspective of the a composite material, a polymer-contained cement mortar, not from the perspective of the chemical deterioration of a polymer itself.

2 TEST MEHTODS

We cured the polymer-contained patch repair materials via air drying for 28 days, and accelerated deterioration by repeating high/low temperature and dry/moisture condition and measured the changes in total porosity and a fine pore size distribution. We also measured a mass transport resistance (water diffusion coefficient, air permeability coefficient, capillary absorption coefficient) and compared it with the change of fine pore structure, so as to review the change in the performance of polymer-contained patch repair materials under the condition where high/low temperature and dry/moisture are repeated.

2.1 Materials Used and Mixing

In this study, 9 types of repair materials were used for the test as shown in Table 1.

2.2 Deterioration Accelerating Environmental Condition

Symbol	***Patch repair materials***	***Mixing***
OPM	Portland Cement Mortar	W/C=55%, S:C=1:2.5
CR	Cation-CR Polymer Modified Cement Mortar	P/C=10%□Fiber 0.5%
ASAL	Acrylic Styrene Acetate Modified Lightweight Cement Mortar	P/C=5%□Blast Furnace Slag
NAM	Non shrinkage Cement Mortar	Expanding Agent, Blast Furnace Slag
EVA	EVA Polymer Modified Cement Mortar	P/C=2%□Fiber 5%
EVAL	EVA Polymer Modified Lightweight Cement Mortar	P/C=2%□Fiber 5%
SBR	SBR Polymer Modified Cement Mortar	P/C=7%□Fiber 1%
SBRL	SBR Polymer Modified Lightweight Cement Mortar	P/C=7%□Fiber 1%
PAE	PAE Polymer Modified Cement Mortar	P/C=4%□Fiber 1.2%

Table 1. Materials Used and Mixing

Condition	***Initial Measurement***	***1st Measurement***	***2nd Measurement***	***3rd Measurement***
Air Dried Cure **(K)**	After Air Dried Cure 28days **(F)**	After 30days	After 60days	After100days
High-Low Temperature Change **(T)**		After 30 cycles	After 60 cycles	After 100 cycles
Dry-Moisture Change **(H)**		After 10 cycles	After 20 cycles	After 30 cycles

For example, T2 represents the result of 2nd measurement of the test body under high/low temperature repeated condition.

Table 2. Deterioration Accelerating Environmental Condition and Measurement

In the test, we measured the initial values after 28 days of air-dried cure at the environment of 20 ℃and 60% (R.H.) so that the change in the performance of patch repair materials may converge to acertain level. Then, as shown in Figure 1, we exposed the specimen to the environment where the change of high-low temperature is repeated suppressing the influence of moisture and where the change of dry-moisture situation is repeated suppressing the influence of temperature, and measured the result 3 times following the initial measurement as shown in Table 2.

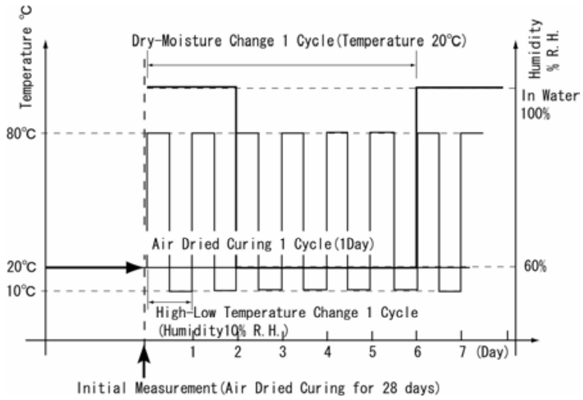


Figure 1. Deterioration Accelerating Environmental Condition

2.3 Measurement Items and Method

In this study, we conducted a series of tests to evaluate the secular variation of the performance of the polymer cement mortar by reviewing the physical change, that is, the change in porosity and fine pore size distribution so as to compare the result with the change in the mass transport factor.

2.3.1 Change in Compressive Strength

We measured the compressive strength according to JIS A1172 (Method of testing the strength of polymer cement mortar).

2.3.2 Change in Porosity and Mass Transport Resistance

The porosity of the porous materials such as the cement mortar can be accessed by the amount of water absorption or a mercury intrusion porosimetry. In case of a mercury intrusion porosimetry, however, we cannot say that its measurement result represents the total porosity since its measurable pore diameter is about 0.003-375 micro meter[C.Galle 2001]. In addition, since the measurement result of air permeability coefficient and water diffusion coefficient is determined by the short-period pressure gradient due to the characteristics of the test, it depends more on the connected big porosity than on the change in capillary porosity. Accordingly, in this study, we determined the total porosity via measurement of water absorption , and compared it with air permeability coefficient and water diffusion coefficient. The measurement method of physical properties are discussed below.

(1)Measurement of Porosity Porosity is computed by the formula (1) after dipping an absolutely dried specimen into a water and measuring the change in weight until saturation point.

$$\phi_w = \frac{m_s - m_d}{v\rho} \qquad (1)$$

where, ϕ_w :Porosity， m_s :Weight of Saturated specimen(kg), m_d :Weingt of absolutly dried specime(kg), ρ :Density of water at 20℃(kg/m^3)

<u>(2)Measurement of Air Permeability Coefficient</u> The concept of the air permeability coefficient is illustrated in Figure 2 [S. Tsivilis *et al.* 2003]. The size of the specimen (test body) is diameter 100 x 30

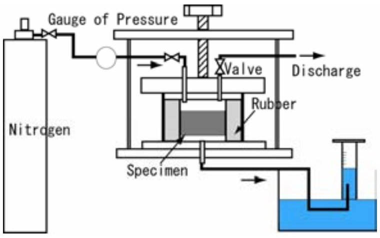


Figure 2. Testing device of Air Permeability Coefficient

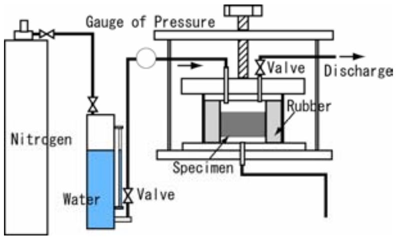


Figure 3. Testing device of Water Diffusion Coefficient

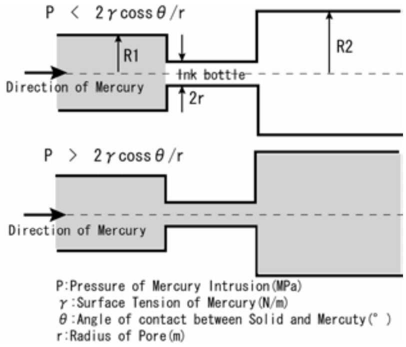


Figure 4. Concept of ink bottle phenomenon

(mm). The air permeability coefficient was computed by the formula (2) by measuring the amount of the air which passed the specimen in a given period of time after pressing a nitrogen gas with 0.5 MPa.

$$K = \frac{2lP_2\gamma_A}{P_1 - P_2} \times \frac{Q}{A} \tag{2}$$

where, K: Air Air Permeability Coefficient(mm/s), Q:The amount of the air which passed the specimen(mm^2/s), A :Area of specimen(mm^2), l :Thickness of specimen(mm), P_1 :Load Pressure(N/mm^2), P_2:Atmosphere pressure(N/mm^2), γ_A:Unit weight of air(1.182×10^{-5}N/mm^2)

<u>(3)Measurement of Water Diffusion Coefficient</u> [Tamura jiro 2002] The overview of the measurement instrument is illustrated in Figure 3. The size of the specimen is diameter 100 x 30 (mm). The water diffusion coefficient was computed by the formula (3) by measuring the water pressure, pressing time and the depth of water permeation.

$$\beta_i^2 = a \times \frac{D_m}{4t\xi^2} \tag{3}$$

where, β_i^2: Water Diffusion Coefficient(mm2/s), D_m:Mean depth of penetration(mm), t:Pressing time(s), a:Factor of pressing time(t $^{3/7}$), ξ:Factor of Pressure

2.3.3 Change in Distribution of Fine Pore Size and Capillary Absorption

The fine pore size distribution curve determined by the mercury intrusion porosimetry represents the distribution of the pore size which contains the most mercury permeation amount rather than that of the accurate fine porosity per pore size of a porous material. This can be explained by the ink bottle phenomenon shown in Figure 4. Given the irregular pore structure and the narrow ink bottle phenomenon (passage), the big pore is evaluated by the small pore's Kelvin-Laplace permeation pressure. As a result, small pores are overestimated but big pores are underestimated.

If there is no external pressure gradient, a liquid generally permeates a material via capillary pores by the capillary tension. Accordingly, in this test, we related the performance change of the patch repair materials resulting from the repetition of high/low temperature and dry/moisture condition to the change of the porosity within the range of capillary pore size in the fine pore size distribution curve obtained from the mercury intrusion porosimetry, and compared it with the result of capillary absorption test.

<u>(1) Measurement of Fine Pore Size Distribution by Mercury intrusion porosimetry</u> In this test, the distribution of fine pore size was measured by using a Mercury intrusion porosimetry.

<u>(2) Measuring an absorption factor of capillary tension</u> We soaked a specimen of 40x40x160(mm) in a water of about 0.5 mm, and measured the secular variation of the water absorbed by the capillary tension, and determined the water absorption coefficient by applying the formula (4).

$$M = a\sqrt{t} \tag{4}$$

where, M : Water absorption by capillary tension(g), a : Water absorption coefficient of the capillary tension, t :Time(s)

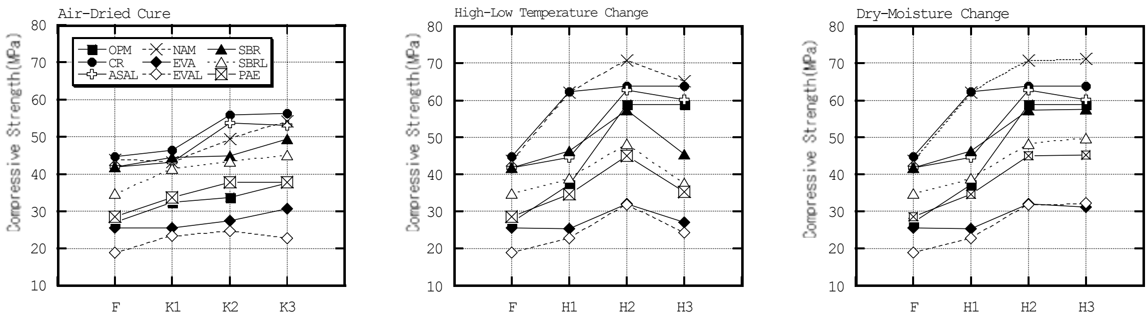


Figure 5. Change of Compressive Strength

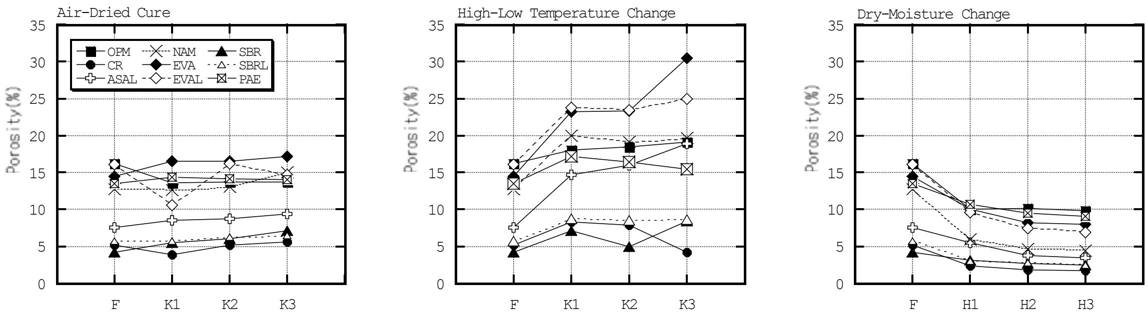


Figure 6. Change in Porosity

2.3.4 Deterioration of Surface Film

In order to evaluate the deterioration of the surface polymer film of the patch repair materials exposed to 100 cycles of low/high temperature change, we observed the surface character by an optic microscope (x250)

3 TEST RESULT AND CONSIDERATIONS

3.1 Change of Compressive Strength

Figure 5 shows the change in compressive strength of the patch repair materials adopted in this study under the condition of repeated low/high temperature and dry/moisture change. As a result of an initial measurement, the compressive strength of such patch repair materials as CR, ASAL, NAM and SBR was high. Within the range of this test, all specimens showed the increase in strength under the repetition of low/high temperature and dry/moisture situation. Inferring from the existing study[Masanobu ASHIDA *et al.* 1991] result that the compressive strength of the polymer cement mortar is little affected by the type of added polymer but by the content ratio of a polymer, we judge that the increase of compressive strength is caused by the hydration of a cement mortar, an inorganic material.

3.2 Change in Total Porosity and Mass Transport Resistance

Figure 6 shows the initial porosity value, and its secular variation; Figure 7, the initial value of an air permeability coefficient and its secular variation; and Figure 8, the initial value of the water diffusion coefficient and its secular variation under an accelerating environmental condition. There was little change in the porosity of all materials under an air dried curing conditions, the porosity increased under a low/high temperature repetition condition, but it decreased under a dry/moisture repetition condition. The increase in the total porosity due to the repetition of low/high temperature change seems to result from the increase in the micro cracks caused by the internal stress which results from the difference in the thermal expansion coefficient of the fiber, polymer, cement paste, and aggregates.

The decrease in the total porosity due to the repetition of dry/moisture situation seems to be the result of restart and progress of hydration by the moisture being supplied to the non-hydrated cement[Takeshi IYODA & Taketo UOMOTO 2003]. The air permeability coefficient and water diffusion coefficient in a mass

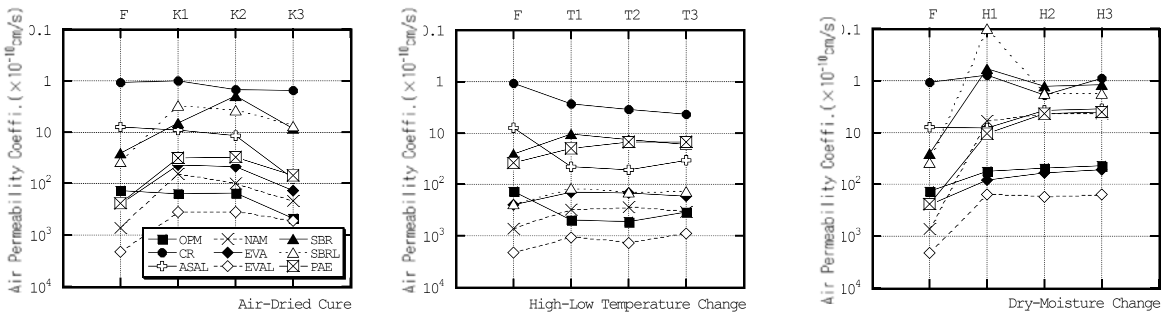


Figure 7. Change in Air Permeability Coefficient

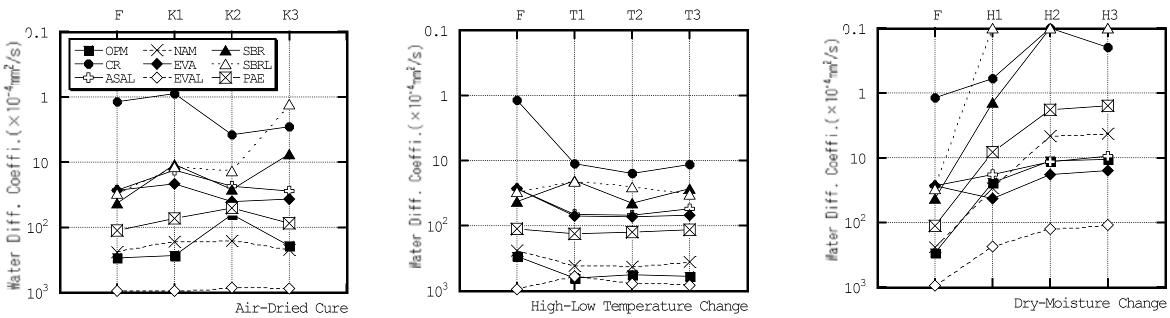


Figure 8. Change in Water Diffusion Coefficient

transport has a correlation with the total porosity. The mass transport resistance decreases as the porosity increases due to the repetition of low/high temperature situation, but the mass transport resistance increases as the porosity decreases due to the repetition of dry/moisture situation. Figure 9 and 10 shows the relationship between porosity and air permeability coefficient and porosity and water diffusion coefficient respectively, and each of the graph shows high correlation. Figure 10 shows that the value of water diffusion coefficient of SBRL is very low, and this is because a fine polymer film was formed on the surface and the water permeation was blocked from the cast surface. This issue will be discussed again in the section **3.4 Deterioration of Surface Film**, the damage of surface film reviewed via an optic microscope.

3.3 Change in Distribution of Fine Pore Size and Capillary Absorption

As discussed in the **2 TEST METHODS**, it is assumed that the fine pore size distribution curve determined by the mercury intrusion porosimetry, due to the ink bottle effect, shows not the correct ratio of each fine pore size but the permeability of the material including the connectivity of fine pores. The range of the fine pore size indicating the capillary pores differs depending on documents, and the data differs depending on the type and age of the measured materials[P. K. Mehta & P. J. M. Monteiro 1993]. As a result of this test, as shown in the Figure 11, for all the used patch repair materials, the peak of porosity which is seen as capillary pores appeared in the range of 0.01-5.26μm. The fine pore size and its height (porosity) that show the peak of capillary pores distribution differs depending on the environmental condition. Under the repeated low/high temperature change, the fine pore size indicating the peak of capillary pore became big, and the corresponding porosity tended to increase. Under a repeated dry/moisture change, the fine pore size indicating the peak of capillary pore became small, and the corresponding porosity tended to decrease.

In this study, we treated the change of the peak of the capillary pores as the change of capillary porosity, and compared it with the capillary absorption coefficient determined in the capillary absorption test and analyzed it. The change of the amount of the capillary pore was determined by the formula (5) which was defined to evaluate, after fixing the peak range of capillary pores as shown in Figure 11, its change via quantification.

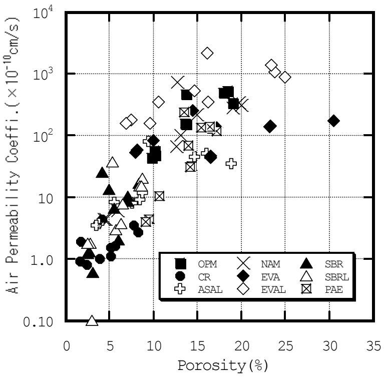


Figure 9. The Relationship between Porosity and Air Permeability Coefficient

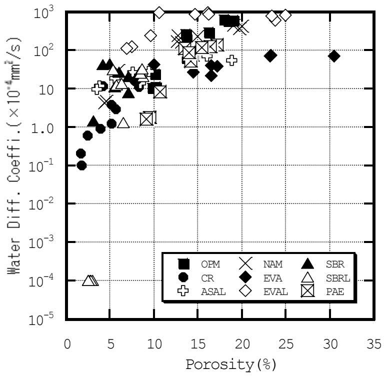


Figure 10. The Relationship between Porosity and Water Diffusion Coefficient

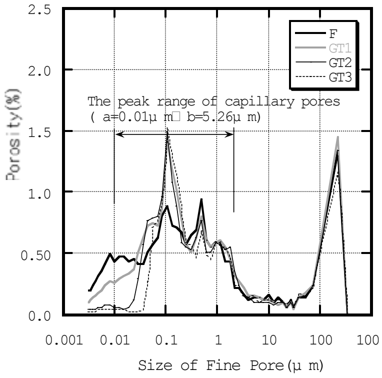


Figure 11. The range of Capillary Pores

$$P_{capillary} = \sum_{N=a}^{b} \left(N \times P_N\right) \qquad (5)$$

where, $P_{capillary}$:The amount of the capillary pore, N :Pore size between a and b, P_N :Porosity of pore size N, a,b :The peak range of capillary pores(a=0.01, b=5.26μm)

Capillary absorption coefficient was determined via regression of the result by using a least squares method. Though the Figure 12 shows the change in the capillary porosity and capillary absorption coefficient of the patch repair materials being subject to the repeated low/high temperature and dry/moisture change, but the change shows almost the same trend as discussed in **3.2 Change in Total Porosity and Mass Transport Resistance**. In other words, the capillary absorption coefficient increases due to the increase of the capillary porosity resulting from the deterioration of the internal matrix under the environment where the low/high temperature change is repeated. On the other hand, if the material is exposed to a dry/moisture repetition, a tendency being contrary to the result of a low/high temperature repetition was observed. That is, as the number of cycles increases, the capillary porosity and the capillary absorption coefficient decreased and the internal matrix structure became more compact. As a result of comparing the capillary porosity and capillary absorption coefficient, we

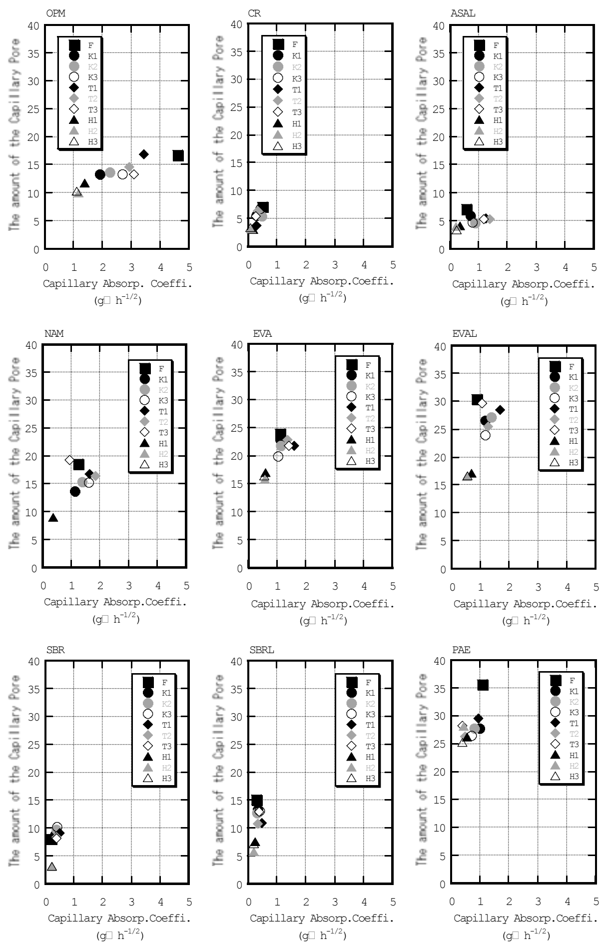
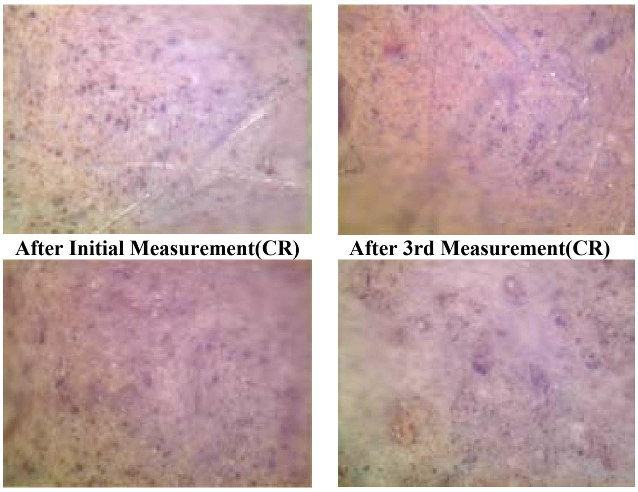


Figure 12. The Relationship between change in the capillary porosity and capillary absorption coefficient

could find some degree of correlation but observed data dispersion as well. This may result from the facts – the patch repair materials used in this test contain not only the main ingredient polymer but also many admixtures; the internal matrix formation and deterioration mechanism of the polymer other admixtures are different from each other; and the pores out of the assumed range of capillary pore size had some influence on the capillary absorption.



After Initial Measurement(CR) **After 3rd Measurement(CR)**

After Initial Measurement(SBR) **After 3rd Measurement(SBR)**

Figure 13. Deterioration of Surface Film(x250)

3.4 Deterioration of Surface Film

As a result of observing the deterioration of the surface film of the polymer-contained patch repair material after exposing it to 100 cycles of the low/high temperature change, we could fine a whitening phenomenon that seemed to be resulted from the cutting due to the deterioration of the chains of polymer molecular structure in CR and SBR. The Figure 13 shows the picture of the surface of the patch repair material which was photographed by a microscope. No special phenomenon was found in other materials.

4 CONCLUSION

1) The compressive strength of a patch repair material is little affected by the type and content of a polymer, but by the progress of the hydration of a cement mortar, an inorganic material.
2) If the patch repair material is exposed to the repeated low/high temperature changes, the porosity increased due to the fine cracks resulting from the difference in the thermal expansion coefficient of components, and this resulted in the increase in the mass transport resistance.
3) When the age of the patch repair material is not old, and if the material is exposed to the repeated dry/moisture changes, water is supplied to the repair material slowly as the wetting period increases during the accelerating deterioration period, and the cement is hydrated. This results in decrease in the porosity and decrease in the mass transport resistance.

5 REFERENCES

C. Galle , 2001, 'Effect of drying on cement-based materials pore structure as identified by mercury intrusion porosimetry, A comparative study between oven-, vacuum, and freeze-drying', *Cement and Concrete Research,* **31**, pp.1467-1477

S. Tsivilis, J. Tsantilas, G. Kakali et al., 2003,'The Permeability of Portland limestone cement concrete', *Cement and Concrete Research*, **33**, pp.1465-1471

Tamura jiro, 2002, Water tinghtness design of Concrete, Gihodo Press, Japan, pp.18-19

M.Ahida, M. Matsumoto & E. Sakai, 1991, 'Fundamental Properties of Polymer-Modified Cement Mortar with Ultra Fine Particles' *CAJ Proceedings of Cem. & Con.,* pp.726-731

T. Iyoda & T.Uomoto, 2003, 'Effect of Drying Condition at Early Age on Microstructures of Hardened Cement', *Journal of Materials, Concrete Structures, and Pavements*, **732**, pp.17-26

P. K. Mehta & P. J. M. Monteiro, 1993, *'CONCRETE: Microstructure, Properties and Materials, Second Edition'*, The McGraw-Hill Companies Inc, pp.26-27

Durability increase of concrete exposed to aggressive chemical media

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ABSTRACT

The risk and the frequency of concrete utilization exposed to aggressive media of XA1 till XA3 categories following the Standard way EN 206 are increase again and again at present time. The demands concerning the composition of concrete applied in this way are defined in the Standard only by the minimum dose of cement, the highest water/cement ratio or by the minimum class of concrete. These solutions are in many cases insufficient and they are not at all economical or ecological solutions.

The concrete resistance increase against the effect of carbonate, acidic or sulphate corrosion is possible by the addition of ferrous dust, which in a fresh suspension doesn't sediment, on the contrary it seals up the porous structure and in this way it improves the resistance even against the sulphate corrosion.

The ferric dust can be characterized as industrial waste formed during the manufacture of pig iron in blast furnaces. These waste materials are classified as dangerous wastes. To achieve the necessary level of anticorrosive protection the following properties of ferrous dusts are of importance: grain fines, content of effective component (purity), the dose, sort and perfect dispersion in the cement matrix.

The aim of experimental work described in this paper was, to built this powder into concrete and in this way to achieve a secure deposition of it under parallel improvement of concrete properties. We checked the effect of different ferrous dust doses on physico-mechanical properties of concrete and mainly on the durability of concrete, stored for a long time in chemically aggressive liquid media (high concentration of sulphate and chloride ions and in acidic medium).

KEYWORDS

aggressive chemical media , durability of concrete, ferrous dust

1 INTRODUCTION

The current state of designed concretes exposed to aggressive chemical media is strongly affected by introduction of EN 206-1 and dividing aggressive chemical media into classes from XA1 through XA3. Decisive for such classification are the concentration limit values of typical aggressive chemical media for solid and liquid state. According to such concentrations (problems of liquid media – contaminated ground water is most often faced in practice), concrete composition is designed. In the Czech Republic, data contained in Annex F1 are, unfortunately, taken as binding requirements for designing of chemically resistant and durable concretes in overwhelming majority of cases, though the Annex contains only recommended limit values for concrete composition and properties, however the data stated therein, especially the minimum dose of concrete and maximum water-cement ratio and minimum concrete class, are taken as binding. Designing of concretes resistant to aggressive chemical media complies, according to this scheme, with the requirements of EN 206 and everything is OK in the terms of administration, but their functionality, correctness, suitability and economics arise doubts. It stands to reason that it is very simple to design such type of concretes according to the same key, including flat-rate use of one type of concrete for different aggressive chemical media but this is, basically, alibi like fulfilment of requirements set forth in the Annex F1.

This research examined a somewhat different approach to designing of durable concretes regardless the points of view of the aforementioned EN, and namely utilising special active admixtures. Tested was the impact of very fine ferrous dusts generated in the course of crude iron manufacture in blast furnaces on increasing of resistance of concretes exposed to some types of aggressive chemical media. With regard to the fact that the objective and truthful testing of durability of exploited concretes requires a long period of time when partial conclusions can be said after minimum 1 year of placing the concretes in solutions of strong concentration and final conclusions can be commented after several years, the results stated herein are just partial and the research is going on.

2. FERROUS DUSTS

Ferrous dust is an industrial waste (of hazardous category due to a high content of heavy metals and zinc) generated during the manufacture of crude iron in blast furnaces. It is a dry, powdery material with ball-shaped grains up to the maximum grain size of cc 0.1 mm which is trapped in electrostatic separators. Specific surface area is about 6,000 m^2/kg. These flying dusts can be added to the concrete to reach special properties of concretes made of Portland cement.

2.1. Properties of ferrous dusts

Ferrous dusts tested within the framework of this project had a character of a dry, very fine flying dust of dark brown colour. Characteristic chemical composition of ferrous dusts is given in the following Table 1.

Table 1. Chemical composition of ferrous flying dusts

Fe	51	% of dry mass
Mn	5.29	% of dry mass
Zn	4.96	% of dry mass
Pb	6608	mg/kg
Cr	2590	mg/kg
Cu	1550	mg/kg
Ni	297	mg/kg
V	145	mg/kg
Sb	77	mg/kg
As	60	mg/kg
Cd	42	mg/kg

Co	17	mg/kg
Hg	4.65	mg/kg
Se	2.2	mg/kg
Tl	2.0	mg/kg
Be	0.5	mg/kg

Loss on ignition does not exceed 5 % by weight; chlorides content recalculated to Cl^- is not higher than 0.1 % by weight. Content of sulphur oxide SO_3 ranges up to 3 %. Concentration of limited harmful substances in dry matter of flying dusts is given in Table 2.

Table 2. Concentration of limited harmful substances in dry matter of ferrous dusts

Indicator		*Ferrous flying dust*
As	mg/kg	60
Cd	mg/kg	42
Co	mg/kg	17
Cr_{total}	mg/kg	2590
Cu	mg/kg	1550
Fe	% of dry mass	51
Hg	mg/kg	4,65
Mn	% of dry mass	5,29
Ni	mg/kg	297
Pb	mg/kg	6680
Zn	mg/kg	4,96% mass.

To identify mineralogical composition of ferrous dusts, differential thermal analysis (DTA) and X-ray diffraction analysis (RDA) were carried out. Records from such assays are given in the following Figures No. 1 and 2. It follows from the results of DTA and RDA assays that ferrous flying dusts contain a great amount of amorphous Fe_3O_4. Flying dusts do not contain any organic substances, e.g. oil products or binders.

Figure 1. RDA

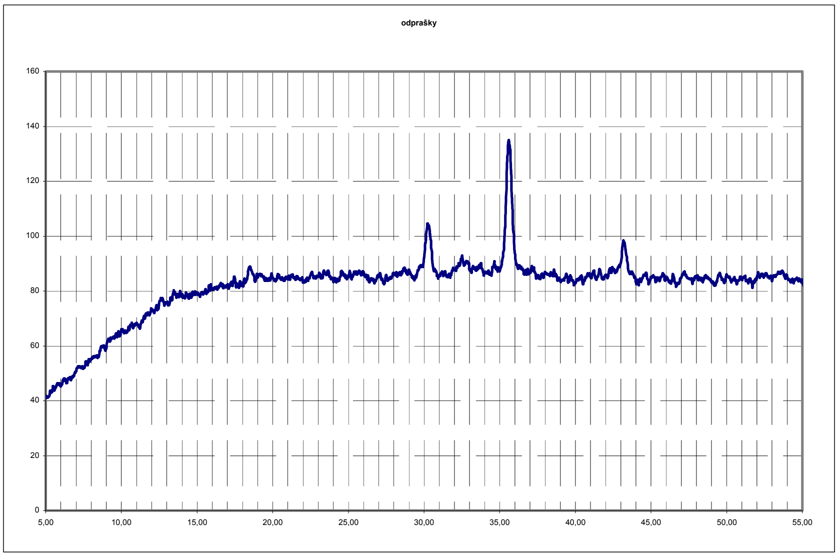
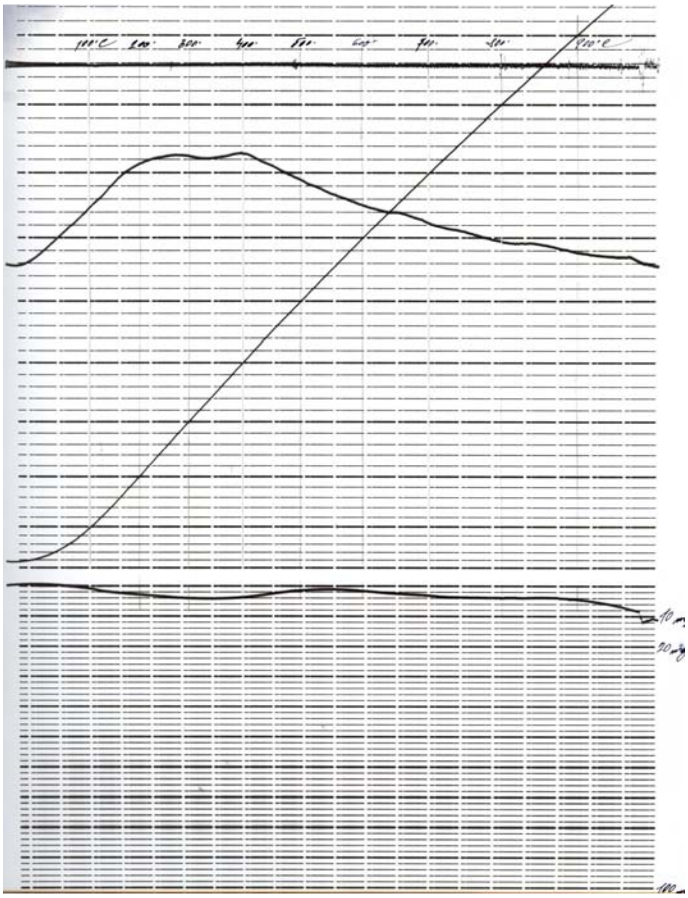


Figure 2. DTA



2.2 Action of ferrous flying dusts in concretes

Ferrous dust can act in concrete on more levels. The first possibility is to use flying dusts as an addition to fine fractions, e.g. to improve workability of fresh concretes, their pumpability or to use them as a correction admixture in self-compacting concrete. Regarding the particle fineness of ferrous flying dusts and their high specific surface area, positive can also be their contribution to reducing the risk of bleeding. In hardened concretes, fine fractions may have a positive impact on porous structure and, thus, e.g. watertightness of concretes or their durability when exposed to frost, chemical de-icing salts or aggressive chemical media. In fact, flying dusts can carry out 2 functions of admixtures in concrete simultaneously, both as an admixture of I type– inert aggregate and as an admixture of type II – active function. Thanks to their chemical composition, they can positively contribute to increasing of long-term strengths of concretes and, mainly, to improving resistance to the action of aggressive chemical media (degrees XA2 and XA3 according to EN 206-1). Protective action of ferrous dusts consists in the process of corrosion of ferrous admixture as an effect of aggressive media in the presence of oxygen. Products of ferrous corrosion – oxides and iron hydroxides as well as their hydrates – have a considerably larger volume when compared with the volume they were generated from. Ferrous powder oxides as a result of reaction of iron with acid aggressive media generating voluminous gel oxides and iron hydroxides or also calciumferrithydrates, and pores and capillaries of hardened cement paste begin to be filled by generated ferrous corrosion products which reduce the penetration of aggressive acid water inside the hardened cement paste.

3 TESTING OF ACTION OF FERROUS DUSTS ON CONCRETE WORKABILITY AND DEVELOPMENT OF COMPRESSION STRENGTH

Measurement of viscosity carried out with cement pastes modified by ferrous dusts confirmed strong retarding effects of ferrous flying dusts. Retarding of hardening is caused, probably, by a relatively high content of zinc. This effect makes use of flying dusts in routine practice considerably complicated. This effect had to be eliminated. Therefore, action of various types of superplasticizers with accelerating effects and accelerating agents were tested. After lots of experimental testing, this problem was solved by an accelerating agent based on aluminium sulphate.

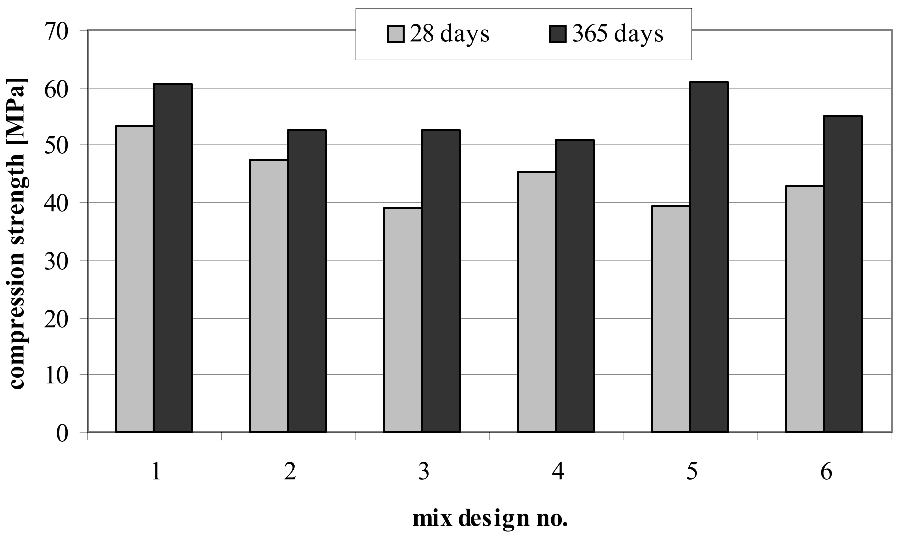
Simultaneous measurement of rheology showed an interesting effect of action of ferrous dusts on workability when an admixture of flying dusts has slightly plasticizing effects. It turns out that increasing of contents of fine fractions by adding ferrous flying dusts does not call for increased requirements for mixing water to maintain the required consistency.

Impact of content of ferrous dusts on long-term strength of concretes was tested with hardened concretes. The tests were conducted with test cubes of standard dimensions 150 x 150 x 150 mm. Example of tested mix design is given in the following Table 3. The composition is expressed in kg per 1 m^3 of concrete. Plasticizers manufactured by SIKA were used to improve workability and the Czech admixture Prestiž on the basis of aluminium sulphate was added as an accelerator. Dose of mixing water was adjusted so as the workability of all mixtures was always in degree S 3 according to EN 206.

Table 3. Composition of concrete mixtures in kg per m^3 of concrete

Designation of formulae	*1.*	*2.*	*3.*	*4.*	*5.*	*6.*
Cement CEM I 42.5 R	357	294	294	294	294	357
Ferrous dusts	63	126	63	63	63	0
Ferrous dusts % from CEM I weight	18	43	21	21	21	0
Blast furnace slag	0	0	63	63	63	63
Sand 0- 4 mm	728	728	728	728	728	728
Broken aggregate 8 -16 mm	1156	1156	1156	1156	1156	1156
Plasticizer SIKA 10 HRB	0	0	0	5.46	0	0
Plasticizer SIKA MULTIMIX 100	0	0	0	0	4.62	4.62
Accelerator PRESTIX						
Water	218	214	228	202	220	204
Consistency /mm/						
Compression strength after 1 day /MPa/						
Compression strength after 7 days /MPa/						
Compression strength after 28 days /MPa/	47.3	39.0	45.3	39.5	42.7	53.2
Compression strength after 365 days /MPa/	52.5	52.5	51.0	61.0	55.0	60.5
Increase in %	11.1	34.6	12.6	54.4	28.8	13.8

Figure 3. Comparison of development of compression strength



4. TESTING OF DURABILITY OF CONCRETES EXPOSED TO LIQUID AGGRESSIVE CHEMICAL MEDIA

Resistance of concretes to action of carbonaceous, acid or sulphate corrosion can be increased by using the admixture of ferrous dust which does not sediment in fresh suspension; on the contrary, it fills the porous structure improving thus resistance also to sulphurous corrosion. To reach the required extent of anticorrosion protection, the following properties of ferrous dusts are important: fineness, content of active ingredient (cleanliness), dose, kind and perfect dispersion in cement matrix.

4.1. Experimental part

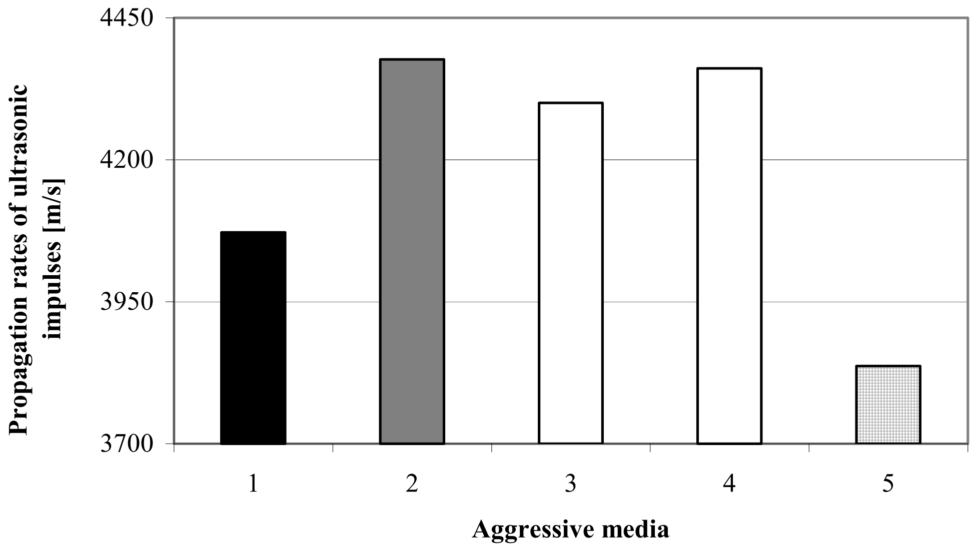
The following chapter presents the results of experimental works carried out with concretes containing ferrous dusts. Example of four formulae with different content of ferrous dusts was picked out from a whole package of performed experimental works and they were compared with the reference formulae. The designed formulae are the same as the ones in Chapter 3 and they are listed in Table 3.

After extensive experimental laboratory tests, pilot tests were performed in high-capacity concrete batching plant. The manufactured concretes were cast into timber formwork of cc 1 m^3 volume placed outdoors. After 28 days of maturing, samples were taken from cast blocks in the form of test cores of 150 mm diameter for further laboratory tests. The samples taken were thoroughly visually inspected, weighed and then each sample was tested for concrete homogeneity. This was detected non-destructively by means of ultrasonic impulse method. After the test, the samples were placed in prepared solutions of different aggressive chemical media:

- solution of Na_2SO_4 with concentration of 34 600 mg/l
- solution of Na_2SO_4 with concentration of 8 000 mg/l
- solution of $Mg(NO_3)_2 * 6\,H_2O$ with concentration of 5 000 mg/l
- solution of 10 % NaCl .

Concentration of solutions exceeded highly the values routinely reached in practice and all are in category XA3 according to EN 206-1. Samples were placed in vessels which were hermetically sealed. Visual inspection of stored samples was performed and concentration of solutions was checked once per month. After one year, the samples were drawn out, dried out, thoroughly visually inspected and weighed and again each sample was tested for concrete homogeneity which was detected by ultrasonic impulse method.
Example of ultrasound propagation rates is plotted in the following Fig. 4.

Figure 4. Comparison of propagation rates of ultrasonic impulses depending on the type of aggressive medium – mix design no. 2.



1. SO_4^{-2} : 34600 mg/l
2. SO_4^{-2} : 8000 mg/l
3. Mg $(NO_3)*6H_2O$: 5000 mg/l
4. 10 % NaCl
5. Non-aggressive medium - water

5. CONCLUSION

Based on the reached results, it may be stated that the admixture of ferrous flying dusts has an advantageous impact on workability of cement suspensions. Flying dusts have a liquefying effect on fresh cement mixtures at maintaining a constant water-cement ratio. After certain period of time, viscosity does not change with time which is caused by the impact of strong retardation of flying dusts.

Compression strengths have a very interesting development. It was confirmed that a more distinctive effect comes with co-effect of active plasticizer whereby the concrete needs not to be liquefied by this way any more. Dose of mixing water needed to reach the required consistency is basically the same. This indicates the plasticizing effect of ferrous dusts which means that there is no need to increase the dose of mixing water to reach the required consistency.

Use of ferrous dusts has a great impact on the result compression strengths at the age of more than 28 days. Increases of concrete strengths have a long-term character and they continue with a very positive trend even when older than 1 year. When suitable accelerators are used, the same strengths like with the concretes without flying dusts at maintaining the same consistency can be reached, as a minimum, after 24 hours. Compression strength of one-year old concretes with flying dusts reached as much as 55 MPa with the dose of only 294 kg/m^3 of cement CEM I 42.5 in combination with finely-ground blast furnace slag in the amount of 20 % from the concrete weight. Strength at the age from 28 to 365 days is increased by as much as 54.4 %. Use of ferrous dusts appears in this respect to be very perspective and it makes possible not only their safe disposal but it may also have positive economic and environmental impacts.

Detected compression strengths are in a very close correlation with volume density of concrete. Empirical formula between compression strength and volume density was elaborated for illustration – its correlation coefficient r = 0.93. From the point of view of practical application, the formulas with the value of correlation coefficient $r \geq 0.85$ are suitable.

Based on the provided experiment works, e.g. achieved water extracts, we can say that ferrous dust had no effect on enviroment.

After one-year placement in solutions of strong concentrations, the samples with ferrous dust admixture do not exhibit any visual defects. Comparing the sample weights before and after the placement, it was detected that in most of the cases the weight had increased. It may be deduced therefrom that ions transferred from aggressive solutions to the concrete structures.

In the same time, it was detected using the ultrasonic impulse method to test the samples for homogeneity that the samples with the admixture of ferrous dust did not change their homogeneity even after being placed in strong-concentrated solutions for one year, i.e. they were not damaged with micro-cracks caused by sulphate or magnesium expansion. On the contrary, propagation rate of US impulses increased in most cases which is proved by structure densification and improved homogeneity and uniformity of concretes.

Increasing of resistance of hardened cement paste to acid and basic aggressive media appears to be a distinct effect. Protective action of ferrous flying dusts consists in the fact that corrosion process of admixture starts due to aggressive media in the presence of oxygen. Products of ferrous corrosion – oxides and iron hydroxides as well as their hydrates – have a considerably larger volume than their original one. Ferrous powder oxides as a result of reaction of iron with acid aggressive medium generating voluminous gel oxides and iron hydroxides or also calciumferrithydrates, which are chemically very resistant, especially to carbonaceous or acid solutions up to pH4 and pores and capillaries of hardened cement paste begin to be filled by generated ferrous corrosion products which reduces the penetration of aggressive water into the hardened cement paste. Ferrous dust incorporates into a small surface layer of hardened cement paste in the so called free corrosion zone until gel microstructure C-S-H and C-A-H are created as hydrated Fe-phase. This considerably increases the prerequisites for the increase of chemical resistance. Gel pores, by contrast to greater capillary pores, are not filled with aggressive water during chemical exposure.

4 ACKNOWLEDGMENTS

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5 REFERENCES

Hela, R., Maršálová, J., Kučerová, H., Fojtík, R. 2003, 'VVZ MSM 261100008, DT5/2: Využití železitých odprašků v betonech', Brno.

Hela, R., Maršálová, J. 2004, 'Zlepšování kvality betonů za pomocí železitých odprašků', in *3. konference Technologie, provádění a kontrola betonových konstrukcí*, Praha.

Efflorescence on clay bricks masonry: towards a new test method

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ABSTRACT

Efflorescence on clay bricks masonry used for veneer wall makes up a real problem appearing frequently just after the building implementation. Even if it affects only the aesthetic of the building, their baneful consequences are however indisputable both on the economical point and the quality concept in the building art. Except some intern laboratory procedures, there aren't any normalized test methods at the European level that take at the same time the clay brick and its mortar into account in the displaying of the efflorescence phenomenon. Now, most of the efflorescence's studies show that the interaction between these two elements is responsible for the efflorescence apparition. In Belgium, the national norm tests only the brick sensibility without evaluated the exchanges with the hydraulic binders material. In this article, an efflorescence test responding to these exigencies is presented. This test is the result of a research based on the understanding of the efflorescent salts apparition mechanism and the reproduction of this mechanism on laboratory samples. This reproduction has been done thanks to the precise determination of each parameter defining the different phases of this salts apparition mechanism: implementation, curing, humidification and drying. Collated to a great number of real cases, the test is now under validation.

The research, of which the goal is to present an efflorescence test in a new evaluation method of the brick/mortar assembly eventually normalizable at the European level in order to complete test method on masonry norm, proceeds from collaboration between the Belgian Building Research Institute and the Belgian Ceramic Research Centre.

KEYWORDS

Efflorescence, salt, masonry, brick, mortar

1 INTRODUCTION

Efflorescence appears generally on the surface of veneer wall build with porous material as clay bricks under an aspect of deposit of white powdery or foggy salts. The baneful consequence of this salt apparition is considered mainly as an aesthetic problem. This phenomenon often arrives just after the end of the construction (see 'Fig. 1') always generating confusion in the attribution of responsibilities between the different parties involved: brick provider, mortar provider, contractor, designer and customer. Sometimes, efflorescence appears few years after the end of construction leaving the building owner face to questions about cleaning or maintenance. In some cases, this salt apparition can be very persistent and hard to remove [Brocken *et al.* 2004].

Figure 1. Example of efflorescence apparition at the end of work.

Unfortunately, up to date, there are no recognized procedures at national or European level allowing to prevent the risk linked to this nuisance and protecting the building actors from litigations. Therefore, the Belgian Building Research Institute and Belgian Ceramic Research Centre have entered upon a study to define a new test procedure in order to characterize the efflorescence sensibility of a brick/mortar assembly. Because of the phenomenon is linked to the interaction between the brick and the mortar, it was indeed really important to study the assembly and not the two elements separately. This research is then supported by the two industries (brick and mortar providers).
The research methodology has been at first to understand the efflorescence apparition mechanism in order to find its sensibility parameters, then to carry out laboratory experiments in order to settle these parameters to their critical value and finally to obtain a reliable test procedure.

2 EFFLORESCENCES APPARITION MECHANISM

Efflorescences are the demonstration of soluble salts cristallisation at the surface of porous materials, due to the migration of solutions induced by evaporation. The most encountered efflorescences on masonry are due to the presence of soluble sulfates of Na, K, Mg and Ca [Barquin *et al.* 1996]. They are resulting from the interaction of bricks with hydraulic binders assemblies. In clays that are used in the manufacture of bricks, we usually find sulphides and especially pyrite. These sulphides become oxydated during firing to form SO_3 species that can react with the basic oxides of the clay. They form sulphates such as $CaSO_4$, Na_2SO_4, K_2SO_4 and $MgSO_4$. The last three sulphates are dissociated when the firing temperature exceeds 950°C but the calcium sulphate needs a temperature of at least 1200°C to be dissociated. However, the firing temperature of bricks varies from 950 and 1200°C so only $CaSO_4$ can still be found sometimes in bricks, but in very low amounts. In cements, oxides such as Na_2O and K_2O form hydroxides (NaOH and KOH) after the mixing. As cements usually contain gypsum ($CaSO_4.H_2O$) added to control setting, it can react with the hydroxides to form Na and K sulphates by the following reaction:

$$CaSO_4 + 2\ NaOH = Na_2SO_4 + Ca(OH)_2$$

Usually gypsum is bound in ettringite during hydration. But, if some $CaSO_4$ is present in the brick after firing, it can also combine with hydroxides and form alkali sulphates that will precipitate as efflorescence. The efflorescence formation is thus a complex phenomenon that also depends on external physicochemical parameters such as the implementation type, the curing duration, the humidification duration and the drying way. For the implementation, the nature of the components, the geometry of the assembly, the moisture of the brick and the mortar consistency are important to control. The duration of the curing is important to take into account as it shows the influence of a first rainfall on masonry after implementation and allows the migration of the available alkali in the assembly. The duration of the humidification is also paramount to allow a good homogenisation of alkali into the assembly, from the mortar into the bricks. Finally, the drying conditions have to be as close as possible to the real meteorological conditions encountered during the period when efflorescence appears on masonry. All these parameters have been investigated and adjusted to define an efflorescence test on assemblies in order to obtain results close to those observed *in situ*.

3 EFFLORESCENCE TEST SET UP

3.1 Materials choice

In order to set up the efflorescence test, we have chosen relevant building materials in accordance with the sector. These materials are considered to be representative of the veneer wall market offer.

Four bricks have been selected with respect to their fabrication process (handmade and extruded bricks) and sulphate concentration (high and low). In Table 1, we give the bricks characteristics and their identification.

Brick	*Low [SO_4^{--}] (<0.15 % wt)*	*High[SO_4^{--}] (>0.5 % wt)*
Extruded	V	Q (0.66 % wt)
Handmade	B	S (1.95 % wt)

Table 1. Overview of the selected bricks characteristics.

When tested according to the norm NBN B 24-209, only the brick V shows efflorescence sensibility. We have chosen to build the assembly with dry bricks for the test result reproducibility.

The composition of the different mortar types has been chosen in consultation with the FeMO (The Belgian Mortar Federation) to cover the building mortar possibilities and is given in Table 2.

Mortar	*Composition*
M11	CEM I 42.5 T HES + river sand
M12	CEM II 32.5 N + river sand
M13	CEM III 32.5 N LA + river sand
M14	Dry ready for use mortar

Table 2. Overview of the selected mortars characteristics.

The mortar is prepared with ‘main water’ and the water quantity has been determined by the norm EN 1015-3 in order to obtain a consistence of fresh mortar of about 2.1 (with the flow table). This consistence allows to work with dry bricks. With this brick and mortar choice, sixteen different combinations will be used for the laboratory experiments.

3.2 Optimisation of the test parameters

3.2.1 Implementation

The test has to be carried out on an assembly brick/mortar characterized by a shape which is representative for a masonry wall. Therefore we have designed an assembly composed of three bricks and three mortar layers and that respects the brick/mortar proportion of a normal wall. The mortar mix is prepared according to EN 1015-2 and the mounting is done in laboratory conditions. At the end of it, a mass is placed on its top in order to increase the adherence between the brick and the mortar during all the curing phase. This assembly is represented at the ‘Fig. 8’.

3.2.2 Curing

The curing duration is an important parameter to take into account as it shows the influence of a first rainfall on masonry after implementation. To determine the time needed to the alkali to be available within the mortar for migration in the assembly, we have tested several curing durations (between 4 and 28 days) on mortar bars first. The concentrations in Na and K have been measured after the various curing durations by ICP after leaching in water. It has been observed that the evolution of the mortar structure during curing has no influence on the availability of Na and K and that the concentrations are similar after only 4 days of curing.

We have then tested different curing durations on small assemblies. One brick has been cut into three parts, one was kept as reference and the two others were implemented into a small assembly represented in 'Fig. 2'.

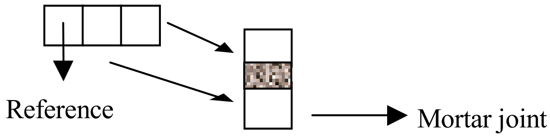

Figure 2. Small assembly used to determine the humidification parameter.

After a certain time, the assembly is put into a drying oven at 105°C to stop the migration of Na and K. The small assemblies are then cut into slices as shown on 'Fig. 3'. The reference piece is also cut into 3 slices. Each piece is ground, leached into water and Na and K are analyzed by ICP.

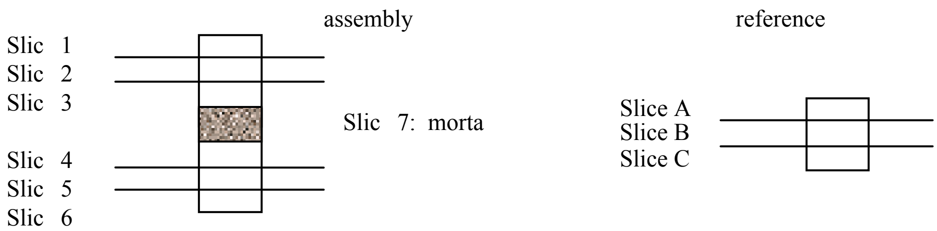

Figure 3. Slicing of the assembly and the reference before leaching and analysis.

It has been enhanced that after 14 days, the availability of Na and K is the highest one as shown on 'Fig. 4'. Moreover, at that time, the strength of the assembly is sufficient for further handling. The duration of the curing has then been chosen to be 14 days.

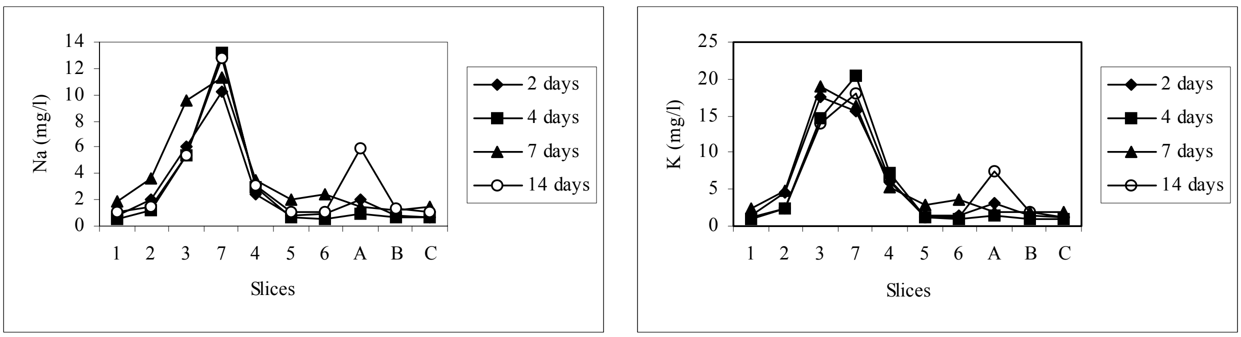

Figure 4. Concentrations of Na and K in the assemblies for various curing durations. Slices A to C correspond to the reference brick pieces.

3.2.3 Humidification

The humidification aims to obtain a good homogenisation of Na and K from the mortar into the assembly. It is then necessary to find the most appropriate duration of humidification. We have conducted similar tests as for the determination of the curing duration except that the small assemblies have been put into a tray within a constant 3 mm water layer after curing from 2 to 28 days. The rest of the experiment is similar.
The results show that after humidification, Na and K are leached from the mortar and decrease from one order of magnitude already after 7 days. The concentrations homogenize into the assembly and are higher into the brick slices of the assembly than into the reference brick slice. We also see that the

longest the duration of humidification is, the lowest the Na and K concentrations in the mortar layer are (slice 7) as shown on 'Fig. 5'.

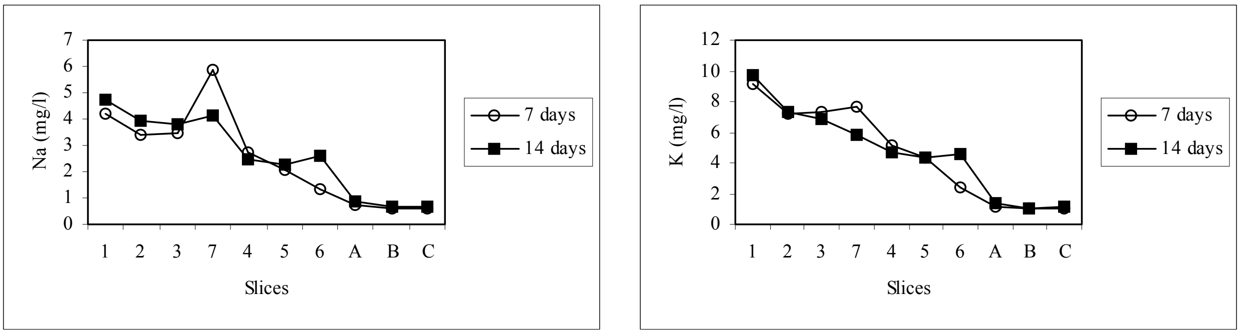

Figure 5. Concentrations in alkali within small assemblies after 7 and 14 days of humidification. Slices A to C correspond to the reference brick pieces.

So, in conclusions, the humidification time has been chosen to be 14 days to obtain a good homogenisation of the alkali into the assembly.

3.2.4 Drying

The optimisation of the drying conditions is really important. A too hard drying could carry away the crystallisation inside the brick and not on the surface. A too light drying could not cause crystallisation. To find out the optimal drying conditions, we have used an exposure site (shown in 'Fig. 6') with among other the selected brick/mortar combinations except the combination with the mortar M14. For each of them, two walls, one in the SW and the other in the NE direction, are built. The walls are 65 cm wide and 1m high. The three last layers of the wall are not assembled with mortar but by a rubber joint to see the behaviour of the brick alone.

Figure 6. Exposure site.

Near this exposure site, a meteorological station records every ten minutes the climatic conditions: temperature [°C], relative humidity [%], wind direction, wind speed [m/sec] and sun radiation [W/m²K]. Every day, a visual survey allows to see which wall shows efflorescence. This information is completed by three webcams that record every hour the evolution of three walls assumed to be really sensitive to the efflorescence apparition. Finally, some walls are fitted out with thermocouples placed inside the masonry in order to evaluate the thermal gradient. Three thermocouples by wall are used: one on the surface, one in the centre and one near the back of the wall.

All this information allows to assess the favourable drying conditions to the efflorescence apparition. These conditions are closely linked to the couple 'temperature/relative humidity' of the air that will determine what we call further the 'Air Drying Power'. The ADP [g/kg] is basically the water vapour quantity that the air can still absorb before reaching the dew-point and it is calculated by the difference between the maximal water content 'Xs' [g/kg] (depending on the air temperature) and the water content 'x' [g/kg] (depending on the temperature and the relative humidity of the air). Thanks to the exposure site monitoring, it is possible to determine the ADP and to couple it with the visual survey to find the ADP critical value above which the efflorescence always appears.

'Figure 7' shows an example of the coupled visual and ADP data. The survey is here reported for a period of three days in September 2002. The 3rd and the 5th September, the ADP was more than 9 g/kg and the pictures, taken at the middle of the day, show that there is efflorescence on the wall. On the other hand, the 4th September the ADP was less than 4 g/kg and the picture for the same wall doesn't show efflorescence.

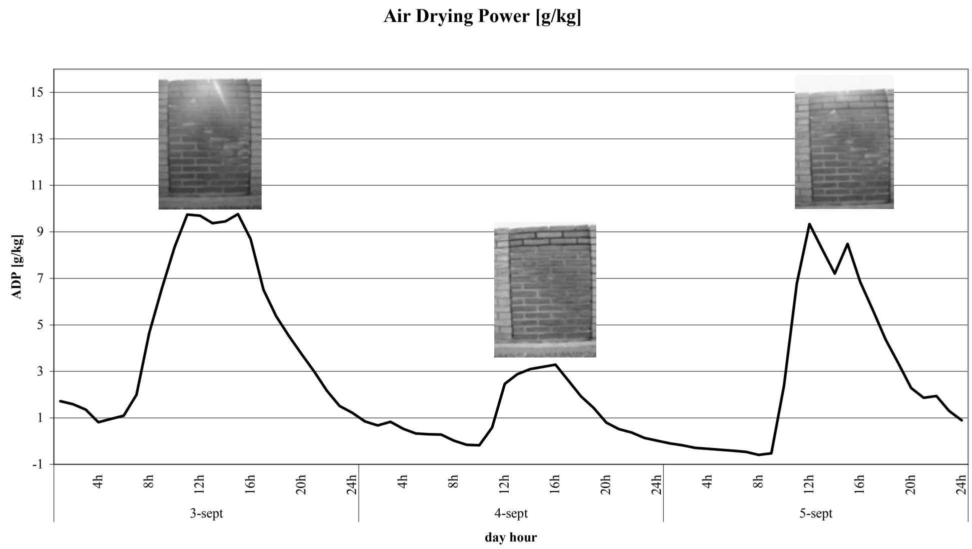

Figure 7. ADP graphic coupled with visual data of one wall of the exposure site.

With two years of recorded data, we have now the knowledge of the ADP critical value above which the efflorescence apparition is almost certain. This ADP value is around 10 g/kg of air. With this ADP value, it's possible to define 'temperature/relative humidity' conditions easy to realize in a laboratory by using the humid air equation [Carpentier *et al.* 1982]. Some examples are given in table 3.

Couple possibilities	*Temperature [°C]*	*Relative humidity [%]*
1	25	50
2	20	32

Table 3. Temperature and relative humidity carrying away an ADP of 10 g/kg.

Influence of temperature gradient. It is important to note that in case of important differences between the ambient air temperature and the wall surface temperature, the ADP critical value must be considered in the air layer close to the surface of the wall. For example, we can have general climatic conditions with a weak ADP of about 4 g/kg (10°C, 50% HR for instance) but sun radiations that lead to an ADP of about 10 g/kg on the masonry surface. These conditions are really favourable to the efflorescence apparition because of a thermal gradient builds up between the centre and the surface of the masonry. These conditions are often present during the spring when efflorescence appears on the masonry. Further investigations are currently realised in order to reproduce in the drying phase this temperature gradient by switching on IR lamps three hours a day. The IR lamps are situated fifty cm upper the surface of the assembly. The room hydrothermal conditions are 10°C, 50 % RH and during the IR lamps radiations, the surface of the assembly reaches 30°C. Up to date, the results seem to be very attractive by showing an increase of sensibility for the tested combinations.

3.3 Efflorescence test protocol

The test protocol is directly in accordance with the results of the optimisation phase experiments. Figure 8 shows the different test steps and the Table 4 gives the parameters of each test phases.

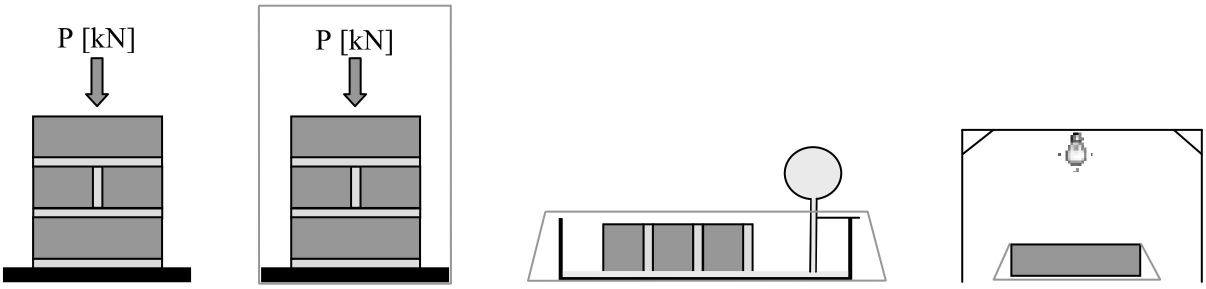

Figure 8. Pictures of protocol phases.

Protocol phases	*Duration [day]*	*Conservation conditions*
Curing	14	under plastic
Humidification	14	under plastic
Drying	until constant massa	25°C; 50% RH

Table 4. Test phases parameters.

At the end of the test, we proceed at the results evaluation following this scheme:
Take a picture of all assemblies;
Select the assemblies presenting efflorescence upon more than 5% of their surface;
Rub efflorescence with a humid sponge three times;
Wait fifteen minutes;
Note separately the assemblies without efflorescence or less than 5 percent of the assembly surface, the assemblies with efflorescence that doesn't appear after the humid sponge rubbing and the assemblies with efflorescence that appears after the humid sponge rubbing.

4 TEST RESULTS AND VALIDATION

In order to assess the reproducibility of the method, each combination has been tested five times on the sixteen combinations and the results are given here under on table 5. For the validation, we have opted for two different ways: the exposure site (presented here above) and the observation of real cases collected in a data base.

4.1 Exposure site

With a survey of two years, we have accurate information concerning the most sensible brick/mortar combinations to the efflorescence phenomenon among the research combinations. This information constitutes a validation base for the laboratory test results.

The table 5 contains the comparison between the test and the exposure site results in terms of apparition percentage. For the test, the percentage is the ratio between the apparition's number and the tests number multiplied by 100. For the exposure site, the percentage is the ratio between the number of days where efflorescence is detected and the total number of days where the site is visited multiplied by 100.

Apparition	*M11*				*M12*				*M13*			
percentage	*B*	*Q*	*S*	*V*	*B*	*Q*	*S*	*V*	*B*	*Q*	*S*	*V*
Test [%]	40	**20**	**80**	100	**60**	**0**	**100**	80	80	**20**	**100**	**20**
Exposure Site [%]	54	**8**	**61**	12	**27**	**6**	**41**	9	16	**12**	**41**	**0**

Table 5. Comparison between test and exposure site results.

The daily visual survey of the exposure site is done sometimes during a humidification phase (just after a rain); so, the percentage has to be consider differently between the site and the test. We give in the table 6 the agreement between the two.

Gravity	*Test apparition [%]*	*Exposure site apparition [%]*
High	[100 - 80]	> 30
Medium	[60 - 40]	[30 - 15]
Low	[20 - 0]	< 15

Table 6. Agreement between the test and the exposure site results .

The analysis of he Table 5 shows that the test and the exposure site are inclined to reveal the same combinations sensible to the efflorescence (in the table, the results from the same gravity category are in bold), except for the combinations with the brick V for which the results are really different. In fact, efflorescence on the brick V contains gypsum (detected by X Ray Diffraction analysis) and this salt is very sensible to the humidity [Muzzin 1982]. The test drying conditions are sufficient to see the gypsum when the external conditions are unusually favourable.

4.2 Real cases

Some Belgian contractors act at our request a data base with building case that presents efflorescence. The latest validation phase will be to apply the efflorescence test with the bricks and mortars of the data base in order to verify that the test results confirm the real case establishment. Currently, we have more than seventy cases.

5 FUTURE PERSPECTIVE

A new efflorescence test method is currently established after having improved his constitutive parameters with laboratory experiments. The test results confrontation to the behaviour of an exposure site built with the same combinations shows that the test is in accordance with it. The future perspectives concern an improvement of the drying phase by using IR lamps in order to cause a thermal gradient between the centre of the assembly and its surface and then to collate the test to a great number of real cases. Finally, the test method will be proposed at the European normalisation level in order to complete the masonry norm.

6 ACKNOWLEDGMENTS

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7 REFERENCES

Barquin, F. de, Gérard, R., Elsen, J., Deplus, P., Dugniolle, E. & Muzzin, G. 1996, 'Les efflorescences sur les maçonneries de briques', *CSTC magazine* printemps, 12-21.

Brocken H. & Nijland T. G. 2004, 'White efflorescence on brick masonry: towards prediction of efflorescence risk', Proc. 13th International Brick and Block Masonry Conference, Amsterdam, Nederlands, 4 7 July, vol. 3, pp. 799-808.

Brocken H. & Nijland T.G. 2004, 'Witte uitslag op baksteen- en betonsteenmetselwerk', *Cement*, **7**, 80-85.

Carpentier, G., De Kesel, J.-P., Hens, H., Uyttenbroeck, J. & Vaes, F. 1982, 'Comportement à l'humidité des éléments de construction', *CSTC revue*, n°1, 17ème année, 4-6.

Muzzin, G. 1982, 'Les efflorescences dans les maçonneries en briques', CSTC revue, n°4, 17ème année, 2-11.

Influence of Sorption Moistening in Research of Moisture-Caused Deformations of Construction Materials

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ABSTRACT

Investigation methods described in literature and standards do not evaluate the influence of sorption moisture upon moisture-caused deformations of building materials. Therefore, the usual deformations measuring methods had to be amended by measuring moisture deformations in various models and by setting sorption moisture. The improved methods was applied for solving the tasks of durability of articles. The basic point of the developed methods intended for investigation of moisture-caused deformations is investigation of the groups of specimens by measuring deformations in the environment of fixed sorption moisture using four models. According to the methods described in this article, the dependence of deformations of various construction materials (concrete, porous concrete, silicate and ceramic bricks, cement-lime plasters) upon moisture and changes therein was investigated. Analysing of materials with different capillary structures enables us to provide – in a very concise form – some references concerning value and nature of deformations. Linear moisture-caused deformations of basic construction materials (concrete, silicate and ceramic bricks, cement-lime plasters, porous concrete) vary within the limits of 0,33 – 0,77 mm/m. In case of articles containing organic fillings they can reach up to 6 mm/m. When the material is of a fine capillary structure the highest relative moisture elongation *K* of a material is observed in the environment of vapour area. Relative moisture elongation of the tested materials with the outset of intense capillary condensation varies from 0,05 mm/(m·%) to 0,51 mm/(m·%).

KEYWORDS

Building materials, moisture-caused deformation, water vapour sorption, improved methods.

1 INTRODUCTION

In case of porous materials, no equilibrium of molecular forces exists among the particles on the surface of the pores, and surface tension occur in the framework of a solid body [Lentinen 1996; Carmeliet & Roels 1996; Semerak & Cerny 1996]. With moistening of a capillary-porous body and under the action of capillary forces the volume of the body increases due to increase of dimensions of material polycrystalline structure.
In case of crystals, moisture adsorption reduces the forces of molecular links (bonds) between the layers and increases the distance between them.

With increased time of exposure to the environment, materials begin to degrade, which can result in either a catastrophic failure or gradual drift out of tolerance of a critical performance property of the material. The durability problem is concerned with the deterioration of the material to such a level that an undesirable or unsafe condition for the material or component is attained [Cerny *et al.* 1996; Miniotaite 1999, 2001; Carmeliet 2001].

Fluid pressure variations in porous materials not only involve mass transfer, but also deformations of the porous material. These deformations, when restrained, can lead to damage and severe cracking [Miniotaite 2001; Hale 1976].

With elimination of moisture from the pores, molecular bond increases and initiates shrinkage of materials. It is supposed that in the environment of super-sorption moisture, the action of capillary forces remains constant due to saturated vapour pressure in the pores; with increase of moisture up to maximum water absorption, the material does not undergo further deformation [Miniotaite 1999, 2001; Freitas 1996].

Investigation methods intended for investigation of moisture-caused deformations of moistened and soaked specimen are described in literature and standards. They are not related to variable processes of sorption and desorption. [Miniotaite 2001; Hansen *et al.* 1995; Aniskevich 1995]. The influence of sorption moistening values upon moisture-caused deformations of materials are not still determined. Therefore, the usual deformations measuring methods had to be amended by measuring moisture deformations in various models and by setting sorption moisture [Miniotaite 2001]. The improved methods was applied for solving the tasks of durability of articles.

2 INVESTIGATIONS OF MOISTURE-CAUSED DEFORMATIONS

According to the methods described in this article, the dependence of deformations (ε_u) of various construction materials (concrete, silicate and ceramic bricks, cement-lime plasters, porous concrete) upon moisture and changes therein was investigated: $\varepsilon_u = f(u)$. Investigation methods were improved and developed on the ground of a cement-wood chip board example; afterwards they were employed for investigation of other materials. The basic point of the developed methods intended for investigation of moisture-caused deformations is investigation of the groups of specimens by measuring deformations in the environment of fixed sorption moisture using four models [Miniotaite 1999]:

Model *I* – specimens were soaked in water at temperature (18 - 25) ^{o}C till absolute absorption. Afterwards, the specimens were dried under room conditions at the air temperature (18 ± 2) ^{o}C and RH (relative humidity) ϕ = (45 - 50) % until equilibrium state was reached. The above state reached, the specimens were dried at an ambient temperature (105 ± 2) ^{o}C until a dry state was achieved;

Model *II* – specimens were soaked at 60 ^{o}C in polyethylene bags. The bags contained such volume of water which was almost totally absorbed by the specimen. Then the specimens were dried at the same temperature in empty (no water) polyethylene bags so that more identical conditions were created for distribution of moisture;

Model *III* – specimens were tested under the conditions identical to those of Model *I*, however, they were moistened gradually in the course of entire sorption process up to complete absorption, while soaked in water;

Model *IV* – specimens were tested under the conditions identical to those of Model *II*, however, they were moistened gradually according to sorption process and then up to the total absorption.

For evaluation of the deformations, the length of a dry specimen was taken as the basic length. The length of each specimen were measured at 0,0001 mm; the mass accuracy of 0,01 g. Moisture-caused deformations of construction materials were measured simultaneously with gradual moistening and then gradually dried following achievement of maximum absorption.

Moisture-caused deformations of specimens were valued by two parameters [Miniotaite 1999, 2001]:
1. Linear deformations of a material ε_u, [mm/m]:

$$\Delta l = l_u - l_b \,; \qquad \varepsilon_u = \frac{\Delta l}{l_b}\,; \tag{1}$$

where l_b – basic length of a specimen, mm; l_u – measured length of a damp specimen, mm.

2. Relative moisture elongation of material K, [mm/(m·%)]:

$$K = \frac{\varepsilon_u}{\Delta u}\,; \tag{2}$$

where $\Delta u = (u_2 - u_1)$, % – accrual of material moisture (essentially, the sorption moisture), %, within the interval from moisture u_1 to moisture u_2.

K is a significant comparative property of the material indicating relative change of material length with moisture change of one percent.

The process of moisture transfer is an inert one and therefore the experiment of investigation of moisture-caused deformation is time-consuming; precision of measurements depends on escaping from the influence of accidental effects. Measuring practices indicated that negligible accidental deviations resulted in distortion of the result and required repetition of the experiment already well-advanced.

3 RESULTS AND DISCUSSION

In Fig. 1 the field of iso-lines of deformations was divided into three areas: in case of the 1st area, the moisture of the specimen corresponds to vapour-state sorption moisture of the material; including adsorption moisture (*1-2*); in case of the 2nd area (*3*), moisture of the specimen complies with that of an intensive capillary condensation; and in case of the 3rd area (*4*) the specimen was moistened by free moisture. In Fig. 2, the areas of moisture states are marked "*1-2*", "*3*" & "*4*".

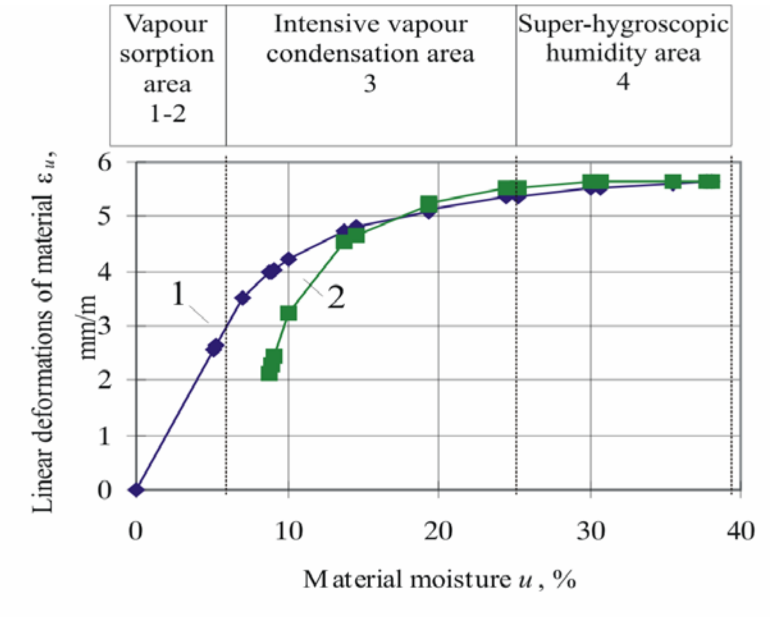

Figure 1. Linear deformations caused by moistening (1) and drying (2) of cement-wood chip boards according to Model *III*.

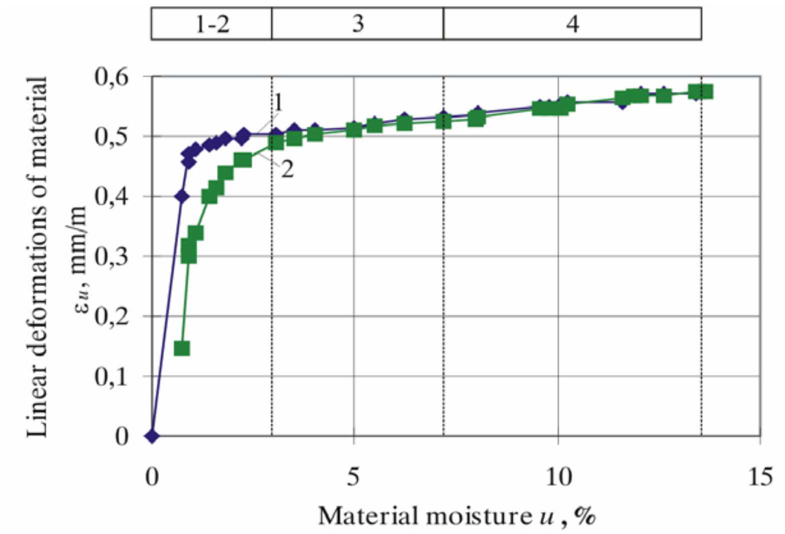

Figure 2. Linear deformations caused by moistening (1) and drying (2) silicate brick according to Model *III*.

Analysing of moisture-caused deformations of materials with different capillary structures enables us to provide – in a very concise form – some references concerning volume and nature of deformations.

Relative moisture elongation of materials of the articles possessing fine capillary and homogenous structure, e.g. deformations of a silicate brick, grew abreptly and rapidly in the area of sorption moisture (Fig. 2) [Miniotaite 2001]. It was determined that when the articles were dried in the environment of temperature 105 ^{o}C, about 0,5% of non-eliminated moisture remained therein, and residual deformations proportional to the quantity of non-eliminated moisture were observed.

Weakly-bonded structures such as specimens of lime mortar undergo deformations in quite a different way (Fig. 3) [Miniotaite 2001]. When soaked in water, they swell rapidly. When water temperature reaches 60 ^{o}C and water viscosity decreases (according to Model *II*), the plaster gets water-saturated in several minutes. Moisture-caused deformations do not increase with further moistening. Residual deformations of the specimens correspond to the value reached during intense swelling.

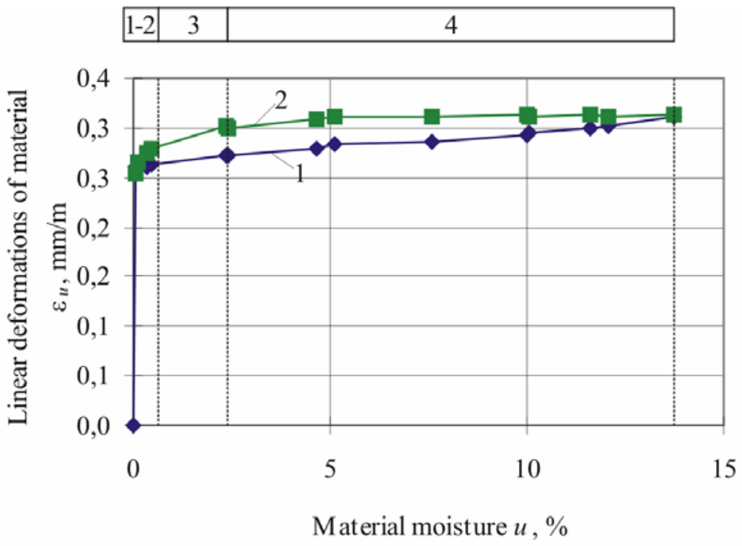

Figure 3. Linear deformations caused by moistening (1) and drying (2) of lime plaster (1:3) according to Model *II*.

Intensive swelling was also observed while testing lime cement according to Models *I* (Fig. 4) & *II* (soaking in water). In the above case, high hysteresis of isotherms' drying and moistening as well as a characteristic shape of moistening isotherm were observed. Within the interval u = (1,5 - 4)% (intense capillary condensation area), relative moisture elongation of the lime cement $K \approx 0$ (the material stops to extend). Afterwards, the elongation slowly grows, later more rapidly up to the limit characteristic of the lime cement. The above can be explained by the presence of two materials (lime and cement) with different abilities of water absorption. At the beginning the lime absolutely disturbs for cement to adsorb water vapour (u < 4%). During saturation the lime less and less disturb for cement to absorb water. Relative moisture elongation at the end of saturation reaches value K = 0,12 mm/(m·%) ($u \approx$ 15%). During the drying process of the plaster the recurrence of volume is very slow, the hysteresis is high.

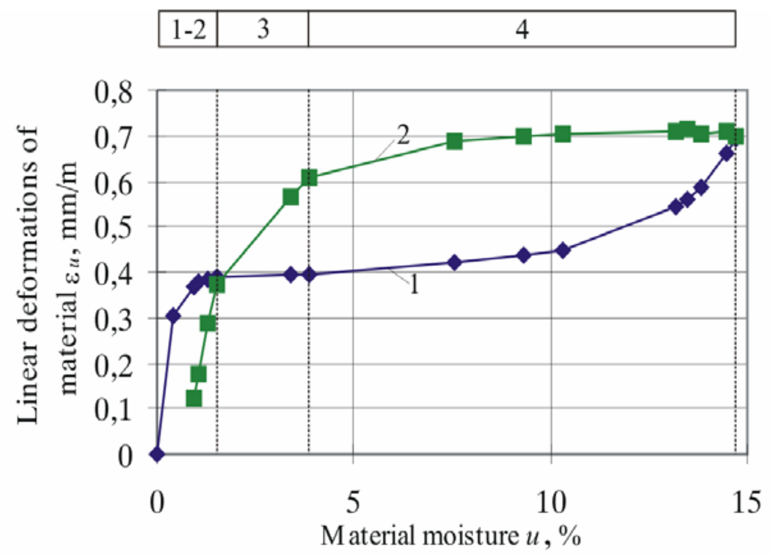

Figure 4. Linear deformations caused by moistening (1) and drying (2) of cement-lime plaster (1:1,2:6,8) according to Model *I*.

When heavy concrete is moistened at 20 ^oC temperature environment according to Model *I* (Fig. 5), moistening rate is practically constant until complete humidification in the area of capillary condensation.

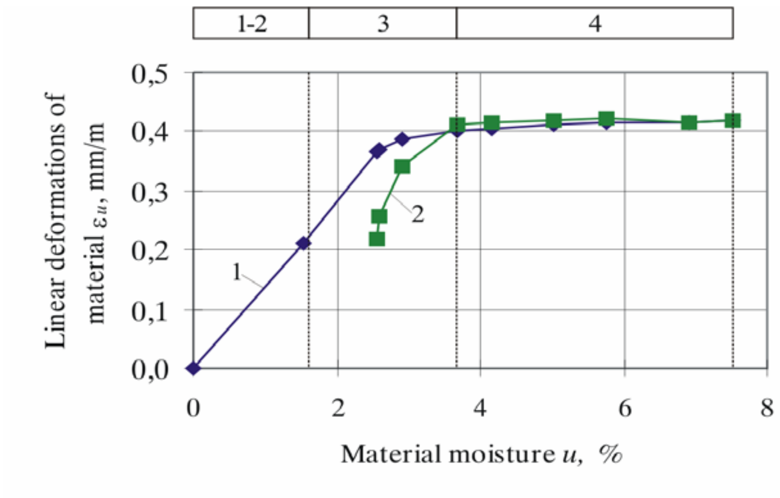

Figure 5. Linear deformations caused by moistening (1) and drying (2) of concrete according to Model *I*.

Samples of porous concrete, tested according to Models *I* (Fig. 6) & *II* (temperature θ, 20 ^{o} and 60 ^oC respectively; moistened by direct contact with water) up to the moistening corresponding to the (*1-2*) sorption area, were also getting intensively moistened in proportion to ambient moisture accrual. In this case, relative moisture elongation of material K = (0,46-0,57) mm/(m·%), at humidity level up to $u \leq$ (3-4)% depends on the ratio of rapidly water-filled open pores and narrow water-sucking capillaries linking the pores. The more open porosity is, the bigger is the relative moisture elongation K of the material and, consequently, more rapidly linear deformations develop up to the limit of their moisture-caused deformation. Isotherms of drying practically correspond to moistening isotherms up to polymolecular (filmy) level of moisture; once the level is reached they practically break - residual deformations and residual moisture are already fixed in the gas concrete. The influence of temperature is low and fits within the limits of testing precision.

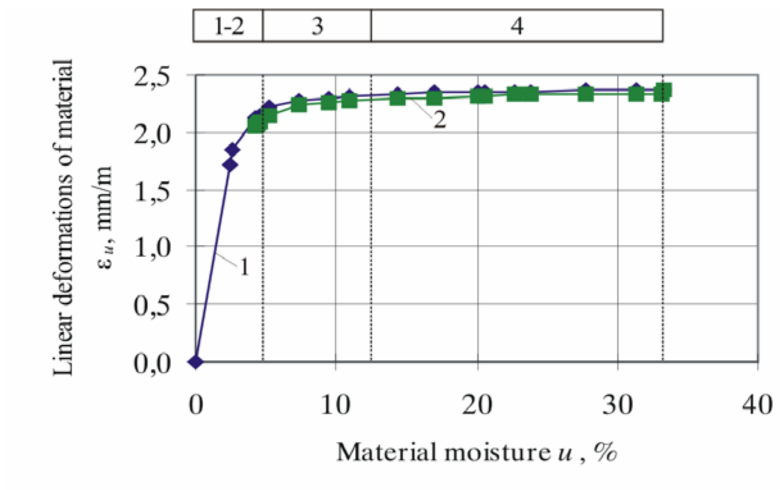

Figure 6. Linear deformations caused by moistening (1) and drying (2) of porous concrete according to Model *I*.

Investigation results indicated that functional dependence of material deformations upon moisture might be put into a certain variation curve close to that of parabola; different are function variation

parameters which depend on the nature of the material, microstructure and molecular link of moisture with the framework of the material.
However, variation parameters of the function $\varepsilon_u = f(u)$ are different for each material and should be calculated individually.

4 CONCLUSIONS

1. All tested construction materials usually get deformed in the area of sorption moisture before reaching the state of intense capillary condensation. Moisture-caused deformations finally settle in the state of intense capillary condensation.
2. Linear moisture-caused deformations of basic construction materials (concrete, silicate and ceramic bricks, cement-lime plasters, porous concrete) vary within the limits of 0,33-0,77 mm/m. In case of articles containing organic fillings linear moisture-caused deformations can reach up to 6 mm/m.
3. In case of a material containing plastifying additives, including lime, residual moisture and residual deformations are observed.
4. The highest relative moisture elongation *K* of a material is observed in the environment of vapour area; it is directly proportional to moisture growth at a large relative surface of capillaries and pores per volume unit, i.e. when the material is of a fine capillary structure. Relative moisture elongation *K* of the tested materials with the outset of intense capillary condensation varies from 0,05 mm/(m·%) to 0,51 mm/(m·%).

5 REFERENCES

Miniotaite, R. 2001, *The durability of finishing layer external surface of buildings' wall's.* Monograph, Lithuania, 175 p.

Miniotaite, R. 1999, *Compatibility of finishing layer and external surface of buildings' walls from the standpoint of durability*, Doctoral Dissertation, Lithuania 180 p.

Carmeliet, J. and Roels, S. 2001, 'Determination of the isothermal moisture transport properties of porous building materials', in J*ournal of Thermal Envelope & Building Science,* vol. 24, pp. 183–210.

Freitas, V.P., Abrantes, V., Crausse, P.1996, 'Moisture migration in building walls – analysis of the interface phenomena', in *Building and Environment,* vol. 31, No 2, pp. 99–108.

Hale, D.K. 1976, 'The physical properties of composite materials', in *Journal Material Science,* vol. 11, pp. 2105–2141.

Semerak, P., Cerny, R. 1996, 'A capacitance method for measuring moisture content of building materials', in *Stavebni obzor*, vol 6, pp. 102–103.

Aniskevich, A. 1995, 'Moisture sorption in aramid composite on different structural and physical levels', Proc. Tenth Int. Conf. on Composite Materials (ICCM-10), Whistler, British Columbia, Canada, Aug. 14-18, 1995, vol. IV: Characterization and Ceramic Matrix Composites. – Abington: Woodhead Publ. Ltd, pp. 367–374.

Carmeliet, J. & Roels, S. 1996, 'Moisture transfer and durability of open porous media', Proc. 4th Symposium on Building Physics in the Nordic Countries, Finland, vol. 2, pp. 587–594.

Cerny, R., Drchalova, J., Hoskova, S., Toman, J. 1996, 'Inverse problems of moisture transport in porous materials'. Proc. Second ECCOMAS Conf. on numerical methods in engineering, pp. 664–670.

Hansen, K.K., Baroghel, V.B., Quenard, D. and Künzel, H. 1995, 'Water vapour absorption isotherms for porous building materials', Proc. Int. Symposium on moisture problems in building walls, Portugal, pp. 248–257.

Lentinen, T. 1996, 'Capillary moisture transfer in combined porous building materials', Proc. 4th Symposium on Building Physics in the Nordic Countries, Finland, vol. 2, pp. 483–490.

The Compatibility of Finish and Base Coatings of External Buildings Walls Influenced by Environmental Effects

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ABSTRACT

The external surface of building walls is continuously effected by the natural climate of variable intensity and the factors occurring due to the anthropogenic activity. The heat and mass exchange between outside air and walls is the most distinct in a relatively thin (1 - 3 cm) surface layer. The activity of physical processes, substantial variations of temperature and moisture and other effects of various origins are particularly distinct in that layer. Durability of the surfaces depends on the prevailing climate effects and on a complex of physical and mechanical values of the materials used. The complex and partial methods have been worked out for application at investigation of destruction processes. Partial methods have been applied in the cases of investigation of the physical and mechanical characteristics, which are necessary to be evaluated before direct weather durability test in the climatic chamber along the program of simulated effects. It is established that the paint under consideration should be grouped according to their composition and characteristics of their macrostructure and base or primer layer to be covered by paint according to their susceptibility to moisture absorption (i.e., capillary structure) and deformation.

KEYWORDS

Paints, physical and mechanical properties, investigation method, durability .

1 INTRODUCTION

The physical and mechanical properties of construction substrate and finishing layers can supplement one another or, on the contrary, stimulate destruction [Lentinen 1996; Miniotaite 1996, 1999, 2001].

In the literature [Lentinen 1996; Puterman 1996; Sandin 1996; Hale 1976] much information is given on paint and coatings, including the physical and chemical nature of paints, structure formation, and the results of investigations of physical and mechanical values. However, the data of complex investigations of the surfaces already coated are insufficient. Usually the physical and mechanical values of individual components – the coating and the wall being painted – are known. However, the knowledge about the resulting of the whole complex of properties of a new derivative – the "surface layer" of the material – is insufficient.

Differences of moisture and temperature cause deformations of the coating and the substrate, which may cause internal stress in the coating and shift in the joint, that can be increased by the pressure of vapour migration to outside [Miniotaite 1999, 2001]. The resistance to shift stresses can be defined by mutual adhesion of materials, which is not yet well enough investigated. On the other hand, for an exterior wall, vapour migration from the interior to the outside and moisturepick-up by the external layers show contrary effects [Carmeliet & Roels 2001; Freitas *et al.* 1996]. This phenomenon creates a certain collision between the necessity to thicken paints against rain penetration and, vice versa, to thin the thickness of paints, so that they do not accumulate vapour between the layers, but allow vapour migration freely so that the material beyond the vapour barrier does not undergo delamination.

The investigation of the external layer of walls and durability of different paints is reported in the present paper.

2 MATERIALS AND INVESTIGATION METHODS

In the literature [Miniotaite 1999, 2001; Carmeliet & Roels 2001; Freitas *et al.* 1996; Brocken *et al.* 1998; Noghabai 1996] we find some data on the physical and mechanical properties of materials intended for finishing, which predetermine the durability of such materials. However, these data are often insufficient and fragmentary. Therefore, the partial comparative investigations have been carried out.

In order to properly investigate the durability of paints, it is necessary to compose a methodical chain - the scheme of stage-by-stage (intermediate) and chamber-type consecutive investigations (Fig. 1).

The methods of generalised complex investigations were designed on the basis of the results of investigations carried out according to the stage-by-stage methods.

It was found that the specific physical and mechanical properties determined for the paints alone may change in the new combination of "paint - substrate". The comparative results of durability were obtained by the classification of coatings in three groups according to their structural nature: 1) paints formed out of aqueous polymeric dispersions; 2) silicate paints; 3) paints formed out of polyacrylates and silicone solutions in organic solvents, or silicone dispersions.

In the case of a bi-laminar system "paint film - wall painted" we encounter water (i.e., rain) flowing from outside towards the wall and water vapour migrating from the wall to outside. The optimum selection of the paint is necessary for the wall to be painted. The water vapour may accumulate in the wall, when precluded from escaping through a very dense (vapour - tight) film, which might in turn cause blisters or result in delamination of the whole film or at least some sections.

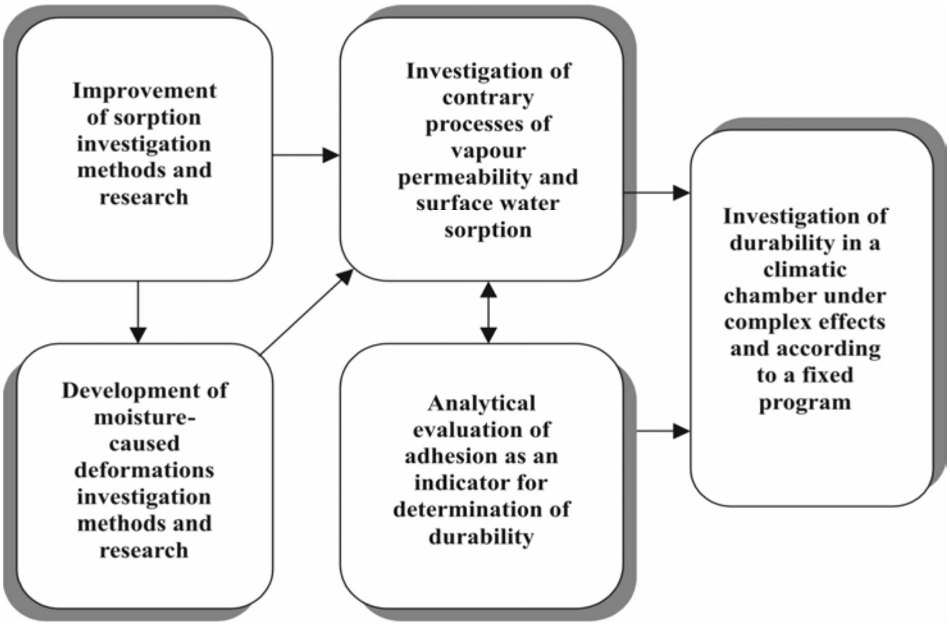

Figure 1. Principal scheme of complex research methods.

Several hundreds of combinations of coating and substrate are possible when a number of surfaces are coated with various paints in this study, silicate brick was chosen for investigation. The choice was predetermined by the advantages of the silicate brick surface and its homogeneous capillary structure in order to have the results less scattered. The moisture diffusion processes in the silicate brick are comparatively constant across the surface [Freitas *et al.* 1996; Brocken *et al.* 1998].

The brick was coated using paints of different origin (total 26 compositions). Analyses of compositions of the paints indicate that vapour permeability depends on the paint used, polarity of film-makers, and bonding agents used.

Water vapour permeability coefficient was determined in 20 ^oC environment according to requirements of the EN ISO 12572:2001 [EN ISO 12572 2001]. Measurements were performed using 3 specimens of 100 mm diameter and 25 mm thickness of the uncoated brick and others materials (concretes, mortars) and 3 specimens with surfaces painted for each of the 26 coatings in the study. The painted specimens were fixed on a cup, paint facing down (cup method).

Water vapour resistance Z_p, [$m^2 \cdot h \cdot Pa/mg$] is in reverse proportion to vapour permeability δ_p, [$mg/(m \cdot h \cdot Pa)$]:

$$Z_p = \frac{d_x}{\delta_p}, \quad (1)$$

where d_x is the thickness of samples of bricks, in meters.

The specimens used for determination of vapour permeability were also used for determination of the surface water sorption coefficient by DIN 52 617 [DIN 52 617 1987].

The specimens oriented with the paint facing downward were soaked in a water bath maintained at 20 ^oC temperature.

Water sorption coefficient w, [$kg/(m^2 \cdot h^{0.5})$] is calculated:

$$w = \frac{m}{\sqrt{t}}, \qquad (2)$$

where m is a mass of absorbed water related to 1 square meter of sample, in kg/m^2; t – duration of soaking, in hours.

The basic conditions and means used for climatic tests were as follows:
a) in the warm part of the chamber room temperature is automatically maintained at θ_i = (18 ± 2) °C and RH ϕ = (50 - 70)%;
b) an automatic climatic regime was maintained in the cold part of the chamber:

- room temperature during 15 hours freezing down to θ_e = -(15 ± 5) °C;
- the temperature of a protective finished layer of the wall θ_{se} = (15 - 20) °C during 8 hours reheating;
- UV light lamp was used during the last hour of heating; irradiation intensivity 600 W/m^2;
- in the cold part of the chamber, water-spray equipment was installed. During a one hour water-spary operation, the finish of the wall had to be covered by a uniform water film (with a spray intensity = 1 L/m^2 min, temperature θ = (7 - 12) °C, and water pressure = 0.15 MPa);

air circulation at the velocity of v = (2 - 4) m/s was maintained by a ventilating device installed in the cold part of the chamber.

3 RESULTS AND ANALYSIS

Physical properties (i.e., vapour resistance and water sorption coefficient) of the surface layer (0.025 m) of silicate bricks coated with aqueous polymeric dispersion paints are compared in Fig. 2 [Miniotaite 1999].

The aqueous polymeric dispersion paints are distributed according to the water sorption coefficient in two subgroups: a) *A1, A2, A3, A4* [w > 0.60 kg/(m^2·h$^{0.5}$)] (Fig. 2); b) *A5, A6, A7, A8, A9* [w ≤ 0.10 kg/(m^2·h$^{0.5}$)] (Fig. 3). As can be seen an the increase of vapour resistance of less than a factor of two decreases the water sorption coefficient more than a factor of 30. The water sorption coefficients of paints *A4* and *A5* in separate subgroups vary by a factor of 6.

Fig. 2 shows that surface water absorption in the case of the paints of subgroup "a" is high – these are relatively "rain permeable" paints: w = (0.81 - 0.60) kg/(m^2·h$^{0.5}$). Their vapour resistance Z_p = (0.47 - 0.62) m^2·h·Pa/mg [δ_p = (0.053 - 0.041) mg/(m·h·Pa)].

Surface water absorption in the case of paints of subgroup "b" is low – these are rather "tight" paints w = (0.024 - 0.10) kg/(m^2·h$^{0.5}$) (Fig.2).

Their vapour resistance Z_p = (0.65 - 0.84) m^2·h·Pa/mg [δ_p = (0.038 - 0.030) mg/(m·h·Pa)] is on the average 35% higher than those of subgroup "a".

As was foreseen before beginning these investigations the theoretical characterization of the paints (low vapour resistance - low rain penetration – "good"; high vapour resistance – high rain penetration – "bad") can be insufficient to evaluate a paint's durability.

Therefore, during the investigations changes in the surface layer resulting in decreases in adhesion between a paint and the silicate brick, were fixed. After the exposures in the climate chamber were finished and the results were analyzed, the reliability of the theoretical statement was verified.

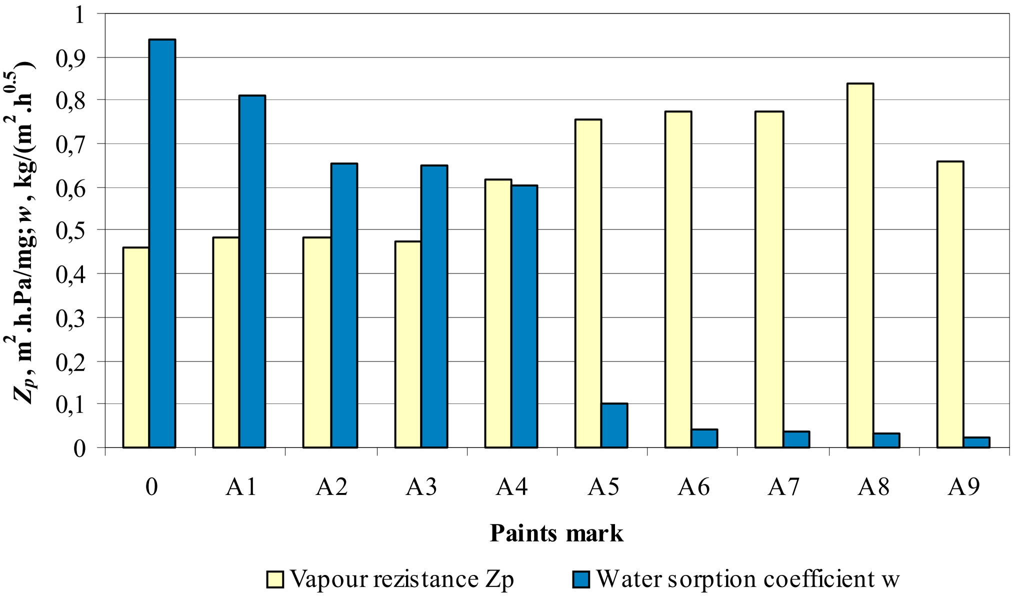

Figure 2. Comparison of variation in the vapour resistance and in the water sorption coefficient of the aqueous polymeric disperse paints applied on silicate bricks.
Notes: paints mark 0 - non-painted silicate brick.

The value of vapour resistance due to complex effects created by application of the coating on the silicate bricks was compared with the resistance of non-painted silicate bricks surface for the selected cycles.

The tests carried out in the chamber indicated that the paints of identical vapour resistance could be compared even though the nature of deterioration and ageing as well as protective significance of such paints for the painted surface were different. The mechanism of such differences is explained taking into account additionally the complex effects of the moisture-caused deformations of the substrate and physical and mechanical values of the paints [Sandin 1996].
Paint - substrate adhesion is explained by interaction between polar and ionic groups of the bonding agent of paint and functional groups of substrate surface.

Durability of the aqueous polymeric disperse paints is described (considering two basic physical properties) in Fig. 3.

In the all cases the vapour resistance increase was influenced by using the acrylic bonding agent in appropriate proportions. However, the increase of vapour resistance is permissible and does not reduce durability of a properly selected composition of the paint. In the case of subgroup “a” permissible water sorption coefficient $w < 0.65$ kg/(m^2·h$^{0.5}$) and the highest value of vapour resistance $Z_p < 0.62$ m^2·h·Pa/mg [$\delta_p > 0.041$ mg/(m·h·Pa)] are suitable with respect to durability. In the case of subgroup “b” all physical parameters are high enough. The reason for classifying two paints *A6* and *A9* as non-durable considers the bonding agent: specifically, the amount of bonding agent was insufficient. The interaction between paint and silicate brick was inadequate.

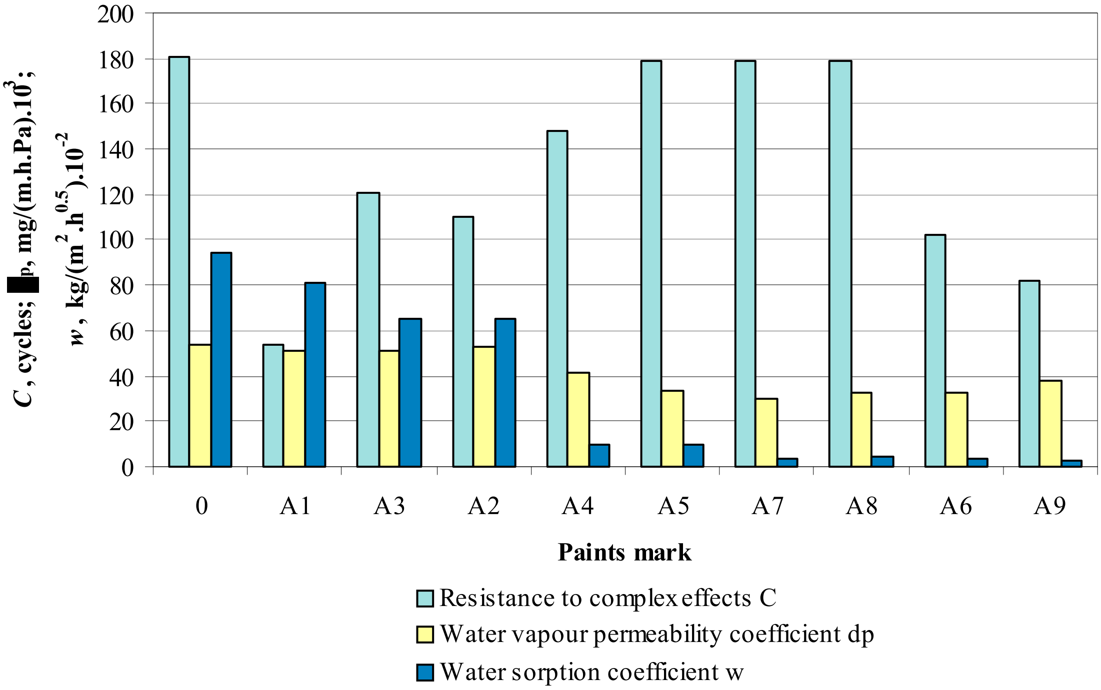

Figure 3. Resistance to climate effects of aqueous polymeric disperse paints on silicate brick walls considering water sorption coefficient and vapour permeability.

The deterioration of silicate bricks in climatic chamber was compared with the natural 12 years duration observation.

Durability of paints in the case of insufficiently stabilized compositions of the aqueous polymeric dispersions is defined as C < 80 cycles irrespective of the values of the water sorption and vapour permeability coefficients. Destruction of the paint specimens in the study is manifest through fast wrinkling of the film, mould formation, loss in the adhesion (small blisters) and washing off after 50-80 cycles.
150 accelerated cycles in the climatic chamber correspond to 12 years at an average natural ageing.

The resistance of the non-painted silicate bricks surface was found to be about 180 cycles. Following the effect of 170-180 cycles, the hydrosilicate crystalline structure of the silicate bricks surface thin layer (0.05-0.2 mm) underwent deterioration. Ground sand particles together with hydrosilicate deterioration products, dirt and other adhered aerosol inclusions crumbled off or were washed away comparatively easily.

By analysing the results of grouped paints it was found that peculiar nature of vapour permeability and water sorption was typical of each group; distibution of the paint destruction is different. The nature of paint destruction depends on the moisture-caused deformation of the substrate on the different level.

Aqueous polymeric disperse paints destruction on building materials is indicated in separate photo Fig. 4, Fig. 5, Fig. 6.

Figure 4. Aqueous polymeric disperse paints A6 after 96 cycles of complex effects (substrate – lime-cement mortar).

Figure 5. Aqueous polymeric disperse paints A2 after 85 cycles of complex effects (substrate – lime-cement mortar).

Figure 6. Aqueous polymeric disperse paints A4 after 140 cycles of complex effects (substrate – lime-cement mortar).

4 CONCLUSIONS

1. Investigation on the durability of the building walls external surfaces paints by modelled complex effects in a climatic chamber is purposive only after intermediate investigations and measurements of the substrate physical and mechanical properties that aid in predetermining durability, that is, sorption-desorption and moisture-caused deformations. The effect of the above mentioned properties upon durability should be evaluated.

2. Proper determination of paint durability is possible only in the case of simultaneous investigation of the wall surface layer.

3. Influence of moisture deformations upon degradation of coatings depends on the porosity of materials of the surface being coated and on the origin and macrostructure of the coating.

4. The nature and signs of deterioration and ageing of paints of the separate groups develop in a different way.

5. The intensity of surface destruction is non-proportional to the number of testing cycles. The increase of the number of modelled cycles accelerates destruction processes.

6. Durability of the paints formed out of aqueous polymeric dispersions as well as the subgroups of paints is distributed (not so distinctly due to some decrease in water vapour permeability) in the direction of fast decrease of water sorption coefficient.

7. Durable coatings should be considered to be thoses which have withstood from 150 to 180 modelled complex cycles of climate effects. The coatings of acceptable (permissible) durability are those withstanding 110-150 cycles. Non-durable coatings are those withstanding less than 100-110 cycles - their aesthetic depreciation usually starts following 30-50 testing cycles. The coatings which begin to deteriorate after 80-100 cycles are not recommended either.

5 REFERENCES

Miniotaite, R. 2001, *The durability of finishing layer external surface of buildings' wall's,* Monograph, Lithuania, 175 p.

Miniotaite, R. 1999, *Compatibility of finishing layer and external surface of buildings' walls from the standpoint of durability*, Doctoral Dissertation, Lithuania 180 p.

Brocken, H. J. P., Spiekman, M. E., Pel, L., Kopinga, K., Larbi, J. A. 1998, 'Water extraction out of mortar during brick laying: A NMR Study', in *Materials and Structures,* vol. 31, No 5, pp. 49–57.

Carmeliet, J. and Roels, S. 2001, 'Determination of the isothermal moisture transport properties of porous building materials', in J*ournal of Thermal Envelope & Building Science,* vol. 24, pp. 183–210.

Freitas, V.P., Abrantes, V., Crausse, P. 1996, 'Moisture migration in building walls – analysis of the interface phenomena', in *Building and Environment,* vol. 31, No 2, pp. 99–108.

Hale, D.K. 1976, 'The physical properties of composite materials', in *Journal Material Science,* vol. 11, pp. 2105–2141.

Lentinen, T. 1996, 'Capillary moisture transfer in combined porous building materials', Proc. 4th Symposium on Building Physics in the Nordic Countries, Finland, vol. 2, pp. 483–490.

Miniotaite, R. 1996, 'Determination method for weather durability of finish layers in external walls of buildings', Proc. Conf. on the subject of Construction and Architecture, Kaunas University of Technology, Institute of Architecture and Construction, Lithuania, pp. 140–145.

Miniotaite, R. 1999, 'Moisture-caused deformations of construction materials', Proc. 5th Symposium on Building Physics in the Nordic Countries, Goteborg, Sweden, vol. 2, pp. 581–588.

Noghabai, K. 1996, 'Environmental effects on bond in reinforced concrete structures'. Proc. 7 Int. Conf. Durability of Building Materials and Components, vol 1, pp. 605–614.

Sandin, K.1996, 'Some facts about hydrophobic facade impregnation', Proc. 4th Symposium on Building Physics in the Nordic Countries, Finland, vol. 2, pp. 515–521.

Puterman, M. 1996, 'Surface deterioration of building paints exposed to natural and accelerated weathering', Proc. 7 Int. Conf. Durability of Building Materials and Components, vol. 1, pp. 658–666.

DIN 52 617, 1987. Determination of the water absorption coefficient of building materials.

EN ISO 12572, 2001. Building materials. hydrothermal performance of building materials and products. determination of water vapour transmission properties.

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Durability of Laminated Veneer Lumber made from Blackbutt (Eucalyptus Pilularis)

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ABSTRACT

Blackbutt (*Eucalyptus Pilularis*) is a common plantation hardwood in New South Wales, Australia, highly regarded for its strength and durability but considered difficult to laminate for "engineered timber" because of chemicals present in the timber (extractives). While previous work by State Forests New South Wales pointed to a viable lamination method using phenolic tannin glues, plywood made from Blackbutt has shown glue failure in exterior applications. At the University of New South Wales (UNSW), a lamination technique was investigated for Blackbutt Laminated Veneer Lumber (LVL) and a cleavage fracture toughness method was adapted to quantify the toughness of its glue-lines.

The durability of Blackbutt LVL with differing extractive content was explored by assessing fracture toughness of glue-lines after exposure to one of three artificial weathering environments and a marine inter-tidal zone. Specimens subjected to the most aggressive laboratory environment and those immersed in the inter-tidal zone showed some loss of fracture toughness with increasing exposure, however Blackbutt LVL was shown to be much more durable than Pine LVL. Exposure to the adverse environments had not compromised the nature of the glue-lines and the mean toughness values remained relatively high at approximately 400 J/m^2. The results suggest that Blackbutt veneer is capable of being glued for application as a durable structural LVL.

KEYWORDS

Blackbutt, Cleavage Fracture, LVL .

2 INTRODUCTION

There are many reasons for the increasing popularity of engineered timber in structural applications: it is sourced from a renewable resource which absorbs carbon, its material properties are less variable than those of sawn timber and its timber elements can be obtained from younger plantation trees. Engineered timbers however, are generally made up by the gluing together of components and glue-line failures have contributed to durability problems.

This project examined "Laminated Veneer Lumber" (LVL) made from the Australian hardwood, Blackbutt. Unlike plywood, the grains of adjacent veneers are parallel. The randomisation of defects leads to better and more uniform mechanical properties than those of the parent timber. In Australia, although pine LVL is now commonly used for structural elements in protected environments, it is not sufficiently durable to be used outside. Blackbutt LVL offers the promise of strength and durability.

Blackbutt is a common plantation species in New South Wales - it produces a strong and durable timber. Although Blackbutt is difficult to glue because of natural chemicals (extractives), previous work at State Forests NSW (SFNSW) has shown that with tight process control, poly-phenolic tannin glues cured with para-formaldehyde can adhere to Blackbutt veneers. Were Blackbutt suitable for durable structural laminates, value would be added to the forest resource. "High" and "Low" extractive veneers were identified using a scanning technique for colour variation and became a variable in the specimen sets.

After examining work at the US department of Agriculture [Scott et al., 1992]; [River and Okkonen, 1993] and at Monash University [Milner, 1996], the project adapted a cleavage fracture toughness method to evaluate the glue-lines in LVL (the current Australian Standard method of evaluating LVL glue-lines does not test fracture toughness of correctly orientated veneers). Those researchers used fracture mechanics to assess joints in the larger laminate structure of "GlueLam" timber beams - some modification of the technique was required to test 3 mm veneers in hardwood LVL. Cleavage was thought to best represent the stresses induced by dimensional change in the timber. A scanning process was evolved to quantify the proportion of wood failure on the separated glue-line faces.

Blackbutt LVL specimens were made up at UNSW on two scales: beam-sized elements (12 ply, 36 x 120mm x 3.6m) for testing mechanical properties and two-ply strips, 6 x 20 x 300mm for fracture testing of glue-lines. The strip specimens were tested either soon after manufacture or after varying periods of exposure to one of three accelerated-aging environments. After testing to determine the considerable mechanical properties of Blackbutt LVL, undamaged parts of the larger specimens were placed in an inter-tidal zone in Sydney Harbour. Some specimens were removed after fourteen months of this cyclic salt-water immersion for glue-bond evaluation using the fracture toughness method.

The results suggest that the most severe of the accelerated aging environments and the intertidal marine exposure have produced a discernable weakening of the LVL glue-lines however because the mean fracture toughness values remain relatively high (approximately 400 J/m^2) and the wood-fibre percentages are similar to those of unexposed specimens, the glue-lines can be said to have maintained their integrity.

3. METHOD

3.1. Extractive assessment:

Yazaki [1994] found a correlation between colour redness and the quantity of hydrolisable tannins known as "extractives" on the veneer surface. Both 250mm square surfaces of 148 veneer sheets were analysed.

A digital image of each surface was analysed using "Adobe Photo Shop". A mean redness number was obtained (approximately 135 for a low (L) extractive veneer , 140 for high (H) extractive).

3.2 Strip specimen manufacture for glue-line analysis:

Pairs of 250 mm square x 3.2 mm thick veneers were glued and pressed (tight side to loose side) using the Poly-Phenol Tannin (PFT) adhesive, "Bondtite 245" to form two-ply LVL. Adhesive was spread on both faces of adjacent veneers using a rubber roller spreader. The following gluing parameters were used:

Veneer moisture content :	10%
Glue mix (Bontite 245) :	100 parts resin, 36 parts hardener, 77.3 parts water,16 parts cure retardant.
Spread rate :	300 g/m^2 (per glue line)
Glue-line moisture :	22%
Open assembly time :	25 minutes
Closed assembly time:	10 minutes
Pre-press settings :	35 minutes @ 1200 kPa
Hot press settings :	35 minutes @ 1350 kPa and 138°C

High extractive veneers were glued only to high extractive veneers (H) - low extractive veneers (L) were similarly paired. Parallel-grain strip specimens, 20mm wide were cut from each sheet to allow for at least five replicate glue-line tests under each condition.

Veneers used to make the larger 12-ply structural specimens were assessed visually and laid up alternatively with high and low extractive. After exposure to the inter tidal zone, a band saw was used to cut two-ply strip specimens from the larger samples.

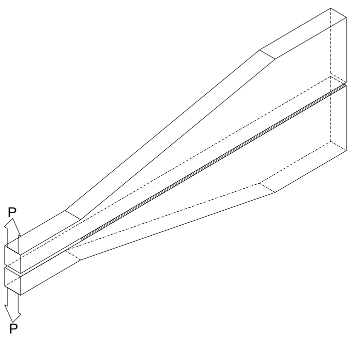

Figure 1. Contoured Double Cantilever Beam Test

3.3 Fracture toughness test procedure

To assess the Mode-1 (cleavage) fracture toughness of its glue-line, each strip specimen was progressively pulled apart in a contoured double cantilever beam apparatus (CDCB) after ASTM D3433-93 (Fig.1). Before testing, the strip specimens were conditioned for two days to 25°C and 60% relative humidity. The top and bottom surfaces of a veneer-pair strip specimen were each glued to an aluminium cantilever whose dimensions were contrived to maintain a constant separating effort in the fracturing glue-line as the cantilever pair was forced open. A saw-cut stiffness calibration after the procedure of [River and Okkonen, 1993] of the aluminium cantilever arms showed the correspondence relation, dC/da to be 0.00001916 (N^{-1}).

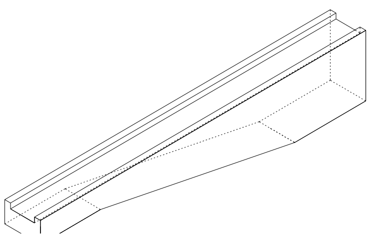

Figure 2. Recess-milled Edge

Because the relative strength of the Blackbutt LVL glue-line caused problems in attaining adhesion between LVL specimen and aluminium beam, the receiving edges of the aluminium CDCB were recess-milled to a depth of 2 mm (Fig.2) thus providing a lip for increased surface area and a shear component for the aluminium-wood bond - failure was directed to the timber glue-line. Strip specimens were bonded to the aluminium using either a strong epoxy resin or a ductile cyano-acrylate adhesive For lab-made strip specimens, failure within the glue-line was initiated by the inclusion of a tab of Teflon tape at the start of the glue-line. For strips obtained from larger specimens, a hacksaw cut initiated failure in the glue-line.

The ends of the cantilever were opened at a rate of 0.5 mm/minute (Photo 1) causing the glue-line failure to progress smoothly along the specimen - propagation and arrest points were not evident. The force required to open the cantilever pair was continuously sampled at a rate of 11hz. allowing a load-deflection curve to be plotted. The relationship between fracture toughness of the glue-line and load on the separating double cantilever is :

$$GI = P^2/2t * (dC/da)$$

where : GI = Mode I fracture toughness (J/m2)
P = Fracture load (N)
t = Width of strip specimen (m)
$dC/da = 0.00001916\ (N^{-1})$

After testing, separated faces of the glue-line were scanned and assessed using "Adobe Photo Shop" to quantify the proportion of wood-fibre failure. Fibres could be discerned on the faces of dry specimens but discolouration of some weathered specimens caused problems.

3.4 Fracture test data handling

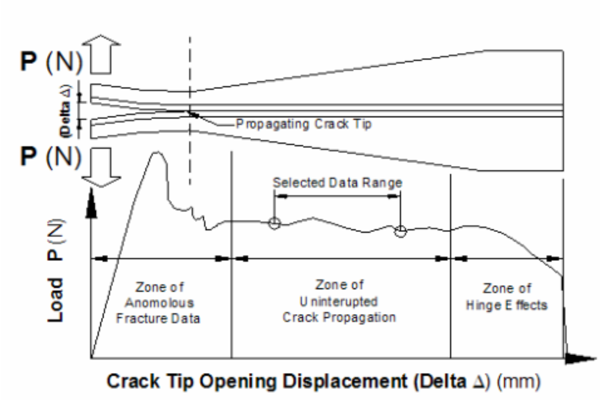

Figure 3. Crack tip and load deflection curve

As the cantilever pairs were separated, load increased to a peak value at initial fracture and dropped to a relatively constant value as the crack propagated along the glue-line (Fig 3). Towards the end of the test, a hinging action affects the load readings [Mostovoy et al 1971]. In this work, the area of interest is the stable crack propagation region of the load/deflection curve, after the initial peak and before the subsequent drop off. The staccato, propagation and arrest crack pattern seen in other Mode-1 fracture records was not observed in the Blackbutt LVL tests.

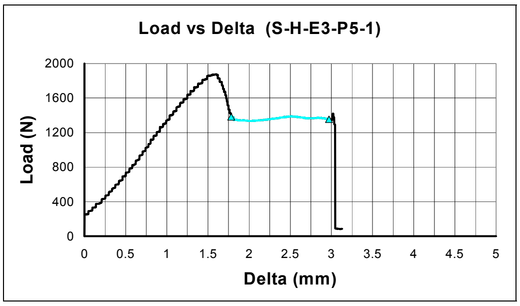

Figure 4. Shortest range load-deflection curve

To determine toughness from fracture load data, all load readings in a selected range of the anomaly-free zone were used (Fig.3). To enable statistical comparison, the selected range contained the same number of points for all tests examined in the series - it was selected to include a balance of higher and lower values reflecting the statistical qualities of the whole anomaly-free zone. Its size (793 points) was determined by the full length of the anomaly-free zone seen in the shortest test to be compared (Fig.4). Using the expression of 3.3, fracture toughness, GI was calculated for force readings over this range and presented as a box plot. The box plot sets out the median, spread and quartiles of the fracture toughness values in the selected range. Outliers are represented as small circles above or below the spread lines (eg. Fig.5).

3.5 Weathering environments

In addition to being tested at ambient conditions (dry, D), Blackbutt LVL strips were held in one of three environments [Milner et al, 1996] for one of four periods: 5, 10, 20, or 40 days before reconditioning for three days at 25°C and 60%RH, and testing. The environments were:

- Environment 1. (E1) Constant low temperature and high humidity (30°C, 90% RH)
- Environment 2. (E2) Constant high temperature and high humidity (100°C, 100% RH)
- Environment 3. (E3) High temperature constant, varying humidity (100°C, 20 - 100% RH)

The exposure conditions were achieved using controlled temperature and humidity chambers. For Environment 2, the specimens were left above boiling water in an autoclave. With increasing exposure to moist environments, the two-ply Blackbutt strips tended to change colour and warp out of plane however they were straightened during gluing to the aluminium cantilever arms.

Generally, five replicate strips of each variable were tested. The variables investigated in the Blackbutt LVL fracture toughness tests were :

- Extractive level as determined by redness scanning "High" or "Low" (H or L)
- Type of environment endured, "Dry" (D) or one of three environments (E1, E2 or E3)
- Period of exposure, (P1 – P4): 5 days, 10 days, 20 days and 40 days.

4. RESULTS AND OBSERVATIONS

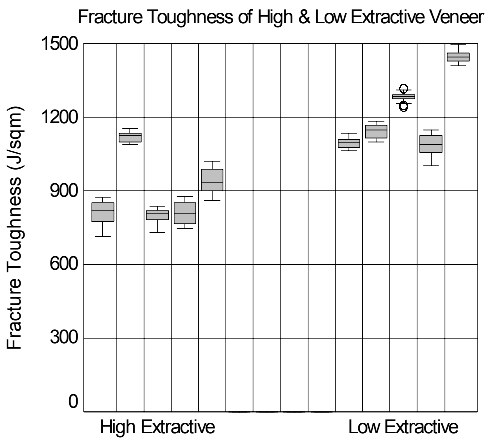

Figure 5. Box plots of fracture toughness for individual dry strip glue-lines

4.1. Unweathered (Dry) Specimens

The central tendency and spread of fracture toughness of each replicate of dry strip specimens are shown in the box plots of Figure 5. High and low extractive specimens have been grouped. Trends evident from the dry plots are:

- Fracture toughness values between 700 and 1500 j/m^2 are high for glue-lines in timber (cf. USA results of 80j/m^2 obtained for Urea Formaldehyde on Yellow Birch [Scott et al, 1992]) .
- Low extractive veneers showed tougher glue-line with slightly less variability. (Although variation within the high and low categories make these assessments problematic, the trend is expected and very clear for dry specimens)

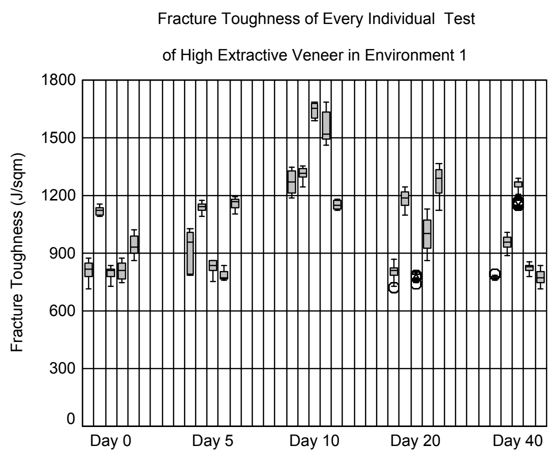

Figure 6. Box plots of fracture toughness for high extractive individual strip glue-lines exposed to Environment 1

4.2 Weathering Environment 1:

The first of the durability environments (constant 30°C and 95%RH) was relatively benign. Box plots for individual toughness tests of high extractive specimens after each exposure period are shown in Fig. 6. Fig. 7 shows the same results where the results for all five high extractive strip replicates have been combined for each time of exposure to E1 - Fig 8. shows similar record for low extractives. While low extractive veneer bonds show less variability, the plots do not indicate a clear trend towards decreased bond fracture toughness with increasing exposure to Environment 1.

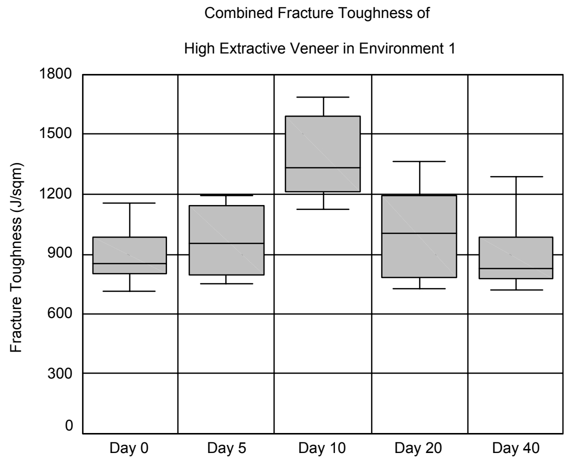

Figure 7. Replicate-combined box plots of fracture toughness for High extractive strips exposed to E1

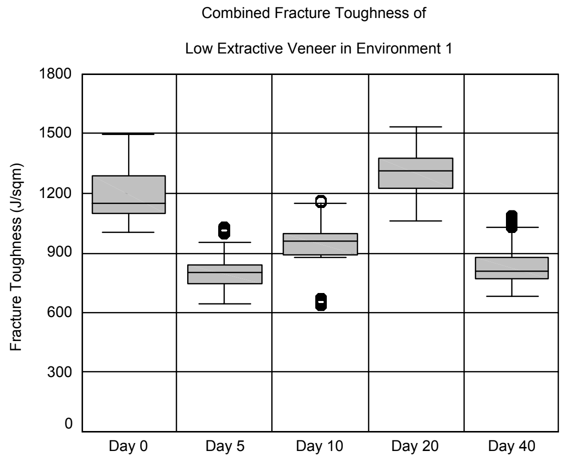

Figure 8. Replicate-combined box plots of fracture toughness for Low extractive strips exposed to E1

4.3 Weathering Environment 2

Environment, E2 (constant 100°C and 100% RH in an autoclave) represented very hostile conditions for glue-lines in timber. All specimens were distorted and discoloured on removal. Figures 9 and 10 show plots for replicate-combined fracture toughness tests of high and low extractive strips respectively. The toughness of glue lines in both high and low extractive specimens trended down with increased exposure to this environment. After exposure to E2, a greater variability can be discerned in the plots of fourth-spreads for high-extractive specimens.

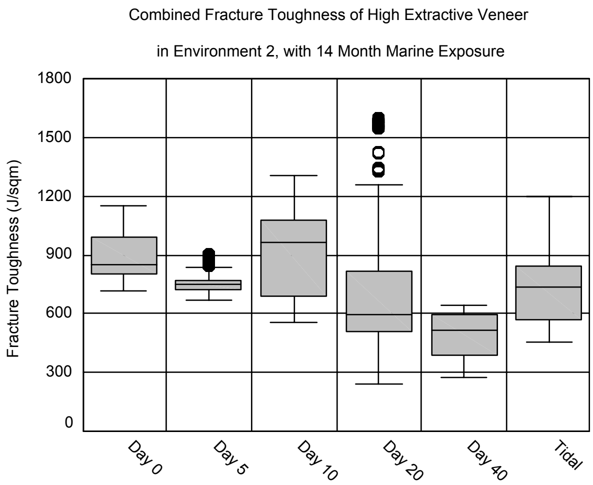

Figure 9. Replicate-combined box plots of toughness for High-extractive strips exposed to E2 or tidal sea-water

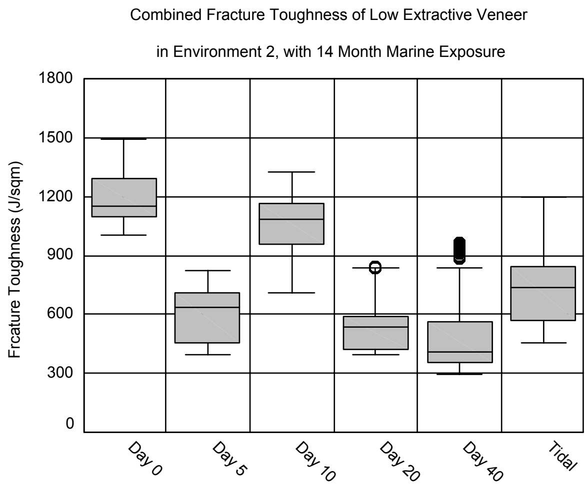

Figure 10. Replicate-combined box plots of toughness for Low-extractive strips exposed to E2 or tidal sea-water

4.4 Weathering Environment 3

Environment 3 (temperature constant, 100°C and humidity cycled between 30% & 90% RH every five days) was explored to investigate the effect of shrink/swell dimension changes. Figures 11 and 12 show plots for individual and replicate combined tests separated for high and low extractive. A slight trend down in the toughness of glue-lines can be discerned in most specimens with increasing exposure to this environment. No clear trend to greater variability is shown between high and low extractive specimens - both are more variable than dry (Day 0) or Environment 1.

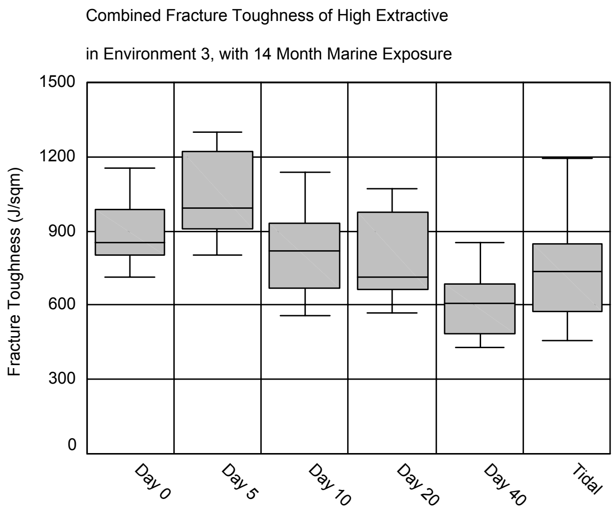

Figure 11. Replicate-combined box plots of toughness for High-extractive strips exposed to E3 or tidal sea-water

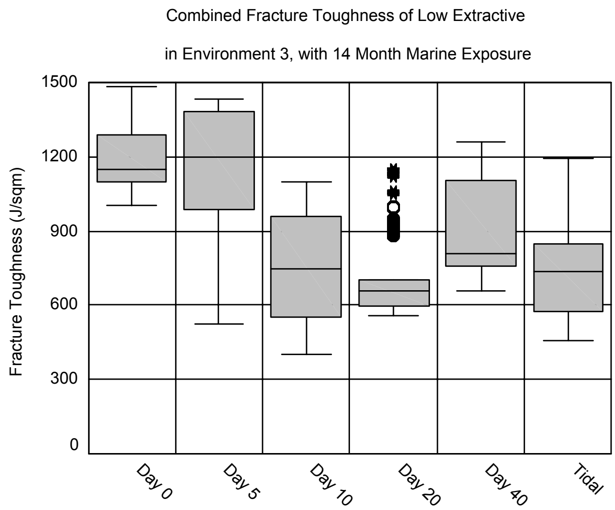

Figure 12. Replicate-combined box plots of toughness for Low-extractive strips exposed to E3 or tidal sea-water

4.5 Cyclic tidal immersion

The box plots of glue-line fracture toughness of two-ply strips taken from immersed Blackbutt LVL specimens are appended to the plots of Environments 2 and 3 shown in Figures 9 to 12 above. Because extractive levels were not identified in the veneers used to make the immersed specimens, the results are appended to both replicate combined sets. While the plots suggest that compared to unweathered specimens, immersion has diminished glue-line toughness, the value of mean toughness of approximately 750 J/m2 and the high level of wood fibre failure observed indicate a strong glue bond has been maintained after 14 months in the adverse climate of Sydney Harbour's inter-tidal zone.

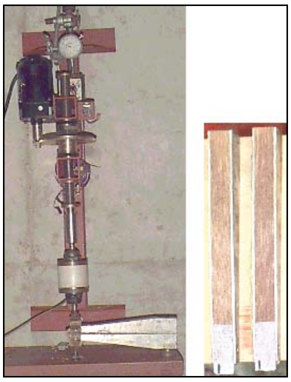

Photo 1 Strip test apparatus and tested glue-line faces

Photo 2 Blackbutt LVL after 14 months of inter- tidal immersion

Photo 3 Radiata Pine LVL after 14 months of inter- tidal immersion

5. DISCUSSION AND CONCLUSIONS

The fracture toughness results did not show a dramatic breakdown of any of the LVL glue-lines tested. Although after considerable exposure to very hostile environments, some downward trends were evident in glue-line toughness, those values remained relatively high and unlike pine, the natural durability of the timber prevented its breakdown (Photos 2 and 3). The lowest fracture toughness observed on the most degraded specimen was approximately 300 J/m^2, greater than the unweathered fracture toughness in yellow birch laminates reported by US researchers - these results suggests that a durable structural LVL can be made from Blackbutt.

Milner [1996] proposed a cyclic environment such as E3 to investigate the effect of shrink/swell dimension changes. The slenderness of 6mm thick two-ply strip specimens did not restrain one side of the glue-line sufficiently to build up shear and opening stresses to the extent that a trend could be established. The intertidal zone specimens, at 36mm thick provide more restraint and could have achieved more shrink/swell glue-line stresses with varying moisture content but the degree of drying under the wharf may not have been great over the low tide. Subsequent work needs to log moisture content each side of the relatively impermeable barrier set up by the glue line.

The relatively good durability shown by the UNSW Blackbutt LVL compared to plywood which we understand delaminated in service could be attributed to two factors:

- Parallel-grain LVL veneers may settle together more intimately under pressure than do the orthogonal grains of plywood.
- While the gluing parameters were similar, the lab specimens spread the PFT adhesive on both sides of the line and controlled the process very closely whereas at the plywood mill, one side of the line only was thickly coated. The glue was intended to be transferred to the other side in the stack.

ACKNOWLEDGEMENTS

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REFERENCES

Milner, H R and Ward, C: The Evaluation of Structural Adhesives, *26th Biennial Forest Products Conference*, Clayton, **2/3** (1996)

Mostovoy, S, Ripling, E J and Corten, H T: Fracture Mechanics: A Tool for Evaluating Structural Adhesives, *Journal of Adhesion,* **3**, 107 (1971)

River, B H and Okkonen, E A: Contoured Wood Double Cantilever Beam Specimen for Adhesive Joint Fracture Tests, *ASTM Journal of Testing and Evaluation,* **21**, 21 (1993)

Scott, C T, River, B H and Koutsky, J A: Fracture Testing Wood Adhesives with Composite Cantilever Beams, *ASTM Journal of Testing and Evaluation,* **20**, 259 (1992)

Yazaki, Y, Collins, P J, and McCombe, B "Variations in Hot Water Extractive Content and Density of Commercial Wood Veneers from Blackbutt" Holzforsprung V.48 1994 W de Gruyter, Berlin, 1994

ASTM D3433-93 "Standard Test Method for Fracture Strength in Cleavage of Adhesives in Bonded Metal Joints", American Society of Testing Materials

Water Entry at Penetrations in a Hardboard Siding-Clad Wood Stud Wall When Subjected to Simulated Driving Rain

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ABSTRACT

Driving rain is one of the significant agents of premature deterioration of wall assemblies. Driving rain deposited on the cladding may penetrate through imperfectly designed, defectively installed and inadequately maintained interface details at wall penetrations under the driving force of coincident wind pressure. Some examples of rainwater-induced damage include the decay of wood based materials, corrosion of metals and failure of finishes. Hence, assessing the degree to which driving rain may penetrate wall assemblies permits determining the risk of damage under given climate conditions and this in turn may provide the basis for estimating the useful life of the assemblies.

Previous experimental work undertaken at the Institute for Research in Construction on water entry focused on determining the quantity of water entry through deficiencies incorporated in the cladding of 17 different wood stud wall assemblies when exposed to simulated driving rain loads. The assemblies were comprised of four cladding types including stucco, brick veneer, hardboard and vinyl siding, and EIFS. This paper reports on a follow-up experimental work in which only the hardboard siding-clad wood stud wall was subjected to repeated water entry trials and incorporated additional levels of spray rate and pressure difference as compared to the previous work. Trials were also conducted to determine whether cascading or spraying water to simulate deposition of driving rain on the surface of the cladding affected the quantity of water entry into the wall assembly. This wall assembly included a drainage cavity and specific deficiencies such as a missing length of sealant at the interface between the cladding and the penetrating components, i.e. window, ventilation duct and electrical outlet. In a typical test, water was sprayed continuously on the wall specimen while subjecting it to a specified air pressure differential. Water entering the deficiencies was collected at troughs located in the drainage cavity behind the cladding and in the stud cavity directly below the penetrating component over a known period.

Experimental results on water entry of hardboard siding-clad wood stud wall provided information on water entry rates in troughs as functions of simulated driving rain loads. Results indicated that cascading water on the cladding surface resulted in higher rates of water entry into the wall assembly than spraying water on the cladding surface for the same nominal average rate per unit area of deposition. As well, significant quantities of water were collected in the drainage cavity behind the cladding as compared to that collected in the stud cavity over a given period. This further highlights the importance of a drainage cavity to reduce the likelihood of water entry to the stud cavity from the incidental water that enters deficiencies in the cladding.

KEYWORDS

Driving rain; Water penetration; Water entry; Performance testing; Wood stud walls

1 INTRODUCTION

Driving rain is one of the significant agents of premature deterioration of wall assemblies [Carll 2001]. Driving rain deposited on the surface of the cladding may enter through the imperfectly designed, defectively installed and inadequately maintained interface details at through wall penetrations under the driving force of coincident wind pressure. Interface details at wall-window, wall-electrical outlet and wall-ventilation duct are the primary points for rainwater ingress [Morrison Hershfield Limited 1996]. Some examples of rainwater-induced damage include decay of wood-based materials, corrosion of metals, degradation of thermal performance of insulations, and failure of finishes [RDH Building Engineering Limited 2001]. Hence, assessing the degree of water entry into wood stud wall assembly permits determining the risk of damage incurred by moisture sensitive components of the assembly under given climatic conditions and this in return may provide the basis for estimating the useful life of the assembly.

Laboratory testing of full-scale test specimens permits relating the likelihood of driving rain exposure to in-service conditions [Lacasse 2003] and hence provides insightful information on water penetration and entry performance of wall assemblies. Previously conducted experimental work [Ritchie and Plewes 1956; Ritchie and Plewes 1961; Roberts 1980; Newman and Whiteside 1981; Rathbone 1982] have provided substantial information on water penetration performance of various claddings under simulated driving rain loads, however, the quantity of water that may enter through the specific deficiencies at penetrations in the cladding, the path of water flow and the location of the water accumulation within the assembly in relation to simulated driving rain loads were not studied. Recently, experimental work [Lacasse et al. 2003] conducted for a study entitled MEWS (Moisture Management of Exterior Wall Systems), focused on determining the quantity of water entry through deficiencies incorporated in the cladding of 17 different wood stud assemblies when exposed to simulated driving rain loads, i.e. spray rates and pressure differences across the assembly. The assemblies were comprised of four cladding types including stucco, brick veneer, hardboard and vinyl siding, and EIFS.

This paper reports on a follow-up experimental work [Sahal and Lacasse 2004] in which only the hardboard siding clad wall specimen that included specific deficiencies was subjected to simulated driving rain loads in order to quantitatively assess the water entry of the assembly. In the MEWS study, the driving rain loads were simulated by different levels of spray rate and pressure difference consistent with driving rain intensities that may occur in extreme climatic events in North America. The subsequent work incorporated additional levels of spray rate and pressure difference as compared to previous work in order to simulate a broad range of possible service conditions that might prevail across North America. Tests were also conducted to determine whether cascading or spraying water to simulate deposition of driving rain on the surface of the cladding affected the quantity of water entry into the wall assembly.

The present experimental study followed a test protocol, previously described in Lacasse et al. [2003], that measured the air leakage, pressure response and water entry of the test specimen. The test protocol was initiated by characterizing the air leakage of the assembly in order to determine the effect of air leakage of the wall assembly to driving pressures across the assembly that may bring about water entry. Pressure response was measured to provide a measure of pressure differences across the wall assembly that in return established the driving pressures across the wall assembly. Quantifying the water entry through the specimen permitted evaluating the degree of water entry of the specimen and deriving the rates of entry in relation to simulated driving rain loads.

2 DESCRIPTION OF TEST SPECIMEN

The primary components of the cladding assembly included hardboard lap siding affixed to 19-mm vertical pressure-treated wood strapping that provided a drainage cavity between the siding and sheathing membrane. The remainder of the wall assembly consisted of two (2) layers of 30-min. building paper applied as sheathing membrane onto a 12-mm glass mat gypsum sheathing board. The sheathing board was installed on 38-mm by 89-mm wood studs. An acrylic sheet was installed onto the outer side of the wood studs to provide an air barrier system. The test specimen nominally measured 2.44-m by 2.44-m.

Typical North American wood-frame wall components such as a vinyl flanged window, a ventilation duct and an electrical outlet were incorporated in the wall assembly. Each of these components penetrated the siding (1st line of defence), sheathing membrane (2nd line of defence) and the sheathing board.

Deficiencies, which simulated either imperfect installation or inadequate maintenance in the field, were introduced in the 1st (siding outer surface) and 2nd (sheathing membrane outer surface) lines of defence in order to provide water entry points in the wall assembly. At the 1st line of defence, the deficiency introduced at the wall-electrical outlet interface was a 50-mm of missing sealant length between the top of the cover plate and the cladding. The wall-ventilation duct interface incorporated the same kind of deficiency as the wall-electrical outlet at the 1st line of defence. At the wall-window interface, a 90-mm length of sealant was missing at the bottom centre of the windowsill, along one edge of the sill and at the bottom of the adjacent jamb. At the 2nd line of defence, the wall-electrical outlet interface incorporated a 6-mm gap between the edge of the sheathing board and the sheathing membrane, and a 50-mm of missing sealant length between the sheathing board and the bottom of the duct. The wall-vent duct incorporated the same kind of deficiencies as the wall-vent duct at the second line of defence. At the wall-window interface, a 6-mm gap between the window flange and the sheathing membrane was incorporated to the assembly. As well, a reverse lap between the rough sill protection and the sheathing membrane was introduced on the sheathing board.

Water collection troughs were located in the 19-mm cavity and in the stud space, specifically in proximity to the specified deficiencies, to facilitate the collection of any water entering deficiencies and passing through either the cladding or sheathing board. Locations of each trough are illustrated in Fig. 1. Troughs D, V, and TW were located in the 19-mm cavity, beneath the respective penetrations, i.e. electrical outlet (D), vent duct (V) and window (TW). As well, troughs T1 to T5 were located at the base of the wall in the cavity behind the cladding. Troughs T1 to T5 collected water that might penetrate the cladding and be deposited in the drainage cavity. These troughs were limited nominally to the breath of a stud space. Additionally, three troughs were situated in the stud cavity beneath the electrical outlet (E), vent duct (L) and window (W).

Pressure taps were located in the 19-mm cavity between the siding and the sheathing board, and in the stud cavity to obtain the pressure differences that may affect the likelihood of water entry past the cladding and the sheathing board.

An acrylic board situated on the outer surface of the wood studs served as the air barrier system. A series of three 6-mm diameter holes were introduced in the acrylic board about the centre of each stud space. Opening or closing the appropriate number of holes in the air barrier system could regulate the nominal rate of air leakage through the assembly.

3 DESCRIPTION OF TEST PROTOCOL

The test protocol was established to characterize the air leakage, pressure response and water entry of the test specimen, a detailed description of which is provided in [Lacasse et al. 2003]. The air leakage characterization of the specimen was achieved by measuring the air leakage rates through the specimen at different levels of static pressure difference and various equivalent leakage areas.

Different levels of equivalent leakage area were obtained by opening or closing an appropriate number of holes in the air barrier system. The required value of less than 0.6 L/s-m^2, and the maximum value of air leakage rate were selected at a pressure difference of 75 Pa.

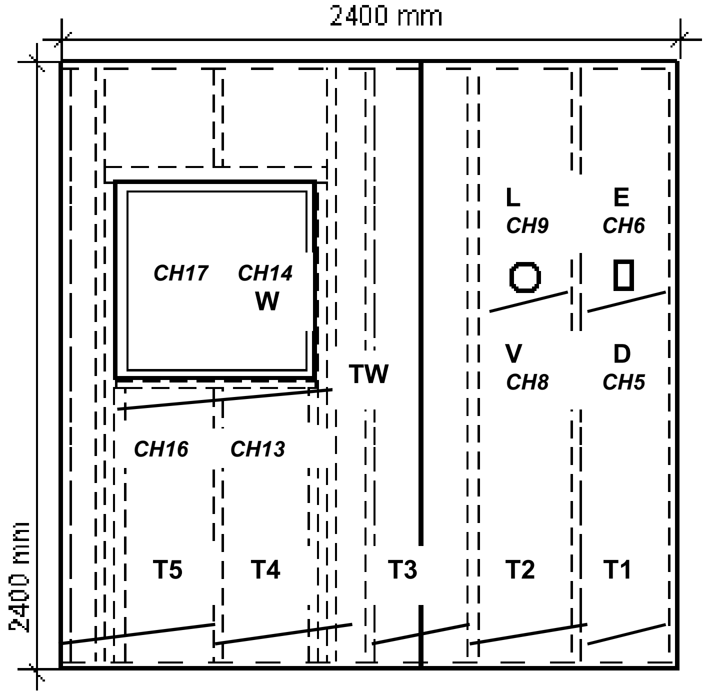


Figure 1. Location of water collection troughs and pressure taps [Lacasse et al. 2003]

At each degree of air barrier system leakage (ABSL), differential pressure transducers were used to measure the pressure drop in both the 19-mm drainage cavity and stud cavity, over a period of 10 minutes, at static pressure differences of 40, 75, 150, 300 and 600 Pa. The driving pressure ratios across the width of the wall at the electrical outlet, the ventilation duct and the window were determined. The driving pressure ratio is the ratio of driving pressure, i.e. pressure drop, across the cavity in relation to the static pressure difference across the specimen. The higher the value, the greater the potential driving pressure across the cavity.

The water entry characteristics of the specimen were determined by subjecting the specimen to varying spray rates and static pressure differences at each degree of ABSL. A spray rate of 3.4 L/min-m^2 was applied to the exterior wall surface through a series of nozzles perpendicular to the surface of the cladding at increasing pressure steps of 0, 40, 75, 150, 300 and 600 Pa. Spray rates of 1.0, 2.0, 3.0 and 5.0 L/min-m^2 were cascaded on the cladding surface from a cascade bar located at the top end of the cladding for the same increasing pressure steps. The intent was to determine the effect of simulated deposition of driving rain on the surface of the cladding to water entry rates. Water entry through the specified deficiencies to the stud cavity was observed. Water entering past the cladding and the sheathing board was collected in troughs over the given time period. Water entry rates were determined at each trough servicing the deficiencies at the electrical outlet, the ventilation duct and the window, in relation to spray rates and static pressure differences.

4 EXPERIMENTAL RESULTS

4.1 Air leakage characterization

An ABSL of 0.3 L/s-m^2 was achieved when all the holes in the air barrier system were closed. Twenty-one holes of the same diameter - three openings in each stud cavity - represented an equivalent leakage area of 508-mm^2 and provided a nominal ABSL of 0.6 L/s-m^2 at a pressure difference of 75 Pa.

4.2 Pressure response characterization

Pressure taps of interest across the width of the wall at the electrical outlet were CH5 and CH6, which were located in proximity to water collection troughs D and E, as given in Fig. 1. When the assembly provided an ABSL of 0.3 L/s-m^2, the driving pressure ratio within the drainage cavity and stud cavity ranged from 0.0 to 0.1 for the given pressure range. Similar results were obtained when the ABSL was 0.6 L/s-m^2. The results indicated that the acrylic sheet provided the greatest resistance to air leakage throughout the test.

The pressure tap locations considered across the width of the wall at the vent duct were CH8 and CH9, which were located at the drainage and stud cavity, respectively. The driving pressure ratio in both cavities and each pressure step ranged from 0.0 to 0.1 for both ABSL levels. The results implied that pressure moderation occurred in the cavities, which in turn decreased the driving pressures across the cladding.

The pressure tap locations considered across the width of the wall at the window were CH13, CH14, CH16 and CH17. Pressure taps CH13 and CH16 were located immediately beneath the wall-window interface deficiency at the right and left drainage cavities when facing the inside of the wall, respectively. Pressure taps CH14 and CH17 were situated at the right and left stud cavities when facing the inside of the wall, respectively. When the ABSL was 0.3 L/s-m^2, the driving pressure ratio at the drainage cavity (CH13 and CH16) for each pressure step ranged from 0.0 to 0.1. The driving pressure ratio at the stud cavity (CH14 and CH16) ranged from 0.1 to 0.2. However, when the specimen was evaluated with an ABSL of 0.6 L/s-m^2, the driving pressure ratio at the drainage cavity increased, i.e. the ratio varied between 0.4 to 0.5 for the given range of pressures. The driving pressure ratio at the stud cavity varied between 0.85 to 0.98, when the ABSL was 0.6 L/s-m^2. The results indicated that when the specimen ABSL increases, the proportion of pressure available to drive water through the openings likewise increases. This further suggest that there is greater likelihood of water entry through the specific deficiency at the given assembly having an ABSL of 0.6 L/s-m^2 as compared to 0.3 L/s-m^2.

4.3 Water entry assessment

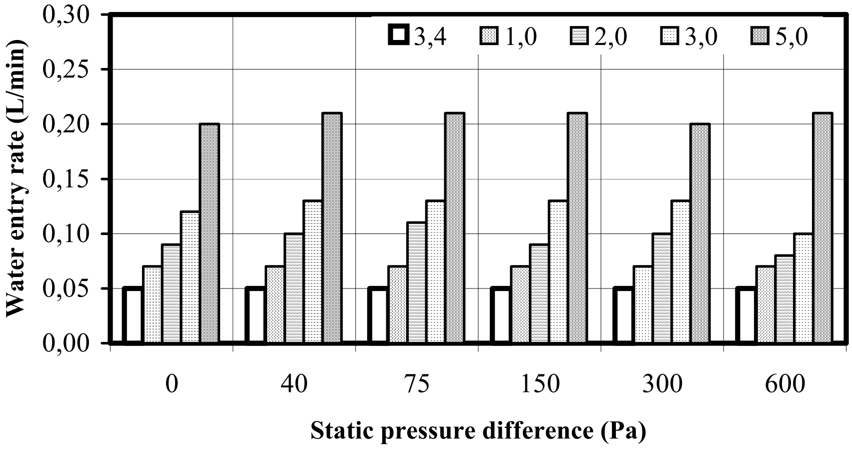
When water was sprayed on the cladding at specified rates and pressure differences across the specimen, water entering the deficiency above the electrical outlet was collected at trough D, and some instances at trough E. No water was collected at trough T1. In other words, under the given simulated driving rain loads, water that passed the cladding through the specific deficiency was mostly drained into the drainage cavity. It should be noted that water ingress to the stud cavity was only observed in a few instances.

Water entry rates in trough D at a given pressure level as a function of spray rates and ABSL is given in Figs. 2a and b. A comparison of the range of values of entry rates in relation to changes in spray rates at a given pressure step to the range of values of entry rates in relation to changes in pressure level at a given spray rate suggest that water entry rates are primarily dependent on the spray rate and not the pressure difference across the assembly.

Water entering the deficiency above the ventilation duct was collected at troughs V and T2, and in some instances at trough L. Small amounts of water were collected intermittently at trough V.

Water entry rates in trough T2 as a function of spray rate and given pressure level when the assembly provided an ABSL of 3.0 L/s-m^2 is given in Fig. 3a. Increases in spray rate tended to slightly increase the water entry rate in this trough at each given pressure level. However, the increase in entry rates was evident for spray rates of 3.0 and 5.0 L/min-m^2 at pressure difference of 600 Pa.

(a)



(b)

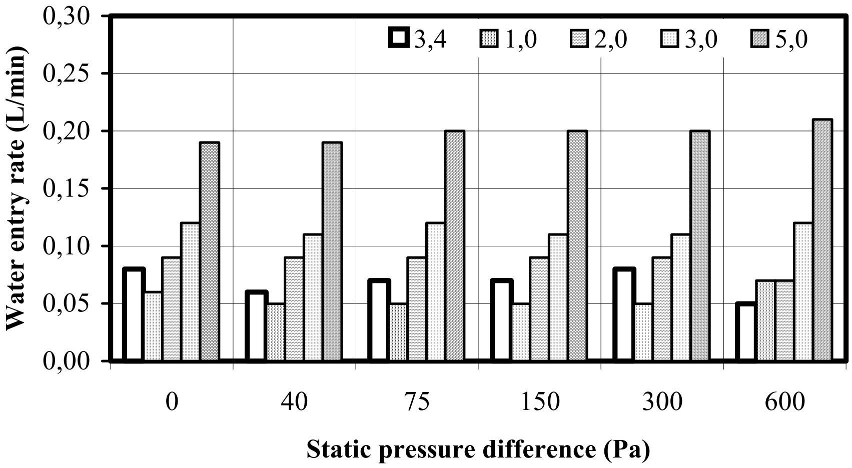
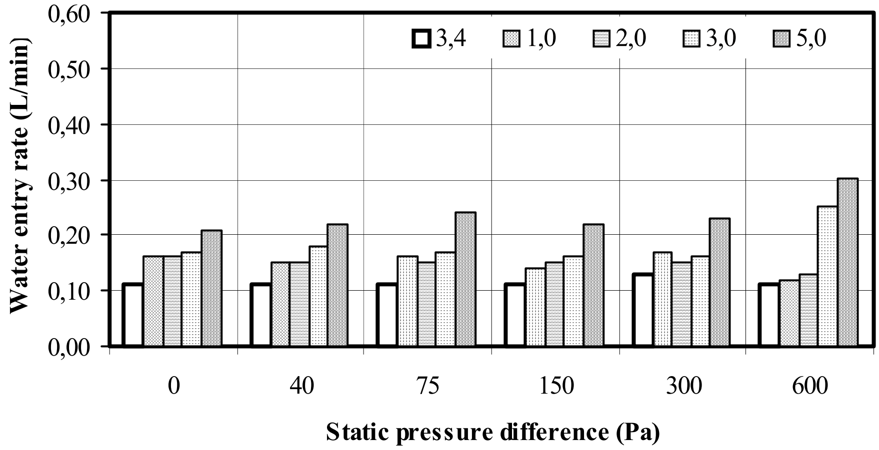


Figure 2. Water entry rates through the deficiency above the electrical outlet collected at trough D, for test conditions (a) ABSL 0.3 L/s m^2, (b) ABSL 0.6 L/s m^2

Water entry rates for the ABSL of 0.6 L/s-m^2 is provided in Fig. 3b. The same pattern occurred at the given ABSL. As well, water entry rates at spray rates 3.0 and 0.5 L/min-m^2 and static pressure difference of 600 Pa increased when ABSL increased.

Water deposited on the cladding found its way through the deficiency beneath the window-wall interface and it was collected at troughs TW and T5. No water was collected at trough W. Fig. 4a illustrates the entry rates as a function of pressure differences across the specimen at given levels of spray rate when the assembly provided an ABSL of 3.0 L/s-m^2. At the spray rates of 3.4, 1.0, 2.0 and 3.0 L/min-m^2, the pressure applied on the cladding was increased at steps from 40 Pa to 300 Pa and the water entry rate tended to slightly increase at each pressure step. However, at the given spray rates of 3.4, 1.0, 2.0 and 3.0 L/min-m^2, water entry rates increased significantly at the pressure difference of 600 Pa. Increases in rates of water entry in relation to corresponding increases in pressure difference across the assembly were evident at the spray rate of 5.0 L/min-m^2.

(a)



(b)

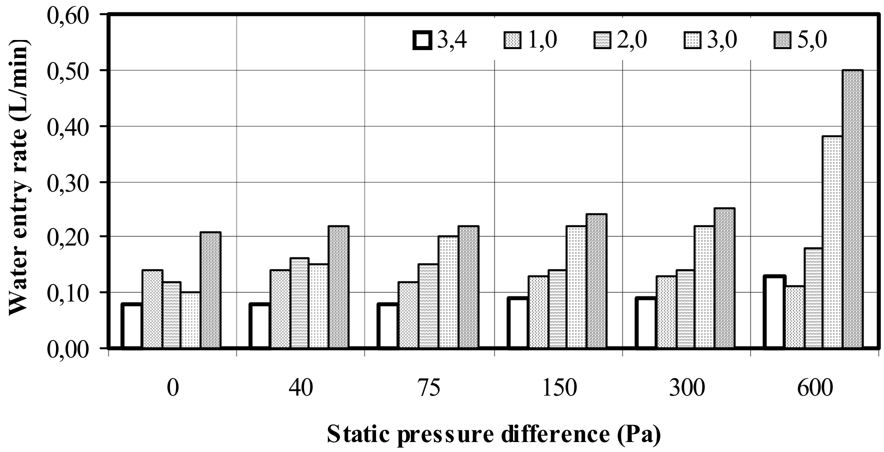


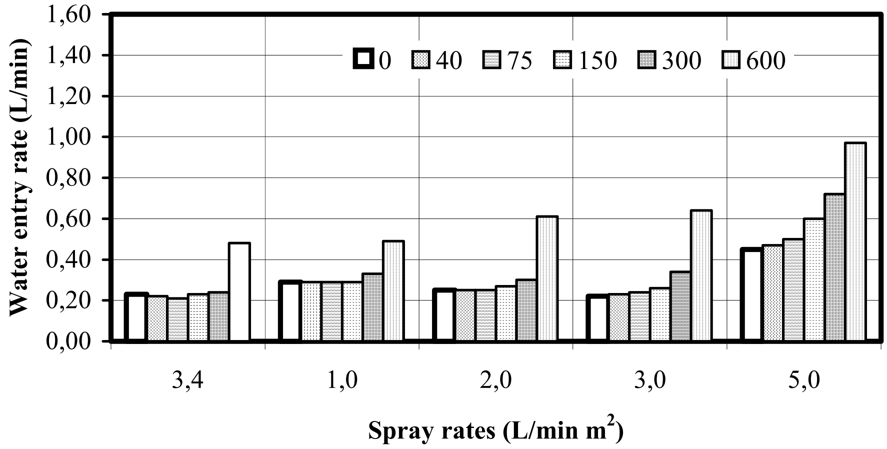
Figure 3. Water entry rates through the deficiency above the ventilation duct that was collected at trough T2, (a) ABSL 0.3 L/s m^2, (b) ABSL 0.6 L/s m^2

Fig. 4b demonstrates that the same pattern had occurred when the system provided an ABSL of 0.6 L/s-m^2. At the spray rate of 5.0 L/min-m^2, water entry rates ranged from 0.44 to 1.5 L/min with corresponding pressure differences of 0 Pa to 600 Pa. Hence, increase in the air barrier system leakage resulted in an increase in water entry rates.

In all water entry tests, water entry through the deficiency at the electrical outlet, vent duct and window occurred even when there was no pressure difference across the specimen, i.e. when there was no driving pressure across the cladding. Hence, water leakage through the deficiency was attributed to gravity.

Figs. 2 to 4 demonstrate that cascading water rates on the cladding surface resulted in higher rates of water entry as compared to uniformly spraying water on the cladding surface. For example, in Fig. 2a, the rate of water entry obtained was 0.05 L/min, when a rate of 3.4 L/min-m^2 was sprayed on the surface of the cladding and the pressure difference across the wall assembly was maintained at 40 Pa. At the same pressure step, water entry rates obtained were 0.07, 0.10, 0.13, and 0.21 L/min for corresponding rates of 1.0, 2.0, 3.0 and 5 L/min-m^2 cascaded on the surface of the cladding.

(a)



(b)

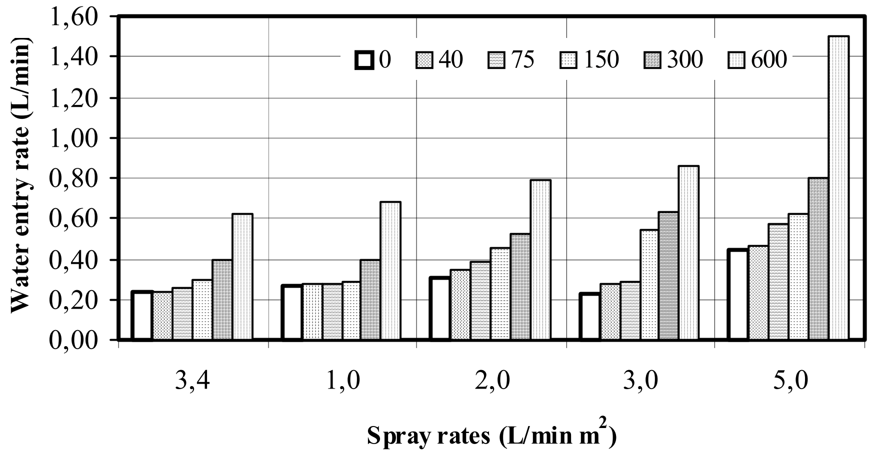


Figure 4. Water entry rates through the deficiency above the ventilation duct collected at trough TW and T5, for test conditions: (a) ABSL 0.3 L/s m^2, (b) ABSL 0.6 L/s m^2

5 CONCLUSIONS

The results of the tests indicated that water entry rates at the electrical outlet were primarily dependent on the amount of water deposited on the cladding. Increased rates of entry were evident for corresponding increases in rates of water sprayed onto the cladding surface.

Rates of entry for the specified deficiency at the ventilation duct at spray rates of 3.4, 1.0, and 2.0 L/min-m^2 were loosely dependent on the amount of water deposited on the wall. However, water entry rates at spray rates of 3.0 and 5.0 L/min-m^2 were dependent on pressure differences, particularly at 600 Pa.

Values for water entry rate at the window were more than those obtained from the electrical outlet or ventilation duct. Water entry through the deficiency at the window was dependent on pressure differences across the assembly; entry rates increased with increasing pressure levels.

Substantial amounts of water entered into deficiencies under gravity alone, i.e. when there was no pressure difference across the cladding. Increases in the entry rate were dependent on the increases in spray rate.
The simulation of water deposition, i.e. spraying water directly onto the surface of the cladding through arrays of nozzles or cascading the same rate of water from the top of the cladding surface had

a direct effect on the water entry rates. Cascading water at rates of 1.0, 2.0, 3.0 and 5.0 L/min-m^2 on the cladding surface resulted in substantial increases in rates of water entry as compared to those obtained when spraying water at a rate of 3.4 L/min-m^2 on the cladding.

As might be expected, significant more water was collected in the drainage cavity behind the cladding as compared to that in the stud cavity over a given period. This further highlights the importance of a drainage cavity as a useful means of reducing the likelihood of water entry to the stud cavity from incidental water entry through deficiencies in the cladding.

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REFERENCES

Carll, C., 2001, “Rainwater intrusion in light-frame building walls”, 2nd Annual Conference on Durability and Disaster Mitigation in Wood-Frame Housing, Forest Products Society, Madison, WI, USA, pp. 33-40.

Lacasse, M.A. (2003), "Durability and performance of building envelopes," BSI 2003 Proceedings (15 Cities across Canada, 10/7/2003), pp. 1-6, October 01, 2003 (Aussi disponible en français: Durabilité et performance des enveloppes de bâtiment) (NRCC-46888)

Lacasse, M. A., O’Connor, T. J., Nunes, S. and Beaulieu, P., 2003, *Report from Task 6 of MEWS Project Experimental Assessment of Water Penetration and Entry into Wood-frame Wall Specimens, Final Report,* Research Report 133, Institute for Research in Construction, National Research Council Canada, 133, (IRC-RR-133).

Morrison Hershfield Limited, 1996, *Survey of Building Envelope Failures in the Coastal Climate of British Columbia*, Canada Mortgage and Housing Corporation.

Newman, A. J. and Whiteside, D., 1981, “Water and air penetration through brick walls-A theoretical and experimental study”, Transactions Journal of the British Ceramics Society, 80: 17-26.

Rathbone, A. J., 1982, *Rain and Air Penetration Performance of Concrete Blockwork*, Technical Report 553, Cement and Concrete Association.

RDH Building Engineering Limited, 2001, *Study of High-rise Envelope Performance in the Coastal Climate of British Columbia,* Canada Mortgage and Housing Corporation.

Ritchie, T. and Plewes, W. G, 1956, *Preliminary Tests of the Rain Resistance of Brickwork Panels,* Internal Report, No. 108, Division of Building Research, National Research Council Canada, (DBR-RP-108).

Ritchie, T. and Plewes, W. G, 1961, *Moisture Penetration of Brick Masonry Panels*, Research paper no. 118, Division of Building Research, National Research Council Canada, (DBR-RP-118).

Roberts, J. J., 1980, “Rain penetration problems with concrete blockwork”, *Chemistry and Industry*; 194-197.

Sahal, A. N. and Lacasse M. A., 2004, *Water Penetration of Claddings-Overview of Physical Parameters, Related Test Standards and Test and Research Facilities*, Internal Report, Institute for Research in Construction, National Research Council Canada; 863, (IRC-IR-863).

Sahal, A. N. and Lacasse M. A., 2004, *Experimental assessment of water penetration and entry into siding-clad wall specimen*, Internal Report, Institute for Research in Construction, National Research Council Canada, 862, (IRC-IR-862).

Evaluation of Durability of PET Fibers Under Diverse Aggressive Environments

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ABSTRACT

The addition of fibers in mortars and concretes improves the performance of these materials when submitted to mechanical efforts, specially tensile and flexural stresses. However, in order to guarantee the durability of the mortar or concrete the fibers should be stable in the cement matrix. The use of [poly(ethylene terephthalate)] or PET fiber reinforced cement-based materials is an effective contribution for environment preservation, since the PET has a long decomposition time – it takes more than 100 years to completely degrade. This research evaluated the degradation of PET fibers, obtained from the recycling of plastic bottles, exposed to aggressive solutions for five months, through Fourier-transform infrared spectroscopy (FTIR) and scanning electron microscopy (SEM). These analyses have shown that PET fibers suffer degradation in sulfuric acid solution, $Ca(OH)_2$ and also in Lawrence's solution, that simulates the pore water of mortars and concretes. The degradation of the fibers was also evaluated through the performance of fiber-reinforced mortar specimens under flexure. The test results have shown evidence of degradation of the fibers after few months inside the mortars, and was confirmed by SEM analysis.

KEYWORDS

PET fibers, degradation, cement-based material, fiber reinforcement.

1 INTRODUCTION

The hydrated cement matrix of mortars and concretes is brittle and shows low toughness, but the incorporation of fibers to can modify such properties. Due to such ability, fiber reinforced mortars [FRM] and concretes [FRC] have been studied for many years, specially after the banning of asbesto fibers.

Many synthetic fibers are currently used to produce FRMs and FRCs, such as polypropylene [PP], polyethylene [PE], nylon, aramid and polyesthers. The poly(ethylene terephthalate) fibers [PET] belong to the polyesther group, and can be obtained from the recycling of PET bottles, which take more than 100 years to completely degrade.

Some requirements are expected to be attended by the fibers, such as chemical stability in the cement matrix, compatibility of the mechanical behaviour with the exposition of the composite, easy dispersibility of the fibers during mixing and appropriate geometric configuration of the fibers [Wang *et al.,* 2000].

However, the durability of PET fiber in alkaline environments is controversial. Some researchers postulate that PET fibers suffer degradation in hydrated cement matrixes, leading to the reduction of their reinforcing capacity [Houget [1992]; Wang, Backer, [1987]; Jelidi [1991]. On the other hand, Jakel cited by Wang *et al.* [1987] and the ACI-544 Committee [1998] reported good performance of PET fiber reinforced mortars and concretes.

The main purpose of this research is to investigate the durability of recycled PET fibers embedded in Portland cement-based materials and aggressive solutions. The environmental aspect of the recycled PET fibers supports the purpose of this research, since PET has a long decomposition time, and its use for the production of concrete and mortars will help the environment preservation. To achieve this goal, the behavior of PET fiber-reinforced, 104 days-old mortar specimens was evaluated through the evaluation of the toughness indexes under flexural load. Scanning electron microscopy allowed the authors to visualize the degradation of the fibers. Moreover, chemical studies of the fibers through Fourier-transform infrared spectroscopy were also performed after exposition to aggressive solutions in order to support the tests with the mortars.

2. EXPERIMENTAL PROCEDURE

The PET fibers used in this research were obtained from the recycling of PET bottles in a rope industry. The fibers were added to the mortars as monofilaments. Their dimensions are approximately 26µm diameter and 20mm length [aspect ratio = 769]. Figure 1 shows a SEM image of a fiber. Table 1 presents the characteristics of the PET fibers investigated in this research.

Physical properties determined by DSC[a]	Melting temperature (°C)	252.8
	Crystallization temperature (°C)	95.0
Mechanical properties determined in accordance with ASTM D 3822-96.	Tensile strength (MPa)	323.5
	Total elongation (%)	70.7
	Yield stress σ_Y (MPa)	196.4
	Elongation at σ_Y (%)	7.18
	Elasticity modulus (MPa)	41.8
	Toughness (MPa)	17279.0

Table 1 Characteristics of PET fibers

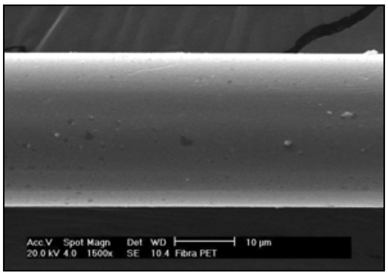


Figure 1 Aspect of the recycled PET fiber observed in SEM (magnification is 1500x)

In order to identify the degradation agents, the fibers were exposed to alkaline and acidic solutions, such as $Ca(OH)_2$ saturated solution [pH = 12.3], 0.1 M NaOH [pH = 13], and 0.1, 1 and 10 M H_2SO_4. The same experimental procedure was adopted by Segre et al. [1998] to study the durability of polypropylene fibers to be used in cement-based materials. The PET fibers were also exposed to the Lawrence solution, which simulates the pore water of cement-based materials. It was prepared with deionized water according the following recipe used by Jelidi [1991]: 0.48 g/l $Ca(OH)_2$, 3.45 g/l KOH, 0.88 g/l NaOH [pH= 12,9]. The fibers were kept immersed in the alkaline and acidic solutions for 5 months under 5, 25 and 50°C. Fibers immersed in deionized water for the same time and at the same temperatures were taken as reference.

A Fourier-transform infrared spectrometer Perkin-Elmer 16PC was used to analyse the fibers after the exposition to the solutions. To enable the analysis in pellets, the fibers were finely cut and mixed to KBr. The spectra were traced in the range 4000-400 cm^{-1} [wave number], and the band intensities were expressed in transmittance [%T]. The microscopy was performed in a Philips SEM XL-30 microscope equipped with an energy-dispersive X-ray analyzer [EDXA]. In order to obtain a conductive surface for the imaging, a thin gold layer was deposited on the fibers surface.

For the preparation of the mortar speciments, Portland cement (Brazilian cement CPII-F 32) and medium-grade quartz sand [2.4 mm maximum diameter] were used. The mixing proportions were 1:3 [cement:sand, in weight basis], the water/cement ratio was kept constant at 0.61 and the fiber content was 0,8% in volume. Deionized water was used to prepare the mortars. The cement CPII F is an ordinary Portland cement with up to 15% of finely ground limestone, in accordance to the Brazilian standard NBR 11578/91.

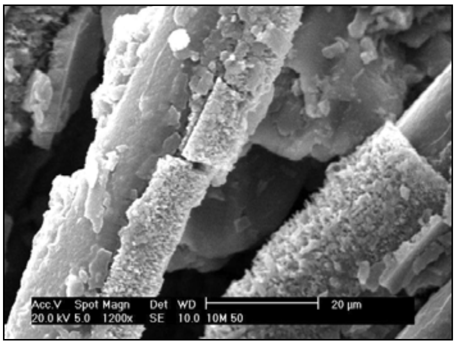
Prismatic mortar specimens [20 x 40 x 160mm] were moulded and kept in lime water for 14 days before the exposition to the laboratory environment. After 28 and 90 days at the lab, i.e., when the specimens were 42 and 104 days-old, they were tested according to the standard ISO/DIS 679 [2002]. The deflection was registered with transducers attached to the specimens and connected to a computer, allowing the plot of stress x strain curves for the whole analyses. Toughness indexes were the calculated according ASTM C 1018/94 [1994] [i.e., I_5, I_{10} and I_{20} indexes, which are the numbers obtained by the division of the integrated area up to a deflection of, respectively, 3.0, 5.5 and 10.5 times the first-crack deflection, by the integrated area up to the first crack].

The fracture surface of the fiber-reinforced mortar was analyzed by scanning electron microscopy [SEM]after the flexural test , in order to detect any evidence of degradation of the fibers inside the cement matrix. The fragments were vacuum-dried and kept over silica-gel until the sputtering process and analysis at the SEM.

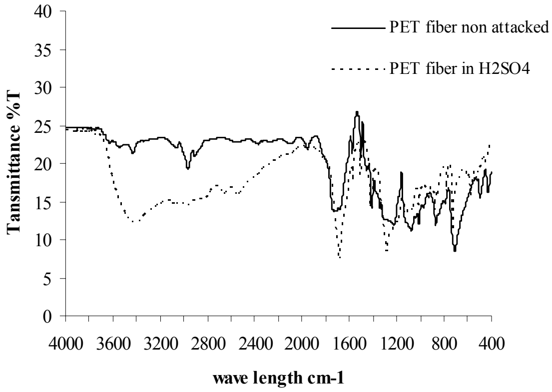
3 RESULTS AND DISCUSSION

3.1 Analysis of the fibers immersed in aggressive solutions

The FTIR spectra of the fibers immersed for 150 days in the $Ca(OH)_2$ saturated solution, Lawrence solution, 0.1M NaOH, and 1 and 0.1 M H_2SO_4 have shown no evidence of chemical modification when compared to the non attacked fibers, regardless the storage temperature. Due to the intensity of the attack, changes in the FTIR spectra of the fibers could only be detected when they were exposed to the 10M H_2SO_4 solution at 50ºC. SEM images of the fibers confirmed the degradation of the fibers. Figure 2 shows the FTIR spectra of non attacked and sulfuric acid exposed fibers.



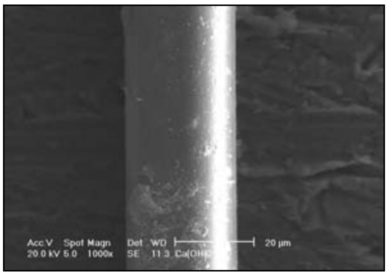
(a)



(b)

Figure 2 SEM image and FTIR spectrum of a fiber exposed to 10M H_2SO_4 solution at 50ºC for 150 days. FTIR spectrum of a non attacked fiber is shown for comparison.

Although no changes were observed in the FTIR spectra of the fibers exposed to the alkaline solutions in comparison with the non attacked fibers, SEM analyses have shown some roughness on the fibers surfaces exposed to $Ca(OH)_2$ and Lawrence solutions at at 50ºC for 150 days., as shown in Figure 3.



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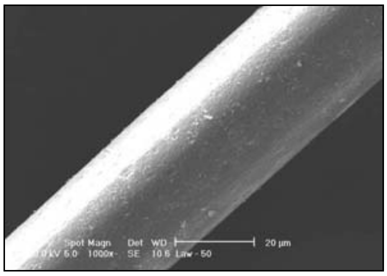
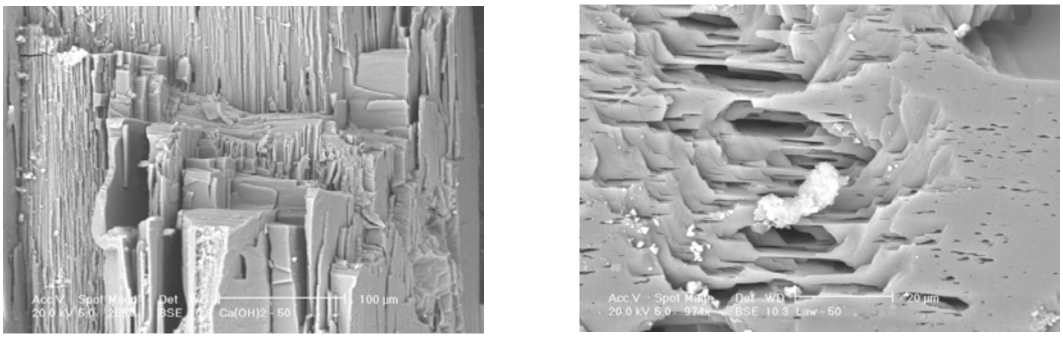


(b)

Figure 3 Micrographs showing the aspect of PET fibers after 150 days of exposition to (a) $Ca(OH)_2$ and (b) Lawrence solution at 50ºC

Moreover, the precipitation of some particles was observed at the bottom of the vessels used for the immersion of the fibers in the $Ca(OH)_2$ and Lawrence solutions [Figure 4]. The infrared spectra of such precipitated particles are shown in Figure 5. It can be seen that bands from PET are present at 1930, 1508, 1018, 870 and 696 cm^{-1}, which are assigned to the aromatic ring of the polymer. Besides such bands, new bands at 3462, 3286, 2228, 1554, 1434, 1386, 834, 738 and 606 cm^{-1} appeared, indicating some interactions between PET and the alkaline solutions.



(a) $Ca(OH)_2$ (b) Lawrence solution

Figure 4 Micrograph of the precipitated particles after the immersion of the fibers in (a) $Ca(OH)_2$ and (b) Lawrence solutions for 150 days at 50°C

The spectra shown in Figure 5 are typical Ca, Na or K terephthalate spectra. According to Jelidi [1991], the mechanism of chemical degradation of PET fibers consists in a depolymerization reaction that breaks the polymer chain. The ions Ca^{2+}, Na^{+}, K^{+} and OH^{-} attack the C-O bonds of PET and split the polymer in two groups: the group of the aromatical ring and the group of aliphatic ester. Alkali-ions and hydroxyls fix themselves to those groups, respectively. The produtcs of such interaction are Ca, Na and/or K-terephthalates and ethylene glycol.

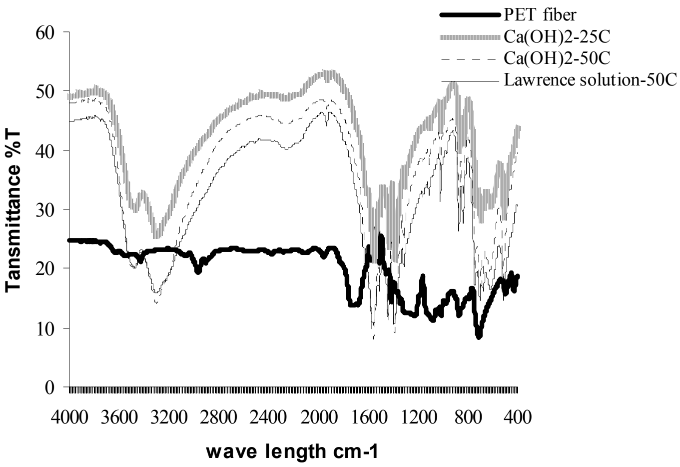


Figure 5 Infrared spectra of particles precipitated in $Ca(OH)_2$ solution (25 and 50°C) and in Lawrence solution (50°C), after exposition of PET fibers for 150 days

No evidences of degradation of PET fibers kept for 150 days in NaOH solution could be detected, regardless the temperature of the environment.

3.2 Results obtained from the tests in mortar specimens

The analysis of variance of the area under the *stress x strain* curves was carried out in order to evaluate the effect of the fibers on the toughness of the mortars. Therefore, the effect of the fibers on the toughness up to the first crack and on I_5, I_{10} and I_{20} indexes could be evaluated for the fiber-reinforced mortar specimens. The toughness indexes could not be measured for the non-reinforced mortars because they failed in a fragile way as soon as the first crack appeared.

Figure 6 illustrates the effect of age on the toughness indexes. For a conventional mortar, an increase of the toughness would be expected due to the higher hydration degree of the cement with elapsed time. Contrarily, the toughness of the PET-fiber reinforced mortars decreases with time, as can be seen. This overall effect is attributed to the degradation of the fibers inside the mortars.

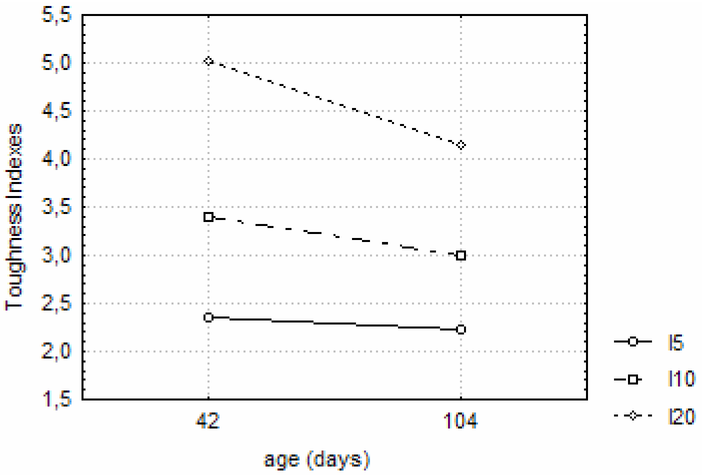
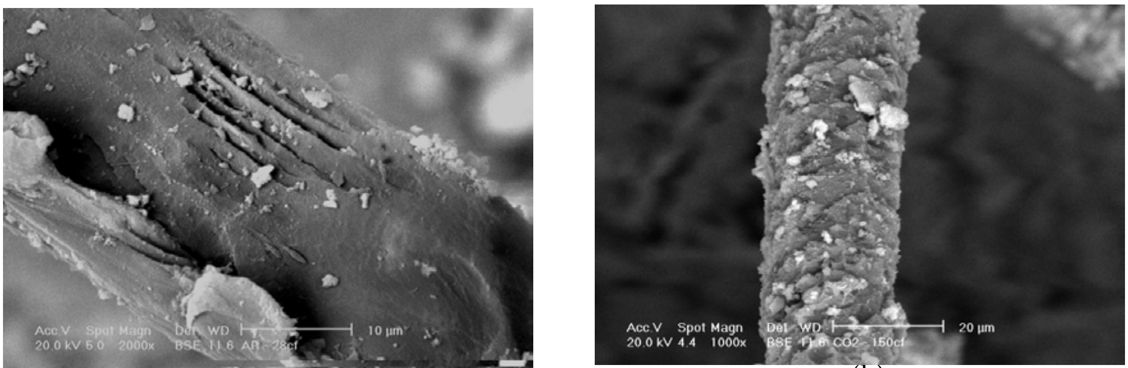


Figure 6 Effect of age on toughness indexes of PET fiber reinforced mortars

The results shown herein are in accordance with other researches who carried out accelerated tests of durability in alkaline environment with some types of fibers and concluded that the toughness of polyester fibers decreases with time [Balaguru et Slattum, 1998]. Pelisser [2002] worked with different contents of PET fibers in concrete and obtained the highest toughness for a 35 days-old concrete, compared to 150 days-old concrete specimens.

Figure 7 shows micrographs of the fibers inside the mortars after 42 and 104 days. As can be seen, the surfaces of the fibers appear quite rough, indicating that the fibers suffered some attack from the cement paste matrix. Figure 7(a) shows a detail where the detachment of superficial layers of the fiber can be seen as soon as 42 days after mortars preparation.



(a) (b)

Figure 7 Aspect of the PET fiber in (a) 42 and (b) 104 days-old mortar specimens.

4 CONCLUSIONS

From the infrared and microscopic analysis performed on recycled PET it is possible to conclude that the fibers interact with $Ca(OH)_2$ and Lawrence solutions. Their surface becomes rough and there is the precipitation of phases identified as alkaline terephthalates. In sulfuric acid solutions, PET fibers suffer some attack which becomes more intense under higher acid concentrations and storage temperatures.

The toughness indexes I_5, I_{10} and I_{20} decreased with time due to the degradation of PET fibers by alkaline hydrolysis when embedded in the cement matrix. Fourier-transform infrared spectroscopy and SEM of phases precipitated in alkaline solutions supported the conclusions. SEM micrographs allowed the observation of the degradation of PET fibers inside 42 and 104 days-old mortars specimens.

Considering the evidences of degradation of the PET fibers in mortar specimens shown in this research, the use of PET fibers in cement-based materials should be made carefully in order to avoid lack of reinforcement in short term. The authors recommend the use of the fibers when a higher toughness is important at early ages, such as in precast concrete and mortar elements, and when cracks caused by plastic shrinkage can be intense.

5 ACKNOWLEDGEMENTS

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6 REFERENCES

American Concrete Institute. State-of-the-art report on fibre reinforced concrete, ACI 544.IR- 96, In: *Manual of concrete practice.* Farmington Hills, 1998. CD-ROM.

American Society for Testing and Materials. **C1018**. Standard Test Method of Flexural Toughness and First-Crack Strength of Fiber-Reinforced Concrete (Using Beam with Third-Point Loading). 1994.

Associação Brasileira De Normas Técnicas. **NBR 7215**. Cimento Portland- Determinação da resistência à compressão. Rio de Janeiro, 1996.

NBR 13276. Argamassa para assentamento e revestimento de paredes e tetos - Preparo da mistura e determinação do índice de consistência. Rio de Janeiro, 1995.

ISO/DIS 679. Methods of testing cement - Determination of strength, 2002.

Balaguru P.; Slattum, K. Test Methods for Durability of Polymeric Fibers in Concrete and UV Light Exposure. Testing of Fiber Reinforced Concrete, American Conrete Institute, Detroit, Michigan, Ed. D. J. Stevens, *et al*. 1995, p. 115-136.

Houget, V. *Etude dês caracteristiques mecaniques et physico-chimiques de composites ciments-fibres organiques.* 1992. 237f. Tese (Doutorado em Ciências dos Materiais)- Devant L'Institut National des Sciences Appliquees de Lyon, França.

Jelidi, A. *Conception d'un materiau composite a matrice cimentaire reinforcee par des fibres de polyester.* 1991. 170f.. Tese (Doutorado em Ciências dos Materiais)- Devant L'Institut National des Sciences Appliquees de Lyon, França.

Pelisser, F. *Avaliação do desempenho de concreto reforçado com fibras de poli-etileno tereftalato. 2002. Dissertação* (Mestrado em Construção Civil)- Universidade Federal de Santa Catarina, Florianópolis.

Segre, N.; Tonella, E.; Joekes, I. 1998, Evaluation of the stability of polypropylene fibers in environments aggressive to cement-based materials. *Cement and Concrete Research*. **28** [1],75-81.

Wang, Y., Backer S.; Li V. C. 1987, An experimental study of synthetic fibre reinforced cementitious composites. *Journal of Materials Science*. 22, 4281-4291.

Wang, Y.; Wu, H.C.; Li, V. C. 2000, Concrete Reinforcement with Recycled Fibers. *Journal of Materials in Civil Engineering.* 314-319.

Comparative Evaluation Between Accelerated and Outdoor Ageing of Brazilian Paints - Part one

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TT2-69: Assessment of environmental effects on materials and products

ABSTRACT

In the study, eight acrylic latex paints, produced by two major Brazilian architectural paint manufacturers, were collected from the consumer market of the City of São Paulo. Traditional paint samples were collected, from each manufacturer, with different gloss finish (semi-gloss, medium-gloss and low-gloss) and also paint samples with elastomeric properties. The resin, pigment and nonvolatile content of the paints were estimated by gravimetric methods. The ageing tests were carried on "free" paint films and on mortar specimens treated with the paints. The outdoor exposure testing was conducted according to ASTM G-7 at three sites, in areas subject to urban, marine and rural environments and the accelerated testing was conducted according to ASTM G 154 in an Atlas UV/ condensation equipment, applying cycles of wetting and drying exposure and light source from fluorescent UVB-313 lamps. The evaluations of the changes in appearance, of exposed and unexposed specimens, were measured by color and gloss instrumental methods. The results obtained in accelerated weathering test and one-year at outdoor exposure site were compared; two additional years of exposure are planned.

KEYWORDS

Paint, acrylic latex paint, weathering, accelerated exposure, outdoor exposure.

1 INTRODUCTION

Coatings are used to protect substrates and to provide an aesthetic appearance. The change in the color of a coating applied in facades is one of the most serious problems in building appearance. The major factors involved in change of color of external walls are the adhesion of micro particles such as dirt, soiling, microorganisms or chemical reaction caused by weathering factors such as heating, solar radiation, acid rain, etc [Kitsutaka & Kamimura 1993]. A typical principle to avoid the change in the appearance of facades is to select material with has higher resistance against the effects of the outdoor environment. Natural and accelerated weathering are extremely important tests for determining the ageing characteristics of the materials. The rate and extension of deterioration depend on the material and the severity of the exposure conditions. These tests offer an initial view of the expected performance of the coating to the environment. The experimental variables involved in the test design deriving from inherent environmental variables and those introduced by the experiment [Jacques, Masters & Lewry 1996].
The natural weathering tests are not as simple as just only setting specimens in the environment and watching what happens. A number of important factors must be carefully considered to conduct the test. The method employed must be designed to be as similar as possible to the intended end use of the material in order to provide a more realistic exposure. To conduct the test a similar exposure condition should be created; besides, it is important to test at many different sites, each with its own particular set of influencing factors [Hicks & Crewdson 1995].
It is usually considered that accelerated weathering has a great advantage because the results are more quickly available than in natural weathering, which is a phenomenon that may take years and which many times does not have reproducibility. It is important to recognize that these data should be used only for comparative purposes. A correlation with natural weathering has not been developed because data obtained through these tests reproduce weathering by subjecting the samples to unnatural conditions. The failure mode would be valid for the tested material under a specific condition. Real field condition usually differs from those found in accelerated exposure [Sherbondy, 1995]. The main objective of this paper is the comparative evaluation between results of laboratory accelerated exposure and outdoor exposure of surface coatings. The paper presents the results obtained in a one-year exposure but two additional years of exposure are planned.

2. THE EXPERIMENT

2.1 Paint samples

Eight architectural acrylic latex paints, produced by two major manufacturers, were collected from the consumer market of the City of Sao Paulo. For each manufacturer, three conventional products and one with elastomeric characteristics were selected. Their chemical characteristics were also determined.

2.2 Specimens preparation

The tests were carried out on films and treated mortar. Films were obtained by applying the liquid paint on polyethylene films with an applicator with nominal wet thickness of 600µm. After drying time (7d) the paint film is separated from the polyethylene film.
For better simulation of in-service condition, mortar specimens (10cmX 10cm X 3 cm) were prepared with portland cement, lime and sand (mix proportion 1:2:9, by volume) and cured for 28d. Two coats of the paints, listed in Table 2, were applied according to manufacturers' instructions with a nylon brush on one face of the specimens and stored at 25 ± 2 ^{o}C and 50 ± 5 % RH for two weeks prior to exposure.

2.3 Weathering Tests

The exposure in natural weathering was done on unpainted mortar, used as reference, and painted mortar aiming to create a more realistic exposure and also on free films to allow gloss and color determinations using instrumental measurements, which demands a flat and homogeneous surface. The exposure in accelerated weathering was conducted only with free films because the size of specimens is not compatible with the exposure area.

2.3.1 Natural weathering test

The exposure in outdoor environment was done according to ASTM G 7-97 "Standard Practice for Atmospheric Environmental Exposure Testing of Nonmetallic Materials", simultaneously, in three different weathering stations in the State of Sao Paulo. The geographic data of the stations are:

a) Urban atmosphere: Sao Paulo site is located on the campus of University of Sao Paulo, Department of Civil Construction Engineering, at latitude 21°57'02''S and longitude 47°27'50''W;
b) Rural atmosphere: Pirassununga site is located on the campus of Pirassununga, University of Sao Paulo, Faculdade de Medicina Veterinária e Zootecnia, latitude 22°59'46''S and longitude 47°25'33''W;
c) Marine atmosphere: Ubatuba site is located on the Marine Base of the Oceanographic Institute of University of Sao Paulo, at latitude 23°30'S and longitude 45°06'W.

The monthly weather summary (rainfall, solar radiation and UV radiation) data at the Sao Paulo site is presented in Fig. 1. Pirassununga and Ubatuba sites are very near the Sao Paulo site; Table 1 presents the temperature and rainfall summary of the three sites.

Site	Period	Temperature (°C)			Rainfall (mm)
		Average	Max. Temp. Average	Min. Temp. Average	
São Paulo	2003-2004	21,3	25,8	16,8	113,4
Pirassununga	2003-2004	22,4	33,4	11,3	167,7
Ubatuba	Historical data	24,3	35,4	13,2	169,9

Table 1: Climate summary data at the sites

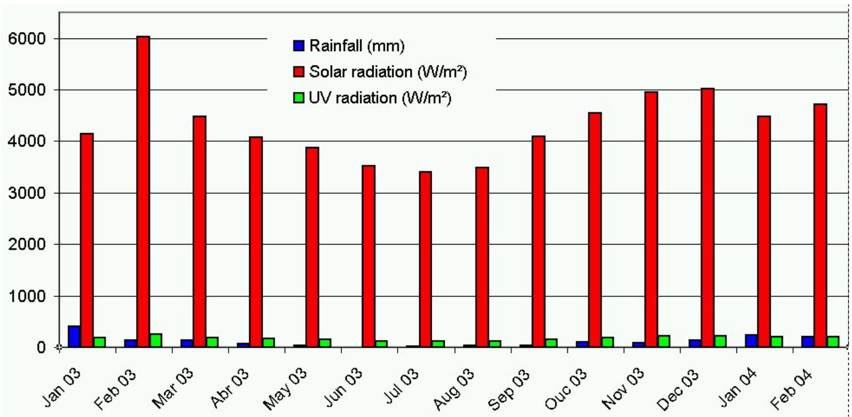

Figure 1: Weather summary (rainfall, solar and UV rad), Sao Paulo site, Jan. 2003-Feb. 2004

The painted and unpainted specimens and films were exposed at an angle of 45 degrees to the horizontal, facing North and 0,45m above the ground. The 45-degree exposure is the most common angle for coatings and it is used when the end-use position is unknown or variable; the orientation selected is the most severe. The ground at Pirassununga and Ubatuba sites is grass and in São Paulo the site is placed on a rooftop so the ground is concrete The appearance of the specimens was periodically evaluated every six months by visual and instrumental techniques, considering gloss and color. In this paper, only the one-year exposure evaluation is shown.

2.3.2 Accelerated weathering test
The exposure was made only with free films according to ASTM G-154-00 “Operating Fluorescent Light for UV Exposure of Nonmetallic Materials”, cycle 3, lamp UVB-313 with typical irradiance of 0,44W/m2/nm (approximate wavelength 310nm) and using the following exposure condition:

- 8h UV at 70 (±3) ^{o}C Black Panel Temperature;
- 4h Condensation at 50 (±3) ^{o}C Black Panel Temperature.

3. RESULTS AND DISCUSSION

3.1 Basic composition of the paint samples

The paints were characterized according to ASTM 3723-84 “Standard Test Method for Pigment Content of Water-Emulsion Paints by Low-Temperature Ashing”. The nonvolatile content was determined by drying the sample at 105^{o}C and the pigment content, by drying at 450^{o}C. Table 2 presents the chemical characteristics of the paints.
The resin content was estimated through the difference between nonvolatile content and pigment content.
The resins of the paints was separated from the pigments by extraction with chloroform and then identified by Nicolet Fourier transform infrared (FT-IR) spectrophotometer technique (Protegé 460) based upon computerized library comparison searches (Hummel). The identification showed that the paints A1, A2, A3, A4, B1, B2 and B3 were copolymers of poli-acrylate-styrene and B4 was poli-acrylate. According to the results of the chemical characterization, the two sets of products are equivalent and the products are constituted by copolymer of poli-acrylate-styrene except for one product (B4) which is a pure acrylic resin.

Manufacturers	Samples/appearance	Determinations			
		Volatile (%)	Nonvolatile (%)	Resin (%)	Pigments (%)
A	A1 (Low-gloss)	50,2	49,8	13,8	36,0
	A2 (Medium-gloss)	58,0	42,0	17,3	24,8
	A3 (Semi-gloss)	52,4	47,6	22,5	25,0
	A4 (Low-gloss 100%)	41,3	58,7	24,2	34,3
B	B1 (Low-gloss)	53,6	46,4	13,0	33,3
	B2 (Medium-gloss)	52,2	47,8	18,0	29,8
	B3 (Semi-gloss)	51,3	48,7	20,6	28,1
	B4 (Semi-gloss 800%)	51,8	48,2	24,1	24,1

Table 2: Chemical characterization

3.2 Visual inspection carried out on painted mortars specimens

3.2.1 Sao Paulo weathering site (urban atmosphere)
After a six-month exposure, during the inspection only a slight color change was observed, with different intensities, probably caused by the deposition of dirt and soiling. On unpainted mortars deposition of dirt and soiling were also observed. Presence of microorganisms was not observed on all the specimens. After a one-year exposure, the presence of dirt and soiling was intensified on the unpainted specimens and those painted with B4; but on specimens painted with A1, A2, A3, A4, B1, B2 and B3 color change was maintained the same intensity.

3.2.2 Pirassununga weathering site (rural atmosphere)
After a six-month exposure, during the inspection only a slight color change was observed on specimens treated with paints A1, A2, A3, B1, B2 and B3 paints, with different intensities, probably caused by the deposition of dirt and soiling on surfaces with differents rugosities. Presence of microorganisms was not observed on any specimens. Specimens painted with B4 show a pronounced color change caused by the deposition of red clay present in an area near the site. After a one-year exposure color change was intensified on all exposed specimens. Figure 2 shows the exposure rack with the specimens after a six-month exposure. The pronounced color change on specimens treated with paint B4 is very evident.

3.2.3 Ubatuba weathering site (marine atmosphere)
After a six-month exposure, during the inspection no visible color change was observed on specimens treated with paints A2, A3, A4, B2, B3 and B4. Presence of green microorganisms possibly organisms like terrestrial algae was observed on unpainted specimens and those treated with paints A1 and B1. After a one-year exposure, the presence of green, brown and black organisms was intensified. According to the chemical characterization, A1 and B1 are high *PVC* (Pigment Volume Content) paints. High *PVC* formulations result in films with high porosity so they do not protect the porous substrate against water penetration. The high humidity of the marine site created an environment especially favorable for the development of these organisms. Figure 3 shows the appearance of the specimens after a one-year exposure.

Figure 2: Appearance of the specimens after a six-month exposure in rural atmosphere. Specimens painted with B4 show an pronounced color change caused by the deposition of red clay in an area near the site.

Figure 3: Appearance of the specimens after a one-year exposure at marine atmosphere. The presence of green, brown and black organisms was observed.

3.3 Instrumental measurements made on films

All the measurements were carried out on films. The instruments used for both determinations require flat surfaces; the measurements in mortar specimens need accessories with a very small aperture because the texture of the specimens affects the results.

3.3.1 Gloss measurements
The gloss of coatings varies greatly with the angle of incidence. For architectural coating, the gloss is measured with geometry of 60° for semi gloss finishes and 85° for low gloss finishes. In the study, the measurements were made according to ASTM D 523-89 “Test Method for Specular Gloss” on free films, before exposure, after a one-year exposure in outdoor environment and after 300h and 600h of accelerated exposure. The measurements were made with a Byk Gardner micro-TRI-gloss. Figure 4 shows the gloss evaluation of coatings obtained with low-gloss paints A1, A4 and B1, after a one-year exposure at natural weathering site and 300h and 600h in accelerated weathering test. Figure 5 shows the gloss evaluation of coatings obtained with medium and semi-gloss paints A2, A3, B2, B3 and B4,

after a one-year exposure at natural weathering site and 300h and 600h in accelerated weathering test. The results of gloss evaluation, presented in Figs. 4 and 5, show that the natural weathering and accelerated tests in this study did not present any correlation; the effect observed on the specimens after both weathering tests was not the same.

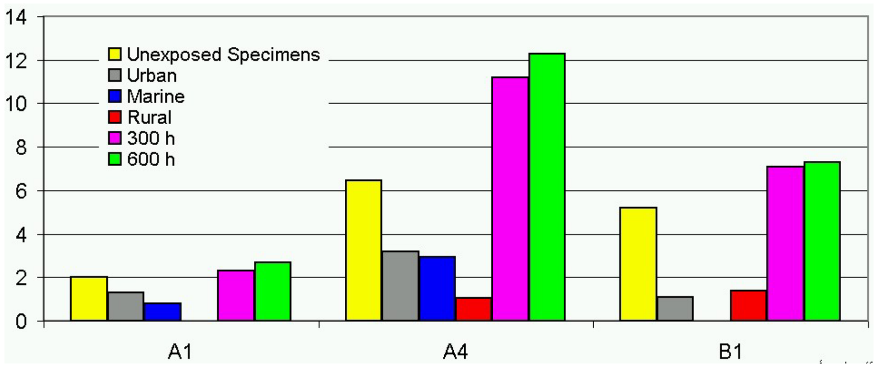

Figure 4: Gloss evaluation 85° of coatings obtained with low-gloss paints (A1, A4, B1), after a one-year exposure at outdoor ageing and 300h and 600h in accelerated ageing.

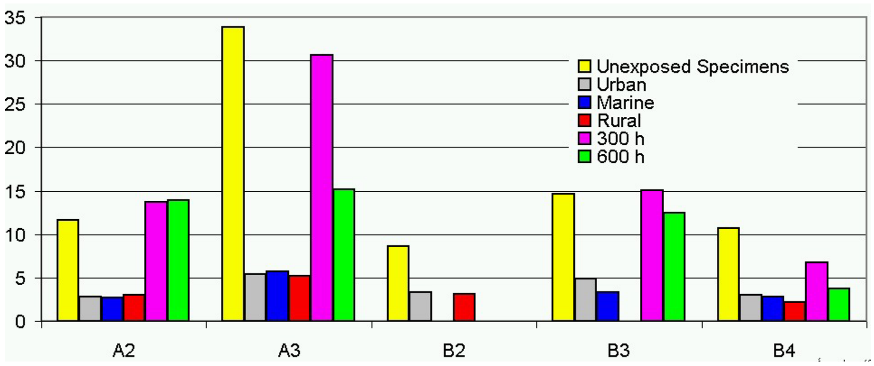

Figure 5: Gloss evaluation 60° of coatings obtained with medium-gloss paints (A2, B2) and semi-gloss paints (A3, B3, B4), after a one-year exposure at outdoor ageing and 300h and 600h in accelerated ageing.

According to Figs. 4 and 5, after a one-year exposure at outdoor weathering site all the coatings presented a change (loss) of gloss. Coating obtained by low-gloss paint A1 showed an insignificant loss of gloss; by medium-gloss paints A2 and B2, a higher loss of gloss; and by semi-gloss paints A3, B3 and B4, a significant loss of gloss. The results showed that coatings with higher gloss value present higher change of gloss after a one-year exposure at outdoor weathering sites; probably the change is caused by the deposition of micro particles such as dirt, soiling etc. and also by the removal of the pigments present on the surface of the coatings during this weathering. The measurements were made on unwashed coatings because maintenance services of building facades by washing is not usual in Brazil.

Figures 4 and 5 showed that coatings obtained by low-gloss paints A1, A4 and B1 and by medium-gloss paints, after 300h and 600h in accelerated weathering test, showed a gain of gloss or an insignificant change of gloss. Probably the factors present during this weathering accelerated the curing process and did not cause deposition of micro particles or removal of pigments.

3.3.2 Color differences measurements

The difference in color between the coatings before and after exposures was measured using a Byk-Gardner color-guide (45/0 geometry). The measurements were made on free films, according to ASTM D 2244/89 “Calculation of color differences from instrumentally measured color coordinates”, before exposure, after a one-year exposure in outdoor environment and after 300h and 600h of accelerated exposure. Figure 6 shows color difference after natural weathering at sites and after 300h and 600h of accelerated weathering is shown. Figure 7 shows the color change of paint A2 (medium gloss appearance) and the color change of paint B4 (semi- gloss appearance).

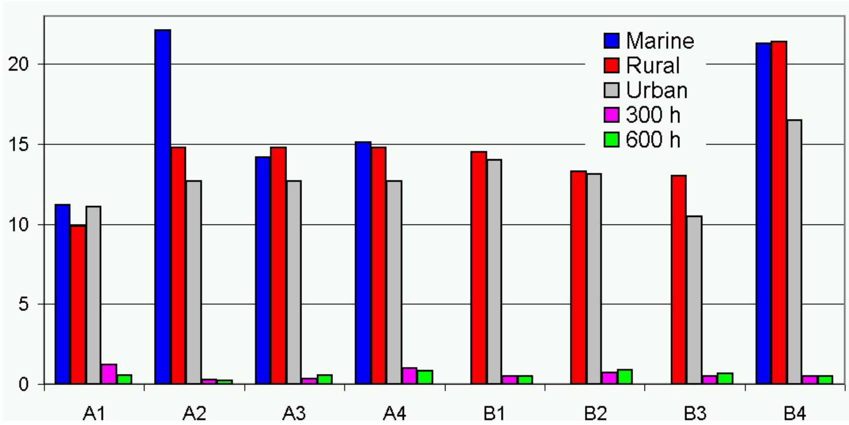

Figure 6: Color change of coatings obtained with paints A1, A2, A3, A4, B1, B2, B3 and B4 after a one-year exposure at outdoor ageing and after 300h and 600h of exposure in accelerated ageing

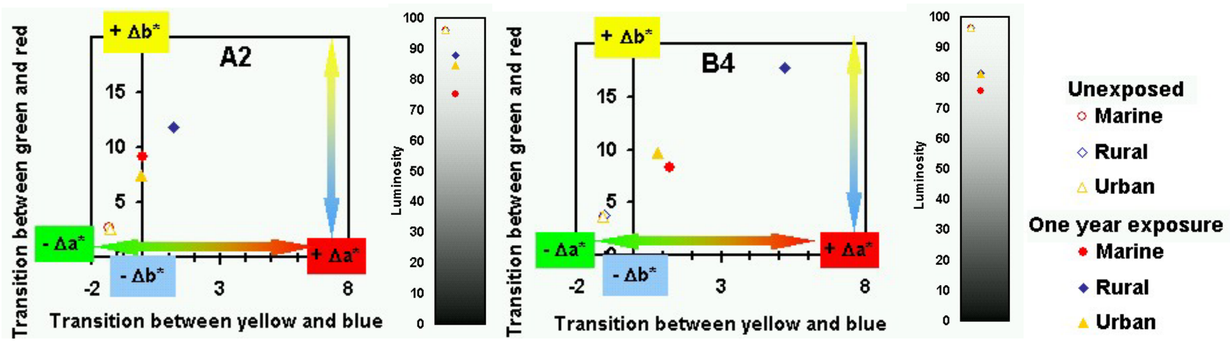

Figure 7: Paints A2 and B4 showed higher color change, the first probably caused by biological organisms and the second by the impregnation of red clay.

The results of color difference presented in Fig. 7 show that the natural weathering and accelerated test in this study, similar to the results for gloss differences, did not present any correlation, either; the effect observed on the specimens after the two types of weathering was not the same.
According to Fig. 7, all the specimens treated with paints showed a very high change in color after the natural weathering exposure and not such a high color difference after the accelerated exposure; besides that it was not observed any significant difference between 300h and 600h of exposure. The change in color is caused by the deposition of micro particles such as dirt, soiling etc. Paints A2 and B4 were the products that showed the highest color change as shown in Fig. 7. Biological organisms probably caused the difference in color observed on specimens painted with A2. In the accelerated test, the paint sample is subject to a specific but unnatural condition. In the case of B4, the paint constituent is pure acrylic resin. According to the manufacturer, it is a product with elastomeric characteristic (800%), probably based on a resin with low Tg (glass transition temperature); the painted surface is tacky, with poor resistance to dirt pick-up, so it is easily impregnated with soil micro particles like clay, present near the Pirassununga station. According to Fig.6, the coatings obtained with semi-gloss paints A3, B3 and B4, after a one-year exposure at the outdoor weathering site, presented a significant change (loss) compare to those of the accelerated weathering test, in which the change in gloss was not significant after 600h.

4 CONCLUSIONS

The results obtained with the selected paints showed that the accelerated and natural weathering did not present any correlation because the tests were conducted in very different conditions. In the accelerated test, many important factors are not considered such as pollution, biological microorganisms, rainfalls, substrate etc. The study also showed the importance of testing the materials at many different sites because each location has its particular set of influencing factors. The three natural weathering sites were geographically near but they presented rural, marine and urban atmospheres, which resulted in great differences in the exposure results, as illustrated in Figs. 3 and 4. Besides, Pirassununga and Ubatuba sites present much higher temperatures and rainfalls (Table 1). Therefore, the conclusion is that there is a need to develop accelerated laboratory tests involving more experimental variables, in order to have conditions as similar as possible to those of the intended end use of the materials.

5 ACKNOWLEDGMENTS

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6 REFERENCES

Hicks, L.S. & Crewdson, M. J. 1995, 'Natural weathering', in *Paint and Coating Testing Manual Fourteenth Edition of the Gardner-Sward Handbook, ASTM manual series*; MNL 17, ed J. V. Koleske, Ann Arbor, Michigan, pp.619-642.

Jacques, L.F.E., Masters, L. & Lewry, A.J. 1996 'Practical approaches to weathering test design for the prediction of material service life', in *Durability of Building Materials and Components 7*, ed Sjöström, C., E&FN Spon, London, pp.1129-1140.

Kitsutaka Y. & Kamimura, K. 1993, ' Formula for the discoloration of external building materials', in *Durability of Building Materials and Components 6*, ed Nagataki, S., Nireki, T. & Tomosawa, F., E&FN Spon, pp. 707-714.

Sherbondy, V. 1995, 'Accelerated weathering', in *Paint and Coating Testing Manual Fourteenth Edition of the Gardner-Sward Handbook, ASTM manual series*; MNL 17, ed J. V. Koleske, Ann Arbor, Michigan, pp.643-653.

The evaluation method for weatherability of membrane materials for architectural membrane structures

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TT2-75

ABSTRACT

Weatherability of architectural membrane materials is usually evaluated by outdoor exposure and accelerated exposure tests. Weatherability is assessed by measuring tensile strength. Membrane materials are usually of PVC-coated polyester fabric [PVC membrane] or PTFE-coated glass fiber fabric [PTFE membrane]. For faster evaluation of weatherability of membrane material, accelerated exposure apparatus are used. However, although the condition of the test apparatuses such as UV light strength differs, the test results are usually compared only on the basis of exposure period. We investigated the weatherability of PVC membranes using three kind of different test apparatus those UV light strengthes were 180, 75, and 41 W/m^2 at 300-400 nm. And we also confirmed the correlation between the outdoor exposure and accelerated exposure tests according to the consideration of irradiation energy of UV. One result shows that 1-year of outdoor exposure test period corresponded to about 500 hours of accelerated exposure tests by using Sunshine carbon-arc lamp.

KEYWORDS

weatherability, membrane materials, architectural membrane structures, PVC-coated fabrics, accelerated exposure test

1.Introduction

Long-life performance should be the most important factor for the usage of membrane materials for architectural membrane structures such as pneumatic, suspended, steel-framed membrane structures. Polyvinyl chloride [PVC]-coated fabrics are widely applied as the membrane materials and their durability is usually estimated to be seven to fifteen years [Ansell & Harris 1980]. In these applications, the weatherability of the PVC-coated fabrics was evaluated by the outdoor exposure test and their durability assessed by measuring the retained tensile strength of samples after the outdoor exposure test. Outdoor exposure tests, however, take a long period to obtain the results. For faster evaluation, accelerated exposure tests are usually adopted and the test results are often compared only in terms of the exposure period although the condition of the test such as UV irradiation energy differs. In Japan, accelerated test methods of membrane materials [coated fabrics] are standardized by the Membrane Structures Association of Japan [MSAJ 2003]. In this method, irradiance for artificial weathering is specified. Here we will evaluate the weatherability of PVC-coated fabrics using three kinds of accelerated exposure test apparatus and discuss the evaluation methods of artificial weathering test.

2.Experimental

2.1 Specimen

PVC-coated polyester fabric [PVC specimen] for tent warehouse was examined. Table 1 shows the characteristics of the specimen.

Base fabric		*Coating material composition* [wt%]					*Thickness* [mm]	*Weight* [g/m^2]	*Tensile strength* [N/cm]
Yarn	*Density* [count/inch]	*PVC*	*Plasticizer*	*Stabilizer*	*Flame retardant*	*Pigment*			
Polyester spun yarn 20s/2/ 10s/1	52/46	100	DOP:70	7.5	Sb_2O_2:20	2.5	0.52	590	530/420

Table 1. Characteristics of PVC-coated polyester fabric for tent warehouse [warp/weft].

2.2 Outdoor expore test

The outdoor exposure test was carried out at the Japan Weathering Test Center located in Choshi-city, Japan. Choshi-city is representative of the average climate in Japan. The specimen was attached to a south-facing racks with rope in the inclination of 45 degrees.

2.3 Accelerated exposure tests

Accelerated exposure tests were carried out in three kinds of test apparatus. Table 2 shows their exposure test conditions.

Condition	*The types of UV lamps*	*UV strength* [W/m^2 at 300-400 nm]	*Testing condition*
A	Xenon-arc source	180	Black panel temperature: 63±3 deg.C Humidity: 50±5% Spray cycle: Duration of spraying: 18min, Dry interval between spraying: 102 min
B	Open-flame sunshine carbon-arc lamp	75	
C	Xenon-arc source	41	

Table 2. Accelerated exposure test conditions.

2.4 Evaluation

Tensile strength was measured after each exposure test and the~~n~~ retained tensile strength was calculated as a percentage of the original strength. The tensile test was carried out on the basis of JIS L 1096. The surface of PVC membrane was observed by SEM.

3. Results and discussion

Figure 1 shows the retained tensile strength for PVC specimen after each accelerated test. The exposure condition such as UV light intensity clearly affects the retained tensile strength. In Condition A, the highest UV light intensity damaged the retained tensile strength most severely to 56 and 39 % after 1000 and 2500 hr accelerated exposure, respectively. However, in Condition C, the lowest UV light intensity retained the tensile strength to 81 and 72 % after 2500 and 5000 hr accelerated exposure, respectively.

Figure 2 shows the retained tensile strength for PVC specimen exposed outdoors. The retained tensile strength decreased to 75 % after the 5-years outdoor exposure test. Here, each experimental curve in Figs 1 and 2 was fitted to an exponential decay function~~s~~. Then, we examined the correlation between outdoor exposure test period and the accelerated exposure test period on the basis of the retained tensile strength. From Figure 2, after the 5-years outdoor exposure test, the retained tensile strength was 75%. The equivalent accelerated aging periods can be estimated from Figure 1 and these estimates are tabulated in Table 3.

Condition	*The accelerated exposure test period* [hr] *corresponding to the 5-years-outdoor-exposure test*	*The accelerated exposure test period* [hr] *corresponding to the 1-year-outdoor-exposure test*
A	553	111
B	2736	547
C	3888	778

Table 3. Correlation exposure times between outdoor exposures and the three accelerated aging tests.

The average UV irradiation dosage [300-400 nm] in the outdoors at the Japan Weathering Test Center was reported to be 135 MJ/m2 per one year [MSAJ 2003]. Then, the accelerated exposure test periods corresponding to the 1-year-outdoor-exposure test are calculated to be the values shown in the 4th column in Table 4. These values correspond roughly to the values in the 3rd column in Table 3. This means that three accelerated exposure test methods presented in this work gave almost the same result, that is, in cases where UV irradiation dosage is the predominant factor for the degradation of tensile strength and the conversion of one period to the other is possible for accelerated exposure tests as far as UV light intensity is known.

Condition	*The types of UV lamps*	*UV strength* [W/m^2 at 300-400nm]	*The accelerated exposure test periods* [hr] *corresponding to the 1-year-outdoor-exposure test*
A	Xenon-arc source	180	208
B	Open-flame sunshine carbon-arc lamp	75	500
C	Xenon-arc source	41	915

Table 4. The relationship between the irradiation energy in the accelerated exposure test and the test period corresponding to the 1-year-outdoor-exposure test in Japan [MSAJ 2003].

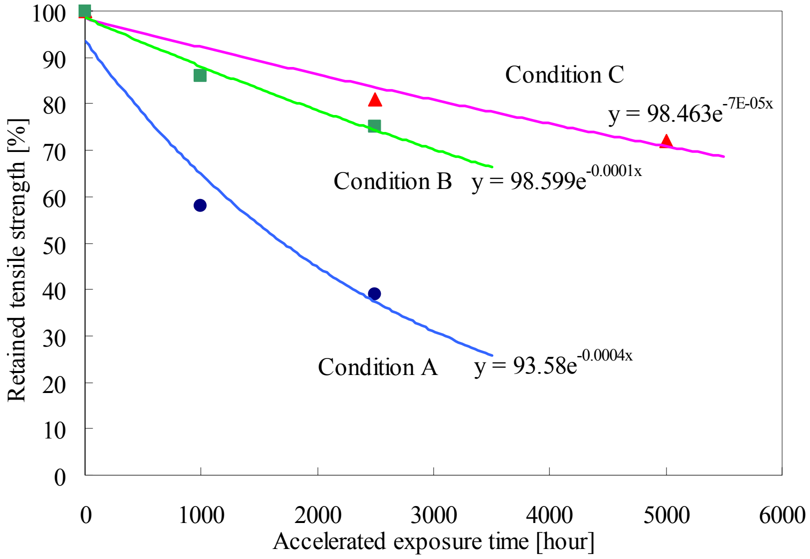


Figure 1. Retained tensile strength of PVC- coated fabrics after accelerated exposure tests.

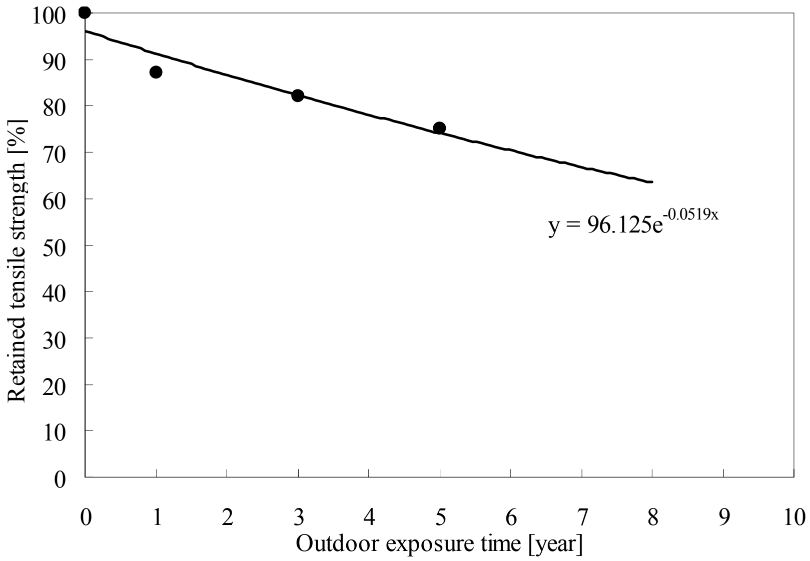
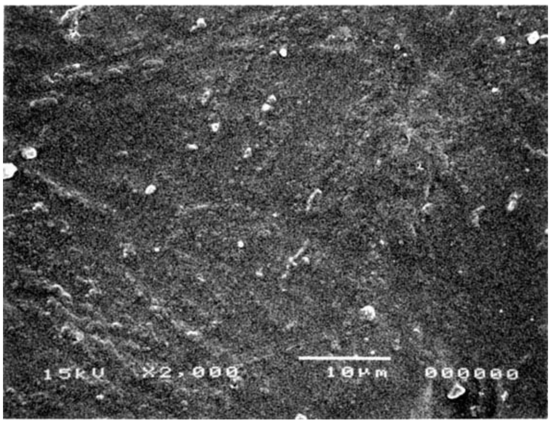
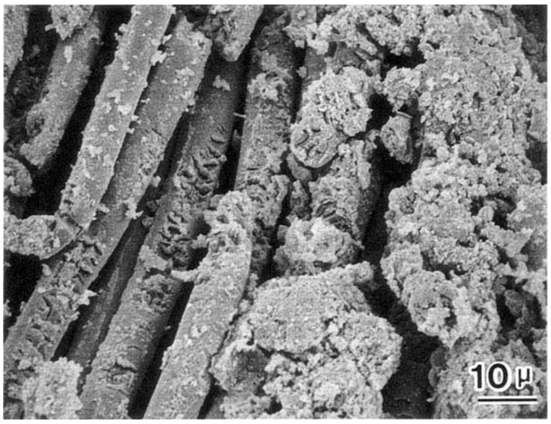


Figure 2. Retained tensile strength of PVC- coated fabrics after outdoor exposure tests.

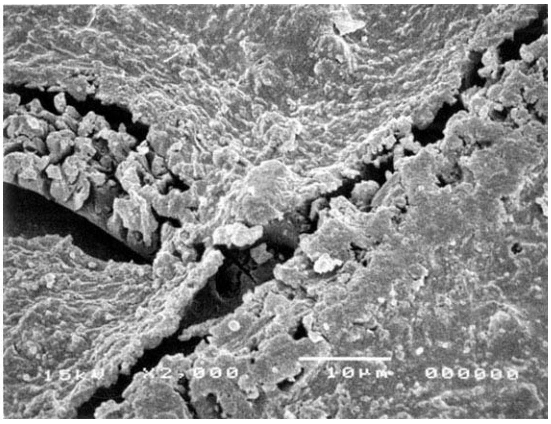
Figure 3 shows SEM photograph of PVC surface of a sample after 2500 hr accelerated exposure test. The degradation of PVC coating progressed with the increase of UV irradiation. For the sample under condition C, micro-cracks were observed on PVC surface. And the cracks were expanded and polymer fibers were exposed out of PVC coating for the sample under condition B. For the sample under condition A, exfoliation of PVC coating was observed. Moreover, polyester fibers under PVC coating were observed to degrade and fracture. The result that the tensile strength decreased severely can be understood from such morphological deterioration.

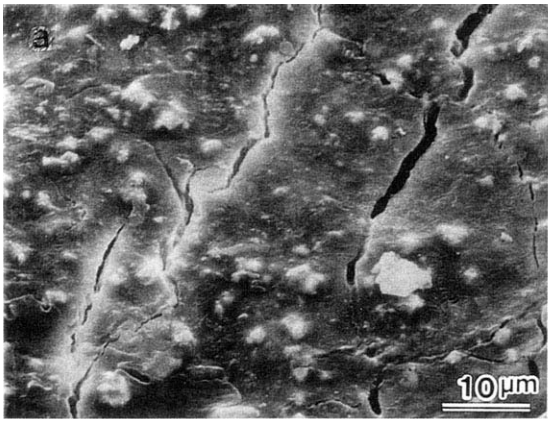
Morphology of PVC surface after 5-years outdoor exposure test observes similar to that of the sample under condition C in Figure 3(Figure 4). This also corresponds to the result of tensile strength.





(a) Initial (b) Condition A (2500 hours)





(c) Condition B (2500 hours) (d) Condition C (2500 hours)

Figure 3. Scanning electron micrographs of PVC membrane surface accelerated-exposed for : (a)Initial, (b)Condition A : 2500 hours, (c)Condition B : 2500 hours and (d)Condition C : 2500 hours.

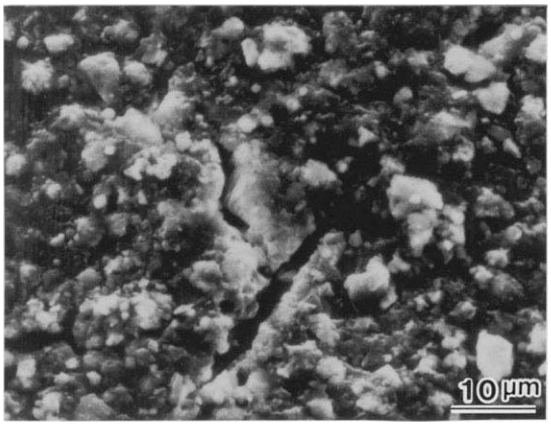


Figure 4. Scanning electron micrographs of PVC membrane surface outdoor-exposed for 5 years.

4. Conclusion

TT2-75, The evaluation method for weatherability of membrane materials for architectural membrane structures . Hiroshi Toyoda and Kenji Motohashi

Three accelerated exposure tests were carried out to evaluate weatherability of PVC membrane materials where UV light intensities were 180, 75, and 41 W/m^2 in the region of 300-400 nm. The results showed that UV irradiation dosage is a main factor for the degradation of tensile strength and the conversion of one period to the other is possible for accelerated exposure tests as far as UV light intensity is known. It is concluded that the accelerated exposure test with higher UV light intensity is preferable for faster evaluation of weatherability.

References

Ansell, M.P. and Harris, B. 1980, 'Air-Supported Structures: The State of the Art', Institution of Structured Engineers, London, 8, p. 121.

The Membrane Structures Association of Japan 2003 [MSAJ 2003], *Membrane Materials [Coated Fabrics]-Qualities and Performances* [MSAJ/M-03: 2003].

Influence Of Composition Of uPVC On The Impact Resistance Of Window Profiles Exposed To Weathering

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TT2-78

ABSTRACT

The durability of unplasticized poly(vinyl chloride) (uPVC) components used in the building industry is a function of the interaction of several factors. Types and quantities of additives incorporated to the PVC resin, defining the formulation of the PVC compound used in the production of these components, are among the most important such factors. White uPVC window profiles with different formulations have been exposed to five years of natural weathering under Brazilian climatic conditions. Results of Charpy impact tests were used to assess the influence of three factors on the durability of the profiles: additivation levels of titanium dioxide, nature of the impact modifier and nature of the thermal stabilizer. Statistical analysis of the results made it possible to rank these factors according to their influence in component durability.

Results presented in this paper indicated the need for several additional experiments in order to check changes in molecular mass, color and elasticity modulus. Such experiments were later carried out. Comparison of all these data with results from profiles subjected to accelerated weathering led to straightforward criteria for prediction of the potential behaviour of uPVC, to be described in a future paper.

KEYWORDS

PVC; Plastics, Durability; uPVC profiles; PVC formulation.

1 INTRODUCTION

The use of rigid PVC profiles for windows and external sidings is just starting in Brazil. Lack of information on the material behaviour during its lifetime, when subjected to solar radiation and temperatures prevalent in Brazil, is one of the main factors that hinder growth of uPVC use. This study addresses this issue.

2 METHODOLOGY

2.1 Experiment planning

Durability was assessed by means of the Charpy impact resistance. Samples cut from white extruded uPVC profiles were exposed to natural weathering conditions, at an angle of 45 degrees north, according to the ASTM D1435 Standard, 1994, for periods up to five years. The natural weathering station is located in the city of Piracicaba, State of São Paulo, Brazil (lat. 22º43' South, long. 47º25' West; 580 m high).

In order to reduce statistical uncertainty, tests were performed on ten specimens for a given set of conditions and the average value was calculated to represent the measured properties (BS 7413, 1994 and ASTM D256, 1984). Thus, the value of the Charpy impact resistance of a given formulation, after a given exposure period, for example, is the average of the breaking energy of ten specimens.

2.2 Selection of formulations

Durability is central to the performance of PVC windows. Resistance of a compound formulation to weathering is therefore an extremely relevant property to be predicted. White profiles were selected for the experiment because as they are pigmented with titanic dioxide, they usually exhibit better durability irrespective of weather conditions (Lee et.aliii, 1995; Lemaire, 1996; Rabinovitch, 1993).
The four impact modifiers then (1988) in use in Brazil were used in the experiment: acrylic (ACR), chlorinated polyethylene (CPE), acrylonitrile-butadiene-styrene (ABS), and metacrylate-butadiene-styrene (MBS). Such modifiers are still (2004) the most widely used in Brazil. Four thermal stabilizers were analyzed: a complex of lead, barium and cadmium (Pb/Ba/Cd), lead (Pb), a complex of barium and cadmium (Ba/Cd), and tin (Sn).

Due to the high cost of titanium dioxide, the experiment was conducted for three additivation levels: 3 combined with anti-UV additives, 5 and 10 phr (parts per hundred resin). The titanium dioxide (TiO_2) pigment promotes opacity and PVC resin protection against degradation due to ultra violet (UV) radiation.

For practical reasons related primarily to the extrusion process, not all possible combinations of these factors were tested: only 19 of 37 compound formulations could be exposed to natural weathering. Table 1 shows all compound formulations that were processed and how many were tested.

	Pb/Ba/Cd	*Pb*	*Sn*	*Ba/Cd*	*Number of compounds processed/studied*
Acrylic	4 / 2(1)	6 / 2(1)	2 / 0	5 / 3	17 / 7
CPE	3 /3	3 / 3	7 / 0	(3)	13 / 6
ABS	3 / 3(2)		1 / 0		4 /3
MBS	3 / 3(2)				3 / 3
Total	13 / 11	9 / 5	10 / 0	5 / 3	37 / 19

(1) Only compounds with 3 and 10 phr of TiO_2 were exposed to weathering.
(2) In this case, as ABS and MBS impact modifiers reduce weathering resistance of the final compounds, it was utilized only one thermal stabilizer, just to compare mechanical properties to others impact modifiers.
(3) CPE impact modifiers were not combined with Ba/Cd thermal stabilizer because it is very difficult to find the correct lubrification.

Table 1. Compound formulations processed/studied

Codes used to identify the compounds are summarized in Table 2.

Impact modifiers 2 letters		*Thermal Stabilizer 1 letter*		*Additive (phr of TiO_2)*
LL		L		N
AC	ACR (acrilyc)	**P**	Pb	**3**
CP	CPE	**S**	Sn	**5**
AB	ABS	**B**	Ba/Cd	**10**
MB	MBS	**C**	Pb/Ba/Cd	

Example: ABC5: Compound with ABS impact modifier, thermal stabilizer Pb/Ba/Cd and 5 phr of TiO_2.

Table 2. Compound coding.

2.3 Natural weathering

Climatic variables were measured several times per day during the 5-year period of the experiment (January, 1989 – December, 1993): global solar radiation (cal/cm².day), hours of sunlight (hours/day), rain (mm), relative humidity (%), wind velocity - maximum, minimum and average (m/s), temperature – maximum, minimum and average (°C). The yearly values of meteorological data for the weathering station are summarized in Table 3.

Year	***Total global yearly radiation kLy(1)***	***Total hours of sunlight per year (1)***	***Maximum average temperature (average of monthly maxima), °C***	***Average temperature (average of monthly average temperatures) °C***
1989	131.4	2 579	27.9	21.2
1990	133.7	2 579	28.8	22.1
1991	131.0	2 511	28.5	22.0
1992	126.6	2 289	28.1	21.8
1993	128.4	2 352	28.6	22.3
Average	**130.2**	**2 462**	**28.4**	**21.9**
Total	**650.9**	**12 310**		

(1) Values obtained from the sum of values of daily solar radiation and insolation measured in Piracicaba during the experiment (1 kLy = 1 kLangley = 1.000 cal/cm^2)

Table 3. Meteorological data for the city of Piracicaba – yearly values between 1989 and 1993.

3 RESULTS

Values of the Charpy impact resistance of each of the 19 evaluated formulations, for different exposure times, are summarized in Table 4.

Formulations	*1*	*2*	*3*	*4*	*5*	*6*
Content of TiO_2 pigment	*3 phr*	*5 phr*	*10 phr*	*3 phr*	*5 phr*	*10 phr*
Modifier	MBS			ABS		
Stabilizer	Pb-Ba-Cd			Pb-Ba-Cd		
0 years	11.8	12.6	18.2	10.7	11.5	16.6
1,17 years	7.7	10.7	13.1	7.0	7.7	13.7
2 years	9.3	9.0	11.3	8.1	8.7	13.5
3 years	5.1	6.5	6.7	5.1	5.8	9.1
4 years	6.0	7.4	8.6	7.1	7.5	10.7
5 years	3.4	4.0	4.5	3.4	4.9	7.8

Formulations	*7*	*8*	*9*	*10*	*11*	*12*
Content of TiO_2 pigment	*3 phr*	*5 phr*	*10 phr*	*3 phr*	*5 phr*	*10 phr*
Modifier	CPE			CPE		
Stabilizer	Pb			Pb-Ba-Cd		
0 years	14.1	16.6	16.6	16.7	16.5	15.5
1,17 years	12.9	16.8	15.0	15.0	13.6	15.8
2 years	14.4	20.4	18.8	14.8	14.3	16.4
3 years	13.6	19.1	15.7	13.0	14.7	15.6
4 years	14.7	19.2	17.1	13.0	14.8	16.1
5 years	10.4	N/D	14.5	7.2	9.8	11.5

Formulations	*13*	*14*	*15*	*16*	*17*	*18*	*19*
Content of TiO_2 pigment	*3 phr*	*10 phr*	*3 phr*	*10 phr*	*3 phr*	*5 phr*	*10 phr*
Modifier	Acrylic		Acrylic		Acrylic		
Stabilizer	Pb		Pb-Ba-Cd		Ba-Cd		
0 years	12.7	18.6	11.1	14.2	15.0	12.2	15.5
1,17 years	13.9	18.3	12.1	17.2	14.1	12.9	14.3
2 years	16.1	19.3	13.3	15.1	15.2	14.1	18.3
3 years	12.6	18.7	11.6	13.9	13.4	13.0	14.0
4 years	11.3	18.3	12.2	16.7	15.0	13.5	14.3
5 years	4.8	16.9	8.4	11.8	13.0	11.6	14.4

Table 4. Charpy impact resistance after natural weathering of the 19 formulations evaluated (kgf.cm/cm or kJ/m^2).

A first attempt at linear correlations of resistance with the independent variables proved unsuccessful (R^2=0,59), as might be expected from the generally non-linear trends observed in Table 4 and in Figures 1 and 2.

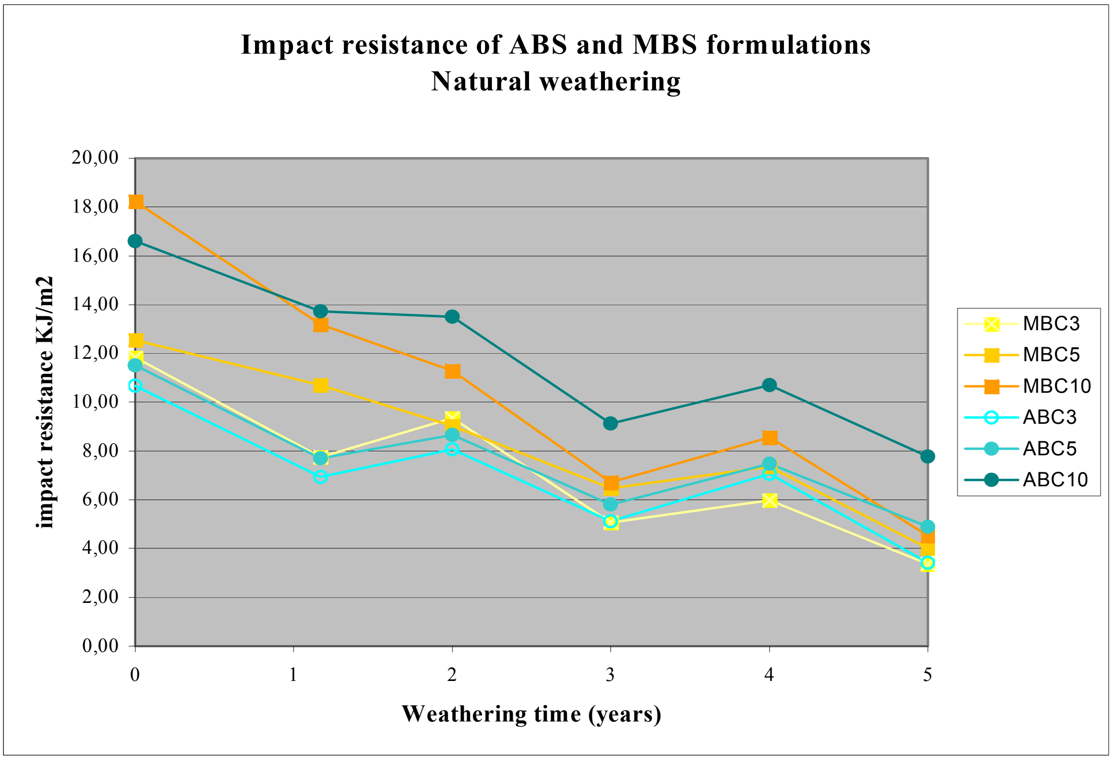

Figure 1. Impact resistance of MBS and ABS formulations, in natural weathering

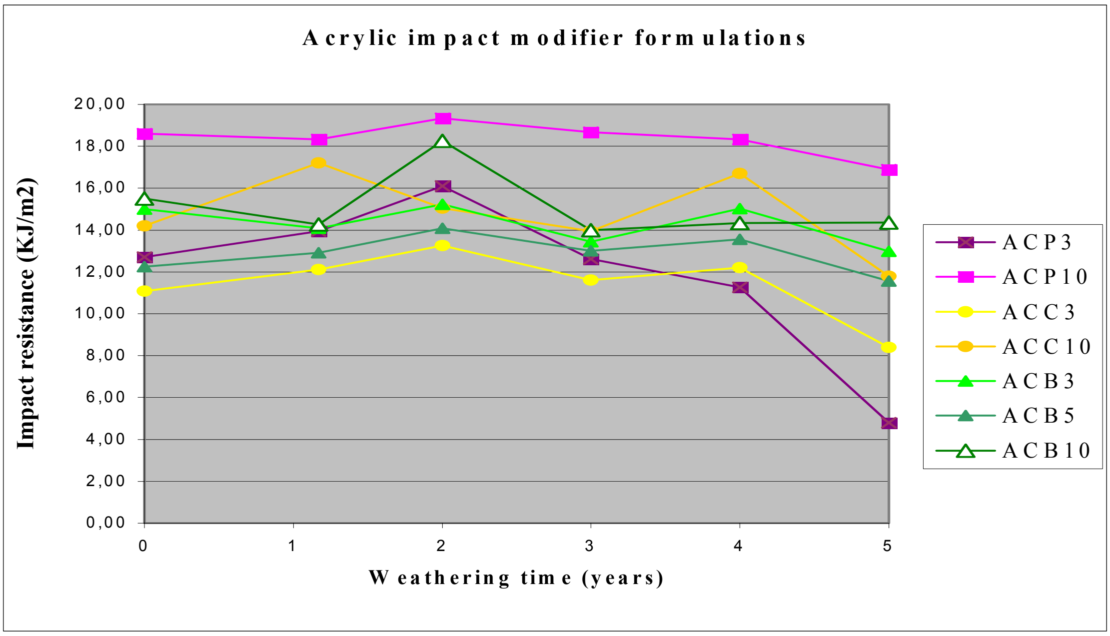

Figure 2. Impact resistance of acrylic impact modifier formulations: thermal stabilizers influence versus TiO_2 content

As a matter of fact, decreasing rates of decay are commonly observed in degradation processes. This conceptual model was not applied due to the rather significant fluctuations observed (see Figures 1 and 2 for example).

Considering the common practice of taking the relative decrease of PVC resistance to represent weathering, a new variable was defined, the relative decrease in resistance at time t = 5 years:

[RI (t=0)-RI (t=5)] / RI (t=0)], or, ΔRI (t=5) / R (t=0)

This new variable, if divided by the exposure period, becomes the average annual rate of decrease in resistance for the period.

A linear multiple regression model was applied to this new variable. Time and TiO_2 pigment content were both, of course, defined quantitatively, while different impact modifiers and stabilizers had to be associated to dummy variables for the purpose of the regression.

Only 18 points were considered for the regression because one of the formulations was lacking the value of the 5-year exposure impact resistance (CPP5 – Table 4).

Table 5, transcribed directly from the electronic spreadsheet used to carry out the multiple regression, shows a value of about 0.8 for the determination coefficient and a high value of the F statistic; thus, pointing at a reasonable linear fit.

More importantly, the t-statistic and the corresponding P-value help rank the regressors: the lower the P-value, the lower the probability of the coefficient of that particular regressor being zero. In other words, the lower the P-value, the higher the importance of that regressor in explaining the decrease in resistance.

Regression statistics	
R multiple	0.888
R-square	0.789
R-square adjusted	0.744
Standard error	0.026
Number of observations	18.000

ANOVA

	d.f.	*SS*	*MS*	*F*	*F significance*
Regression	3.000	0.034	0.011	17.488	5.2E-05
Residual	14.000	0.009	0.001		
Total	17.000	0.043			

	Coefficients	*Standard error*	*t*	*P-value*
Interseccion	0.215	0.021	10.428	5.5E-08
Variable X 1=TiO_2	-0.005	0.002	-2.463	2.7E-02
Variable X 2=modifier	-0.028	0.007	-4.063	1.2E-03
Variable X 3=stabilizer	-0.018	0.010	-1.821	9.0E-02

Table 5. Summary of results

4 CONCLUSIONS

The results of the experiment indicate that the choice of impact modifier is the most important factor controlling decrease in resistance in the experimental conditions analyzed. The TiO_2 pigment content comes next, followed by the type of thermal stabilizer.

These conclusions can be better visualized in Figure 3, which derives from the linear multiple regression.

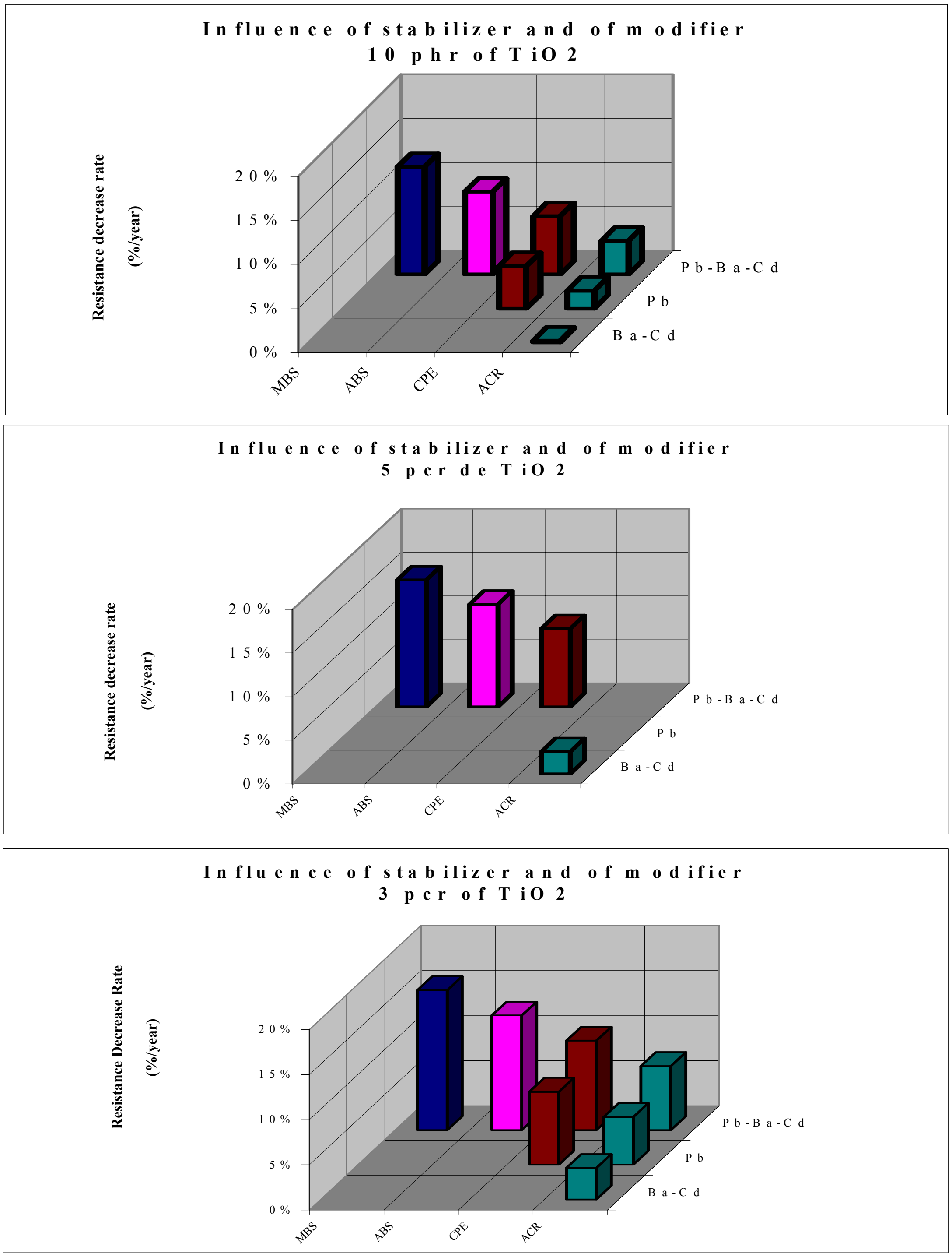

Figure 3. Influence of modifier and of stabilizer

5 FINAL REMARKS

The results presented herein were the basis of a more encompassing set of experiments, which included determination of the changes in molecular mass, color and elasticity modulus, as well as their extension to uPVC specimens subjected to accelerated weathering.

This experimental program led to the proposal of straightforward criteria for prediction of the potential behavior of uPVC compounds under Brazilian climatic conditions, as shown in Hachich, 1999.

6 ACKNOWLEDGEMENTS

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TESIS - Tecnologia de Sistemas em Engenharia S/C Ltda

7 REFERENCES

AMERICAN SOCIETY FOR TESTING AND MATERIALS. *Standard Test Methods for Determining the Pendulum Impact Resistance of Notched Specimens of Plastic;.* ASTM D256. Philadelphia, 1984.

____. *Standard Practice for outdoor weathering of plastics;* ASTM D1435. Philadelphia, 1994.

BRITISH STANDARD INSTITUTE - BSI *Specification for white PVC-U extruded hollow profiles with heat welded corner joints for plastic windows: materials type A*. BS 7413. London, 1994.

HACHICH, V.C. FERNANDES. *Minimum criteria for evaluation of Rigid Poly (Vinyl Chloride) (PVC) products for exterior use in buildings.* São Paulo, 1999. Thesis (for obtain the title of PhD) – Polytechnic School of University of São Paulo.

LEE, Chul-Ho; SHIM, Dae-Sup; Kim, Sang-Wook. Effect of pigment on weatherability of PVC. *Kongop Hwahak*, 6 (1), p.22-8, 1995.

LEMAIRE, Jacques. Predicting polymer durability. *Chemtech*, 26 (10), p.42-47, 1996.

RABINOVITCH, Elvira B. Use of titanium dioxide in vinyl building products. *European Polymers Paint Colour Journal*, 183 (4339), p. 521-4, 1993.

Impact of Severe Service Conditions on Hygrothermal Performance of Sandwich Panels

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ABSTRACT

The impact of severe service conditions on the hygrothermal performance of sandwich panels was experimentally investigated. Three test houses with a surface area of 10 m^2 were constructed with three different types of sandwich panels i.e., polyurethane (PU) foam, rock wool and glass wool. The panels were made of steel face sheets in dark grey or in light grey. The first house fabricated with a panel 80 mm thick is located in French Guyana (Cayenne). The second house with a panel 80 mm thick and the third house with a panel 150 mm thick are located in Finland (Hameenlinna). These houses were equipped with 195 sensors in total. The set of sensors comprise heat flux, RTD, wetness, water content, temperature-relative humidity (T-RH), thermal conductivity and pyranometers. A special data acquisition system was developed so that all measurement can be monitored remotely (France). The room air temperature was controlled using an air conditioning system. It was initially set up to be about 5-10 °C and about 20 °C afterwards for Guyana, and about 20-21°C at the beginning and 15-19 °C afterwards for Finland. This experiment was monitored over a one-year period.The properties of the core materials were measured in a laboratory. The results showed that the orientation of the fibre in mineral wools has no effect on the water vapor permeability, but a significant effect on the thermal conductivity. The volumetric rate of open cells in PU was measured using a pycnometer. The rate was found to be on the order of 7.6 %, which it corresponds to the permeability of PU being much less permeable than the mineral wools.The field measurements revealed that the temperature on the exterior surface of dark grey panel is higher than that of the light one. Thermal conductivity deduced from heat flux measurements varied with temperature throughout the experiment. Dispersion of measurement of thermal conductivity obtained from the hot wire probes increases with an increase in initial temperature and absolute humidity. Measurement with the T-RH probes showed that water vapor diffusion in the presence of a temperature gradient is dominated by temperature rather than absolute humidity. Under severe service conditions, condensation occurred on the inner surface of the interior steel face sheet for the climate of Guyane while on the inner surface of the exterior steel face sheet for the climate of Finland. However, this condensation had a small impact on the properties of the insulation throughout the experiment. The polyurethane foam seems to have the best performance in comparison with the two other materials. This indicated that three types of panels are durable. The sandwich panel was modeled and simulated using a common software tool, WUFI 2D. In this study, a two-dimensional heat and moisture transfer through the panel was allowed for. As the surfaces of panels are impermeable to vapor and water, it was difficult to make simulation. However, it was found that imposing exterior relative humidity on both edges of the panel as the boundary conditions in the simulation gave a good result regarding temperature and relative humidity, compared with the measurements for Finland, but not for Guyana.

KEYWORDS: Degradation, Simulation, Field Test, Condensation.

1 INTRODUCTION

Moisture is one of the factors inducing degradation of thermal insulation, an important component of buildings. Thus installation of insulation by keeping it away from moisture is necessary in order to maintain its performance. In practice however, it is unavoidable to admission of moisture. The influence of moisture on the variation of thermal conductivity of fibrous insulation in presence of temperature gradient was experimentally investigated by Langlais et al. [1982]. The thermal conductivity of the specimen were measured over the moisture range of 0-150 percent by weight The results show that the variation of thermal conductivity with respect to dry state may reach 10 percent for a moisture content of about 30 percent by weight, while in practice, moisture presence in fibrous insulating material due to adsorption at 90 percent relative humidity are of 2 percent or less (in mass). For this small amount of water the variation of thermal conductivity with respect to dry state may be negligible. Similar experiments were performed by Kumaran [1987] and Kumaran [1988]. Two coefficients in the moisture transfer equation, associated with the properties of material were determined using least square analysis. These two coefficients are water permeability and the coefficient of thermal vapor diffusion. Further analysis on physical models of vapor diffusion and condensation in the fibrous insulations was carried out by Wijeysundera et al. [1989]; [1992]; [1995]. Besides thermal conductivity, water vapor permeability of fibrous insulation was also studied. The effect of fibre on the impeding of diffusion is relatively small. Water vapor diffusion in fibrous insulation is 1-2 folds of that of still air referring to IEA [1996]. Measurement of water vapor permeability of polyurethane and polyisocyanurate by modified cup method in the presence of a temperature gradient was reported by Schwartz and Bomberg [1989]. The results indicated that the accumulation of moisture occurred in the material when a presence of a temperature gradient acts in the same direction as the pressure gradient. Although the properties of moist insulation regarding thermal conductivity and water vapor permeability have been researched, there is still a lack for in-situ implementation and for varieties of tested materials, which need to be investigated further.

A widely used building component nowadays is sandwich panel whose main core material is thermal insulation. Though steel face sheets of panels serve as moisture barrier of wall, water vapour pressure is still able to penetrate through the wall via fixing joints. This vapour may condense and accumulate in the wall under the severe climates or service condition. Apart from variation of thermal insulation, it probably results in adhesive detachment of inner surface of steel face sheet from the insulation layer.

2 MATERIALS AND METHODS

2.1 Test houses

Three test houses with a surface area of 10 m^2 were constructed with three different types of sandwich panels i.e., polyurethane (PU) foam, rock wool (RW) and glass wool (GW). The panels which are usually named according to core material used (thermal insulation) were made of steel face sheets in dark grey or in light grey. In this study, the mineral wool panels were fabricated by cutting insulation slab into small pieces and turning up their side on the steel sheets. The first house fabricated with a panel 80 mm thick is located in French Guyana (Cayenne). The second house with a panel 80 mm thick and the third house with a panel 150 mm thick are located in Finland (Hameenlinna), Fig.1. These houses were equipped with 195 sensors in total. The set of sensors comprise heat flux, RTD, wetness, water content, temperature-relative humidity (T-RH), thermal conductivity and pyranometers. The positions of sensors in the panels are shown in the figure 2. The panels equipped with most sensors oriented east for Guyana and south for Finland. A special data acquisition system was developed so that all measurement can be monitored remotely (France). The room air temperature was controlled using an air conditioning system. It was initially set up to be about 5-10 °C and about 20 °C afterwards for Guyana, and in the range of 20-21 ^{o}C at the beginning and 15-19 ^{o}C afterwards for Finland. This experiment was monitored over a one-year period.

Figure 1 Test houses in Guyana and in Finland

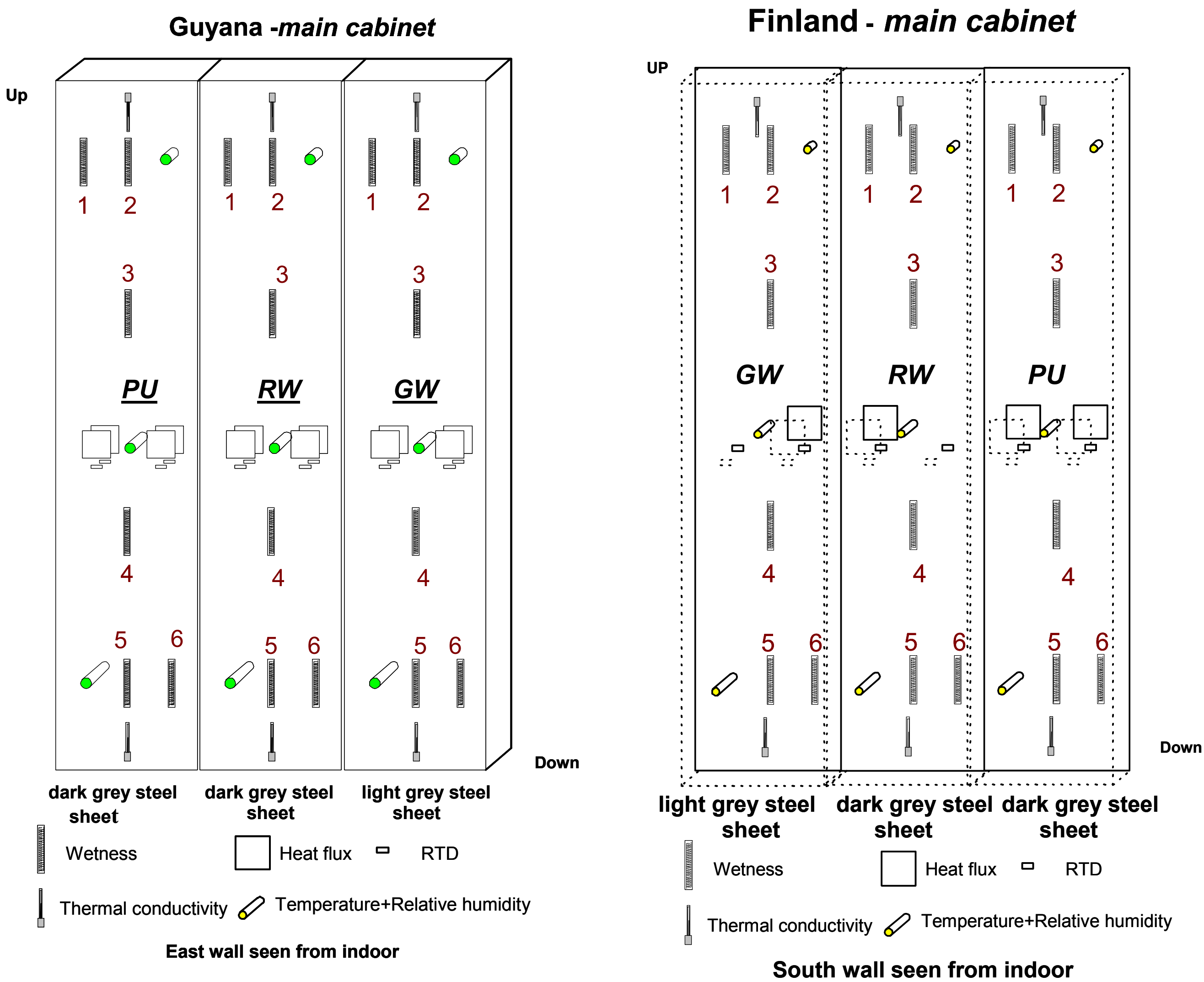

Figure 2 Position of sensors in the panels

The properties of the core materials were measured in a laboratory. The measurements conformed to NF EN ISO-12086,-12570, -12571 and ASTM D2856-94. The results showed that the orientation of the fibre in mineral wools has no effect on the water vapor permeability, but a significant effect on the thermal conductivity. The volumetric rate of open cells in PU was measured using a pycnometer. The rate was found to be on the order of 7.6 %, which it corresponds to the permeability of PU being much less permeable than the mineral wools.

2.2 Weather and indoor test conditions

As French Guyana is located at 4 N 53.0W, the weather is hot and humid. The room air temperature for this site is set to be relatively low throughout the experiment. For Finland (61 N 24.25 E), the weather is cold and relatively humid in the winter. The room air temperature of the test houses is shown in Fig. 3

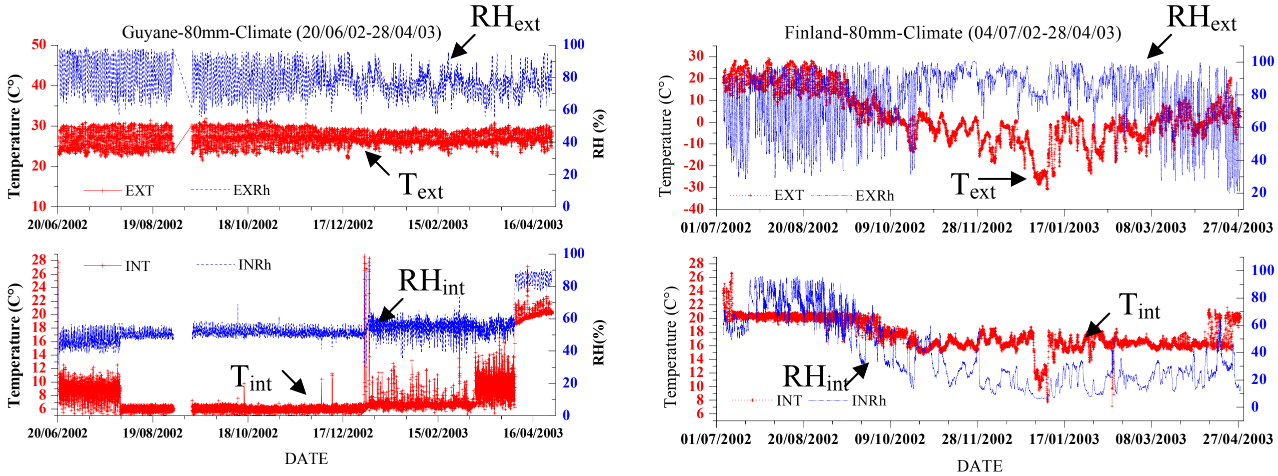

Figure 3 Outdoor and indoor climate of test houses in Guyana and in Finland

3 RESULTS

3.1 Surface temperature

The field measurements revealed that the temperature on the exterior surface of dark grey panel is higher than that of the light one.

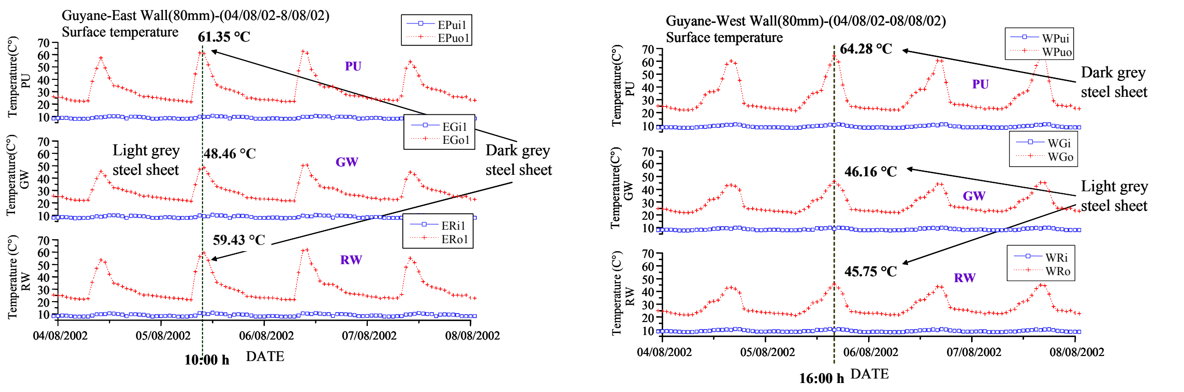

Figure 4 Temperature on the surface of wall oriented East and West for test house in Guyana

3.2 Condensation on the surfaces

The condensation on the surface according to positions in Fig. 2 was detected using wetness sensors. As thermostat was set to the low temperature, condensation occurred on the inner surface of interior steel face sheet under the climate of Guyana. In contrast with Guyana, condensation occurred on the inner surface of the exterior steel sheet for Finland. Figures 5 and 6 presented the histogram of condensation for three consecutive periods (3 months of each period) of the experiment.

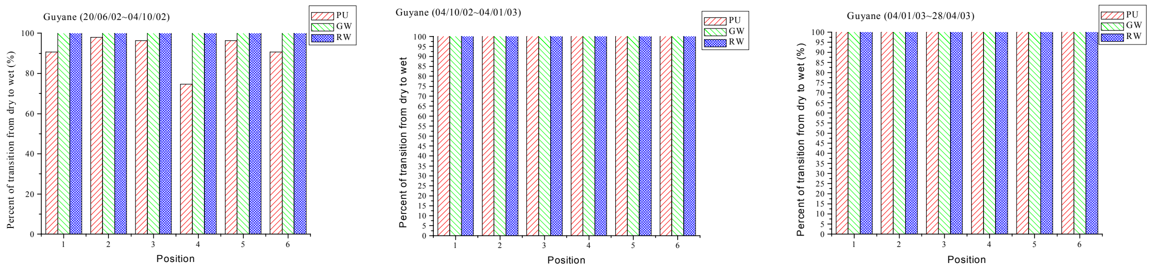

Figure 5 Three consecutive histograms of condensation on the inner surface of interior steel face sheet for the climate in Guyana

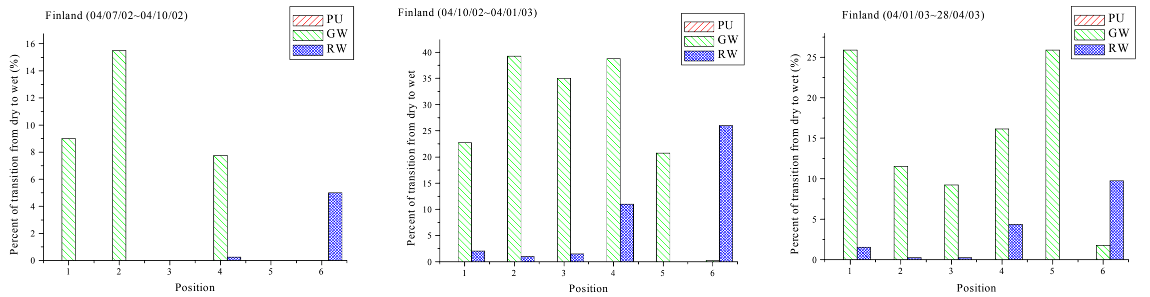

Figure 6 Three consecutive histograms of condensation on the inner surface of interior steel face sheet for the climate in Finland

3.3 Gradient coexistent

The climate of Guyana referred to Fig.3 is considerably stable for a whole year. Thus, any period of experiment can be used as representativeness. Instantaneous temperature, relative and absolute humidity were plotted along the distance of the panel, Fig.7. It was found that in case of Guyana where the daytime exterior temperature is higher than the nighttime, gradient of temperature and absolute humidity decreased towards to the interior while in the opposite direction for the relative humidity.

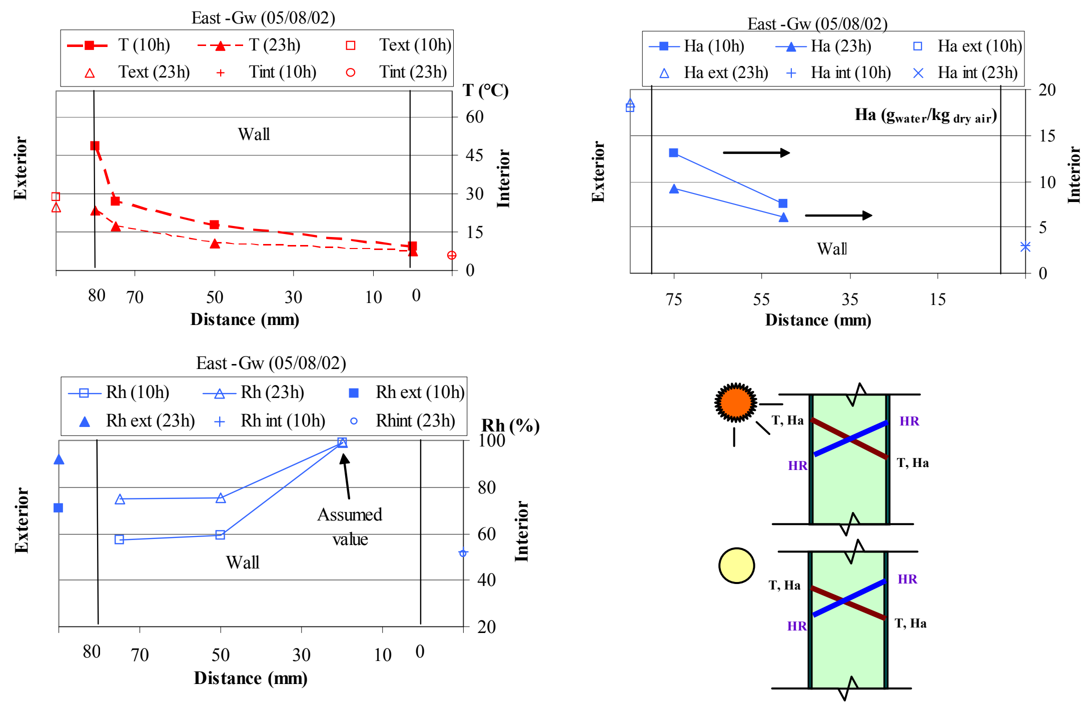

Figure 7 Temperature, relative humidity and absolute humidity gradient of the panels for the climate in Guyana

In case of Finland, the gradient of absolute humidity across the panel is very small in the winter, the analysis was then considered on the summer period. As shown in Figs. 8 and 9, the results showed that the direction of gradient is in agreement with that of Guyana for the mineral wool but PU panel.

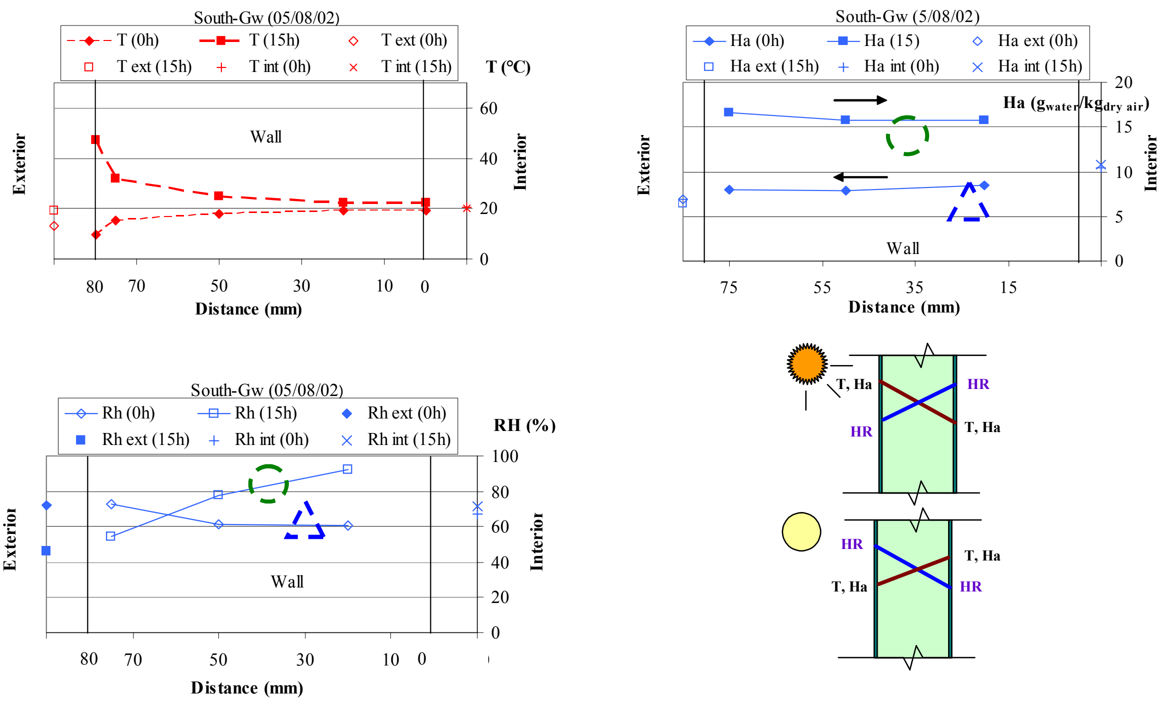

Figure 8 Temperature, relative humidity and absolute humidity gradient of the mineral wool panels for the climate in Finland

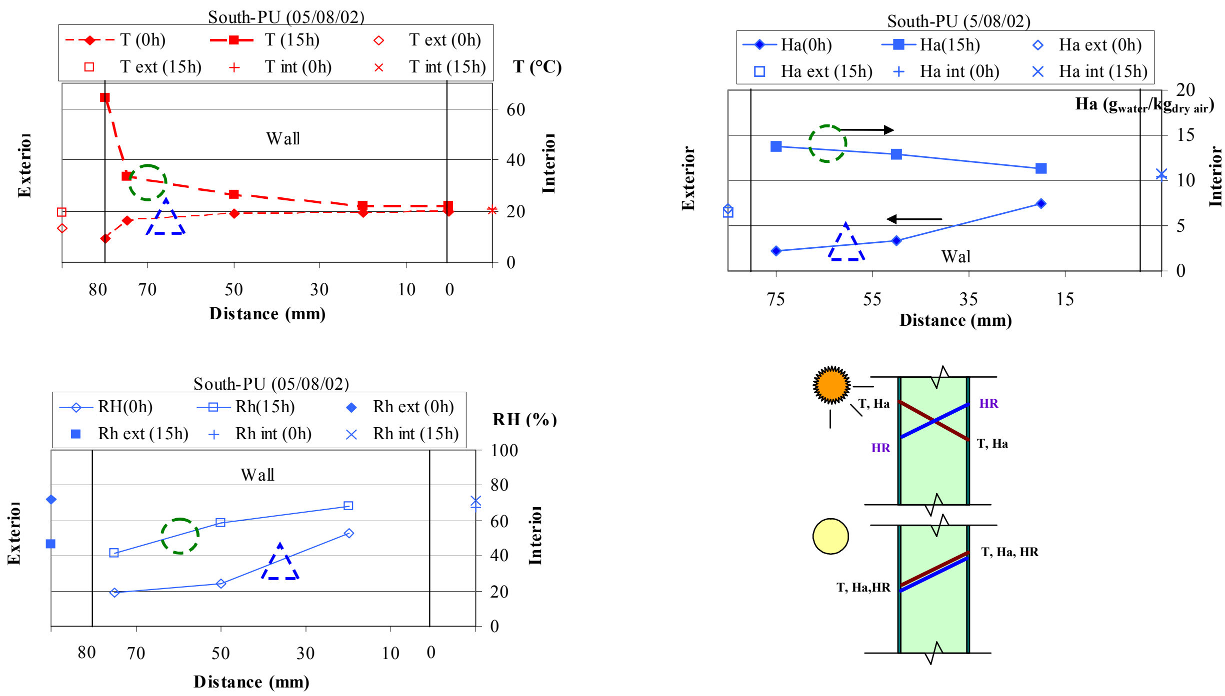

Figure 9 Temperature, relative humidity and absolute humidity gradient of the PU panels for the climate in Finland

3.4 Variation of thermal conductivity

3.4.1 Thermal conductivity deduced from measured heat flux and temperature

Thermal conductivity of core material can be deduced from the measured heat flux and the temperature on the panel surfaces. As indicated in Fig. 10, the variation of thermal conductivity is presented in term of the conductivity ratio, measured thermal conductivity to the reference value (at 23 ^{o}C, 50 % RH). For Guyana, it is obvious that thermal conductivity of three types of materials vary between 1-1.5 times of reference one almost throughout the experimental period except for the end of experiment where the room temperature was increased. For Finland, less variation in thermal conductivity was found, Fig. 11.

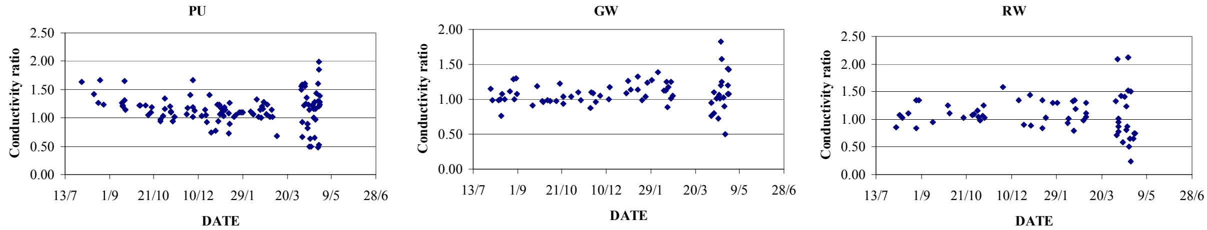

Figure 10 Variation of thermal conductivity of core materials for the climate in Guyana

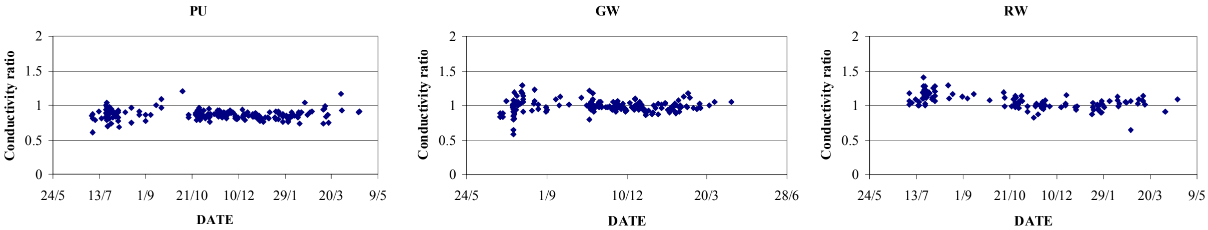

Figure 11 Variation of thermal conductivity of core materials for the climate in Finland

3.4.2 Thermal conductivity measured using hot wire probe

Two hot wire probes were installed in each type of panels, one at the top and another at the bottom. As a result of condensation at the inner surface of interior steel face sheet for test houses in Guyana, there is a disturbance of measurement in humidity (T-Rh probe), the variation of measured thermal conductivity was then plotted only in a function of the initial temperature of measurement according to the usage of probes. Figure 12 indicated that the thermal conductivity increases with an increase in temperature, which is in a polynomial form.

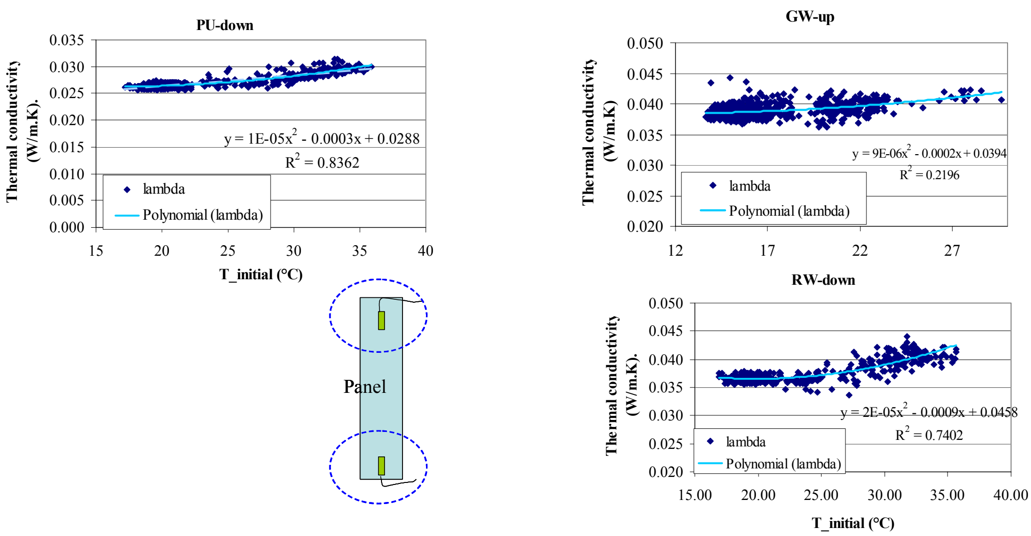

Figure 12 Variation of thermal conductivity with temperature for the test house in Guyana

In case of Finland, condensation occurred at the inner surface of exterior steel face sheet. The wetness varied throughout the experimental period as referred to Fig. 6. This small condensation has no any significant effect on measurement. It is found that thermal conductivity vary linearly with temperature and the its dispersion of measurement increases with an increasing in absolute humidity as shown in Fig. 13, bottom right.

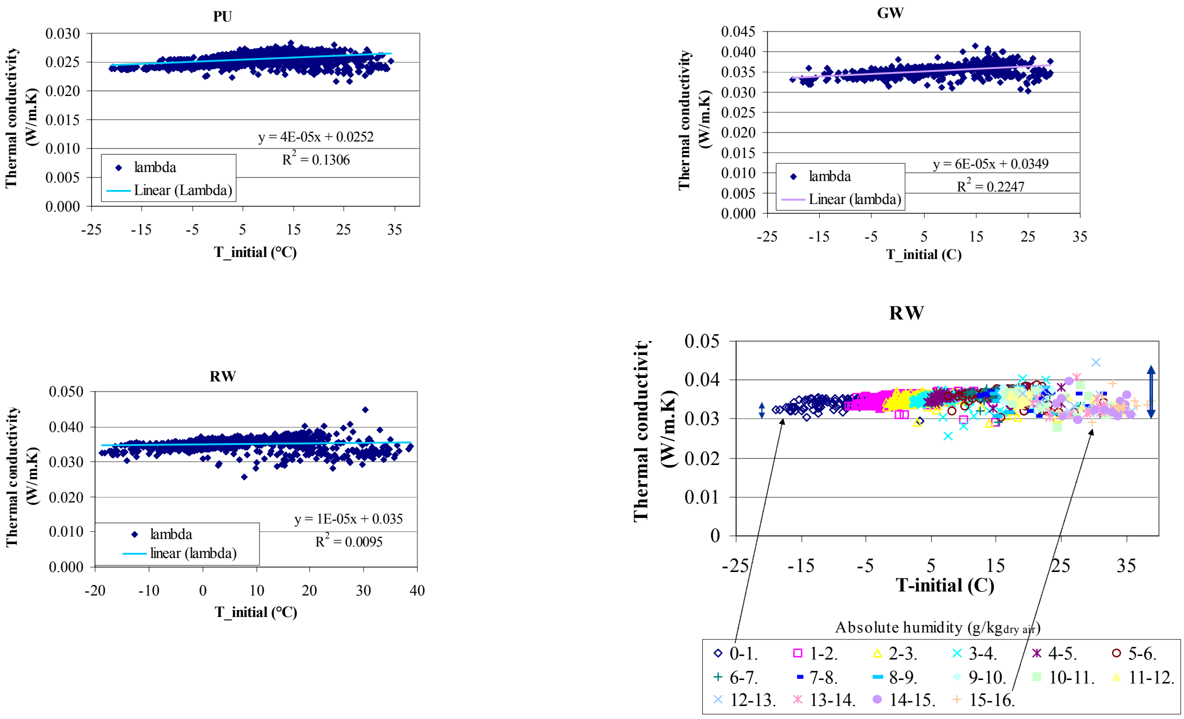

Figure 13 Variation of thermal conductivity with temperature for test houses in Finland

3.5 Simulation results

The hygrothermal behavior of sandwich panel was simulated using a common software tool, WUFI-2D. In simulation, the difficulties had been found since the system of simulation is quite tight to humidity. Several cases of panel models with corresponding boundary conditions were simulated and the simulation results were compared with the experimental one. For mineral wool panel, the model where the exterior humidity relative was applied at the left side and right side edge of panel gives a good result regarding temperature for the climate of Finland but Guyana, Fig.14

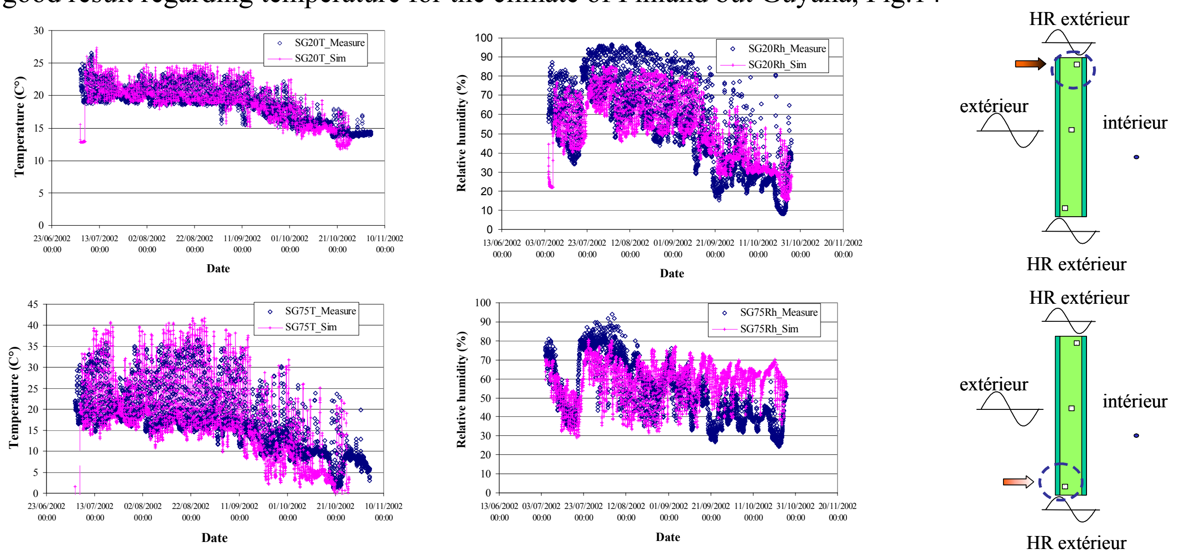

Figure 14 Comparison between simulation and experimental results of mineral wool panels in the climate of Finland

4 CONCLUSIONS

The condensation on the inner surface of the interior steel face sheets was found for all panels of the experimental house in Guyana and was found not to vary with the season. It was also found on the inner surface of the exterior steel face sheets of all panels of the houses in Finland but here it varied with the position and time according to the season. Thermal conductivity, analyzed using heat flux methods, varied 0.5-2 times the reference value for Guyana, and 0.75-1.2 times for Finland. These indexes were properly used for designing or estimating energy consumption of such buildings under the service conditions. Using transient hot wire methods, we found that the correlation between thermal conductivity of the insulation and temperature was in polynomial form for Guyana and in linear form for Finland. In the case of the test house in Finland, it is obvious that the deviation of the measured thermal conductivity increases with the increases in moisture (absolute humidity) and temperature. Among the three types of panels, the performance of the glass and rock wool during this experiment period are close. The PU foam seems to be the best. In this experiment, it is indicated that durability of all types of sandwich panels is adequate. However, long term monitoring is still important. To avoid an effect of moisture, minimize initial moisture content at the early stage of construction is necessary, thus all materials should be kept in a dry condition before using in construction. A simulation of the hygrothermal behavior of a sandwich panel was performed using software tool WUFI 2D. It was feasible to develop a simply physical model and determine the boundary conditions corresponding to the possible channels, which admit the water vapor. In comparison to the measurement, the model developed for mineral wool panels used with the climate of Finland gave a satisfactory result.

5 ACKNOWLEDGEMENTS

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6 REFERENCES

IEA, 1996, Final Report, Volmue 3, Task 3: Material Properties,. IEA ANNEX 24 Heat, Air and Moisture Transfer Through New and Retrofitted Insulated Envelope Parts(Hamtie).

Kumaran, M. K. 1987, ' Moisture transport through glass-fiber insulation in the presence of a thermal gradient.' *Journal of thermal insulation* **10**, 243-255.

Kumaran, M. K. 1988, ' Comparison of simultaneous heat and moisture transport through glass-fibre and spray-cellulose insulations.' *Journal of thermal insulation* **12**, 6-16.

Langlais, C., Hyrien, M. & Klarsfeld, M. 1982, 'Moisture migration in fibrous insulating materials under the influence of a thermal gradient and its effect on thermal resistance', in *Moisture migration in buildings, ASTM STP 77,* eds M. Lieff & H. R. Trechsel, American society for testing and materials, pp. 191-206.

Schwartz, N. V., Bomberg, M. & Kumaran M.K. 1989, 'Water vapor transmission and moisture accumulation in polyurethane and polyisocyanurate foams', in *Water vapor transmission through building materials and systems, ASTM STP 1039*, eds H. R. Trechsel & M. Bomberg. Philadelphia, American society for testing and materials, pp. 63-72.

Wijeysundera, N. E. & Hawlader, M. N. A. 1992, ' Effects of condensation and liquid transport on the thermal performance of fibrous insulation.' *Int.J. Heat Mass Transfer* **35**[10], 2605-2616.

Wijeysundera, N. E., Hawlader, M. N. A. & Tan Y. T. 1989, ' Water vapor diffusion and condensation in fibrous insulations.' *Int.J. Heat Mass Transfer* **32**[10], 1865-1878.

Wijeysundera, N. E., Zheng, B. F., Iqbal, M. & Hauptmann, E.G. 1995, ' Numerical simulation of the transient moisture transfer through porous insulation.' *Int.J. Heat Mass Transfer* **39**[5], 995.

Adapted vapour control for durable building enclosures

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ABSTRACT

Well-insulated building envelope systems are subject to alternating vapour pressure gradients. Therefore the installation of traditional vapour barriers or retarders to avoid interstitial condensation may have undesirable side-effects. Numerous moisture damage cases can be attributed to the fact that a vapour barrier is nearly impermeable in both ways, i.e. it does not allow any dry-out either. Some wall and roof assemblies are only durable if they can dry to the interior side too. The attempt to create a perfect seal is rarely successful and should be better replaced by controlled moisture management. Therefore, the transient hygrothermal behaviour of the building enclosure is investigated and the importance of moisture leaks is discussed.

Recently, adaptable vapour retarding systems have been developed in order to assure a sufficient drying potential. Two of these retarders are presented in this paper. The humidity controlled retarder reacts to local humidity conditions by increasing its vapour permeance when drying conditions prevail. The capillary active retarder relies on capillary suction to remove moisture from the interior of the envelope. By way of several field tests their performance has been evaluated and compared to that of conventional vapour barriers. The results clearly show a faster drying of construction moisture and diminished long-term humidity within the building envelope.

The improved drying potential through the application of adaptable vapour retarders increases the durability of insulated constructions because rot, corrosion and fungal growth are less likely to occur under dry conditions. Because these retarders become more permeable in the case of condensation or high humidity their application must be restricted to buildings with normal indoor air conditions. In the case of the capillary active retarder only non hygroscopic insulation materials may be employed. Otherwise the summer condensate will be absorbed by the insulation layer before it has a chance to reach the retarder and be wicked to the other side. Despite of the limits, it is worthwhile to consider the application of the adaptable retarders in practice because the durability benefits are significant.

KEYWORDS

Vapour retarder Vapour control Drying potential Hygrothermal performance

1 INTRODUCTION

In the past questionable moisture protection paradigms like the sealing of the building enclosure against vapor diffusion processes by vapor barriers or retarders [Rose 2003] have lead to numerous damages, as unavoidable moisture uptake during and after the construction process is not taken into account in practice. Building materials that appear to be dry often contain a considerable amount of hygroscopic moisture that may migrate within the building assembly if subjected to a temperature gradient. The best workmanship cannot exclude some lateral diffusion through embedding elements, like partition walls or floors. In addition there are convective vapour entries through small defaults despite an air tightness of the building fabric that is in total satisfactory. However, real vapor barriers are tight in both directions and do not allow any drying. Even a minor accidental moisture intrusion may therefore cause considerable damage.

That is why builders should look for a better vapour control strategy which is potentially less detrimental in the case of imperfections or unfavorable construction conditions. This paper explains under what circumstances moderate vapour retarders or those with special drying characteristics represent a better solution.

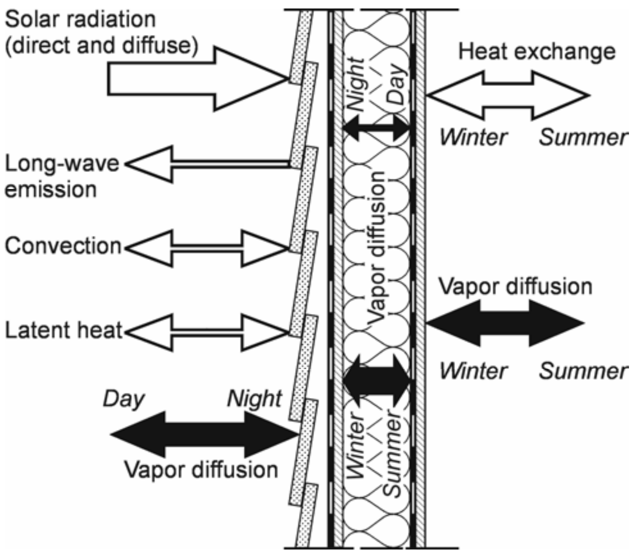

Figure 1. Schematic representation of the hygrothermal effects and their alternating diurnal or seasonal directions in a wall construction.

2 TRANSIENT HYGROTHERMAL CONDITIONS IN THE BUILDING ENCLOSURE

The main function of the building enclosure is the protection of the indoor spaces from natural weather. Besides precipitation and wind that occur only sporadically the solar radiation and the outdoor air conditions are essential. In Figure 1 those hygrothermal parameters and their directions are represented schematically for the example of an external wall. Generally they show diurnal variations at the exterior surface and seasonal variations at the interior surface of the building enclosure. During the daytime the exterior wall surface heats up by solar radiation: this leads to an increase in temperature until there is a balance with the transfer of heat to the interior through thermal conduction and to the exterior through long-wave radiation and convection. Even before sunset when the solar radiation decreases the long-wave (infrared) emission may lead to an overcooling (cooling down below air temperature) of the exterior surface which means that condensation of the ambient humidity may occur.

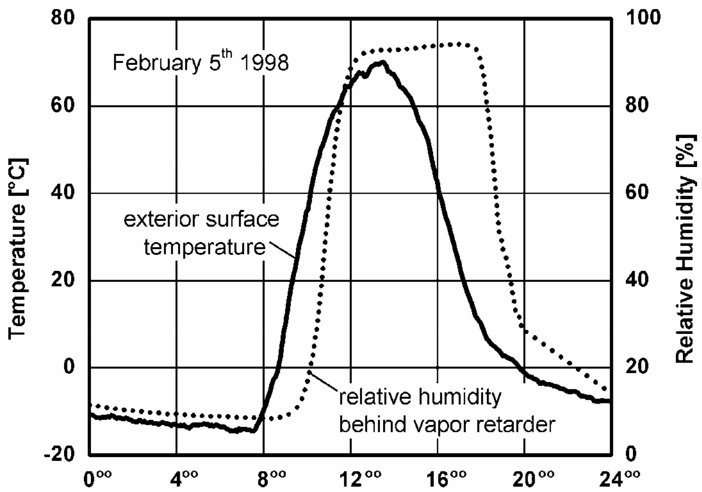

Figure 2. Diurnal evolution of surface temperature and RH between the vapor retarder and the fiber glass insulation measured at a south facing cathedral ceiling during a sunny winter period.

The processes on the exterior surface bear also consequences for the transient temperature and humidity conditions in the construction. When the exterior surface temperature rises during daytime it may cause vapor diffusion out of the exterior layers to the interior side of the wall. The extent of moisture transport to the interior side can be estimated from results in Figure 2 which were recorded in a sheet-metal cathedral ceiling (orientation: south, inclination: 50°). The measured variations of the exterior surface temperature and the relative humidity between the vapor retarder and the mineral wool insulation during a bright winter day show a very large range. Under wintery conditions the temperature of the sheet-metal covering rises from -15°C during the night to 70°C at noon. This strong increase in surface temperature drives the moisture of the wooden sheathing (bearing of the sheet-metal) to the interior side. For that reason the relative humidity between the vapor retarder and the insulation increases with a small delay from less than 10% to more than 90%. During the next night, when the exterior surface temperature falls again below the temperature of the conditioned space the direction of the vapor diffusion flow changes and the relative humidity behind the vapor retarder goes back to its initial state. These experimental results show clearly the diurnal humidity variations that may appear in the building enclosure due to vapor diffusion processes. In general the balance between nighttime and daytime diffusion fluxes results in a seasonal net flux to the exterior side in winter and to the interior side in summer.

The use of vapour barriers in building envelope systems subjected to dynamic hygrothermal loads has caused a lot of problems in the past. Many damage cases originate from the deficiency of such barriers because of inadequate workmanship or a lack of durability. Instead of improving the hermetic sealing the modern moisture protection concentrates on the moisture management of the building enclosure. That means that a limited entry of moisture is accepted when a sufficient and quick drying is assured later on. The admitted quantities of moisture and their presence in the building enclosure depend on the type of load as well as on the materials' capacity of storing moisture. In general, after a characteristic load cycle a building component is not allowed to contain more moisture than in the initial situation. For example the condensation during winter has to dry completely during summer. Infiltrating rain water must drain and dry away before the next precipitation period. Furthermore water that has been collected during a cycle must not exceed a certain limit of acceptance which depends on the moisture tolerance of the building materials in the envelope system. The evaluation of the transient temperature and moisture behavior is rather complicated and requires specific experience. In some cases computer simulations are necessary in order to assess the hygrothermal performance of a construction. Good moisture management can be recognized by its emphasis on the drying potential. The vapour control membranes described below enhance the drying capacity of a construction and may thus lead to a higher tolerance of the building envelope concerning smaller defects or property changes due to ageing.

3 HYGROTHERMAL PERFORMANCE OF THE VAPOUR CONTROL MEMBRANES

The capillary active retarder is a water permeable membrane which is composed of a synthetic fabric sandwiched between staggered strips of polyethylene film. Its brand name “Hygrodiode” is somewhat misleading because liquid water may be wicked through the fabric both ways. With a vapor diffusion resistance of 14 m equivalent air layer thickness (i.e. the vapour diffusion resistance is equal to that of a stagnant air layer which is 14 m thick) it is tighter than most kraft papers but liquid water can penetrate through capillary action via the sandwiched fabric. The membrane has a thickness of 440 µm and weighs 160 g/m². A detailed description can be found in Korsgaard & Pedersen [1992].

The humidity controlled retarder [Künzel 1996] is a nylon-based membrane (PA-film) with a thickness of 50 µm. By absorbing water vapour from the air and thereby opening the molecular pores it changes its vapor permeability with the ambient humidity conditions. Typically, its vapopur diffusion resistance lies above 4 m during the heating season and between 0.1 m and 0.4 m in summer when the assembly should dry out. This can be explained by the difference in ambient conditions at the retarder between winter and summer due to the inversion of the temperature gradient in the assembly, which results in low R.H. at the warmer side and high R.H. at the colder side.

3.1 Laboratory test (cup test)

The vapour permeabilities of the two innovative retarders and a conventional kraft paper were determined by a series of cup-tests. Because the capillary active membrane only becomes more permeable when condensation occurs, a special condensation-cup test was designed. Experimental results from a north oriented pitched roof in Central Europe [Künzel 1999] have shown that the mean roof surface temperature is about 2 K above the indoor air temperature during the summer months.

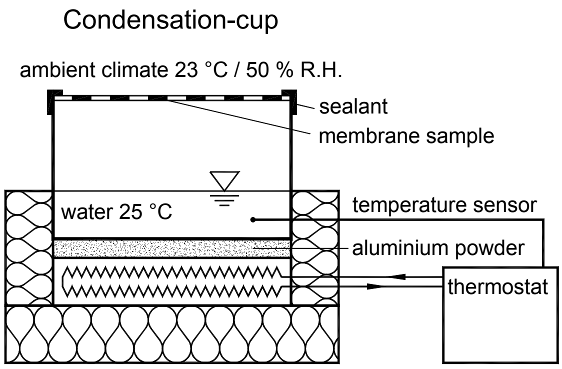

Figure 3. Test set-up for the condensation-cup measurement.

The test set-up in Figure 3 consists of a standard cup containing pure water which is kept at 25 °C by heating the bottom of the cup with a hot plate. This difference to the ambient air temperature ensures sufficient condensation on the bottom side of the tested retarder. The series started with a dry-cup test followed by a wet-cup test and the condensation-cup test. In the end another dry-cup test was carried out with the kraft paper in order to determine whether any irreversible alterations of the material could be detected (past experience has shown considerable degradation of moisturized kraft paper).

The results of the test series are listed in Table 1. The “Hygrodiode” has the highest diffusion resistance under dry conditions. During the normal wet-cup test with pure water some condensation occurs but apparently not enough to get a continuous water film in the fabric. During the condensation-cup test the capillary transport started within 24 hours and led to an apparent vapor diffusion resistance of 0.3 m. The PA-film already becomes very permeable under normal wet-cup conditions (a factor of 15 compared to dry-cup value). But also here the diffusion resistance is further reduced by the conditions in the condensation-cup. The kraft paper also shows a significant reduction of the diffusion resistance under humid conditions. However, the initial diffusion resistance under dry conditions is not regained during the final dry-cup test. This proves that the moisture load led to an irreversible alteration of the paper membrane. More kraft paper samples from different producers were also tested but the results cannot be reported here because the samples either decomposed or developed heavy mould growth in the wet-cup tests. The “Hygrodiode” and the PA-film showed neither alterations of their properties nor any mould growth during and after the test series.

Vapour control membrane	Vapour diffusion resistance [m]		
	dry-cup 23°C / 3% RH	wet-cup 23°C / 100% RH	condensation-cup 25°C / 100% RH
“Hygrodiode”	13.5	2.4	0.30
PA-film	3.8	0.25	0.15
Kraft paper	5.5 (1.2)	1.5	0.21

Table 1. Vapour diffusion resistance of the vapour control membranes measured by cup-tests (conditions in the cup are indicated below) in a climatic chamber at 23°C, 50% RH. The dry-cup diffusion resistance of kraft paper after the condensation-cup test is indicated in brackets.

3.2 Field Test

Laboratory tests can provide useful information about the intrinsic properties and application prospects of new building materials or compounds. To predict their behavior under practice conditions necessitates comparative field tests which should be as close to the building reality as possible. The following field test was carried out in Holzkirchen, a location 680 m above sea level close to the Bavarian Alps. Test object is a habitable roof construction whose detailed description can be found in Künzel [1999]. The considered roof component is an unvented cathedral ceiling with a pitch of 50° oriented to the North. The composition from outside to inside is as follows: Zinc covering, wooden sheathing (initial moisture ca. 40 M.-%), 180 mm mineral wool insulation between the rafters, vapor retarder (“Hygrodiode” / PA-film / PE-film), gypsum board. The moisture of the rafters and the sheathing was monitored continuously by electrical resistance sensors. The test field with the “Hygrodiode” was installed one year after the other test fields by replacing the PE-film. Therefore results from two different summers have to be compared.

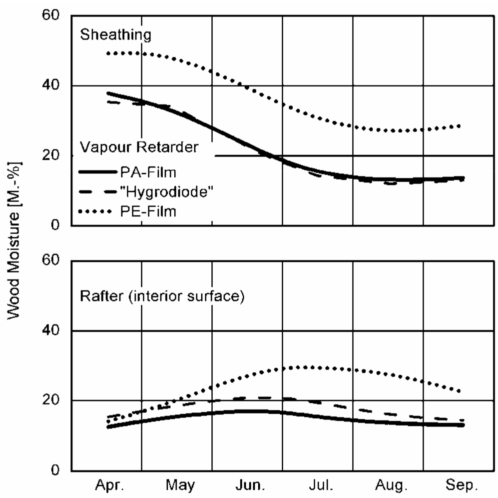

Figure 4. Moisture content of sheathing and rafters in the roof test fields with different vapor retarders during the drying period.

Figure 4 shows the measured wood moisture in the sheathing and at the interior surface of the rafters in the test fields from April to September. Starting from an elevated moisture well above the critical point of 20 M.-%, the sheathing dries under the influence of the sun and reaches uncritical conditions in late June in the test fields with the special retarders. Part of the moisture coming from the sheathing leads to a short term increase in water content at the interior surface of the rafters. But towards the end of the summer the effect of the “Hygrodiode” and the PA-film alike result in dry conditions throughout the cathedral ceiling assembly, while the situation stays critical in the test field with the PE-film. This proves that these new vapour controlling membranes have a clear advantage over the conventional polyethylene retarder in unvented roof assemblies. Similar results have also been obtained previously in the same test roof by comparing the PA-film to kraft paper [Künzel 1999].

3.3 Numerical sumulations

Since the hygrothermal performance of building envelope systems depends heavily on the exterior climate it is difficult to transfer field test results to another climate zone. However, hygrothermal transport models validated by comparison with field tests in one climate zone may be employed to transfer the results to another climate. To this end, several simulation tools are available for the practitioner to assess the performance of vapour control components under any climate condition [Trechsel 2001]. The simulations in this paper are done with a tool commonly used in Europe and North-America called WUFI®. This model has been validated by a number of common exercises [Hens et al. 1996] and by well-defined benchmark cases [Künzel 1995]. The reliability of the model has also been confirmed by independent authors who compared experimental data with model predictions [e.g. Straube & Schumacher 2003, Kalamees & Vinha 2003].

Considered are building enclosure components with 16 cm of mineral fiber insulation, an exterior OSB sheathing covered by a water and vapour tight (diffusion resistance s_d = 30 m) bituminous building paper beneath vinyl siding - res. bituminous membrane when component employed as a flat roof - with a bright colour (solar absorptivity 0.4). The interior finish is gypsum board with a moderate vapour retarder (s_d = 3 m)) underneath (reference case). As locations for these building components, three cities of the northern part of the United States - Boston, Minneapolis and Seattle - are selected. The outdoor climate data (cold years) are taken from the WUFI-ORNL/IBP database [Karagiozis et al. 2001]. The choice of the indoor climate for January is 21°C and a relative humidity depending on the mean outdoor air temperature of 40% (Minneapolis) res. 50% (Boston) or 55% (Seattle). In July the indoor conditions are set to 24°C and 60% RH for all cities. The OSB is initially wet (30% by mass).

The resulting monthly averages of the vapour pressure differences between the OSB and the indoor climate is plotted in Figure 5 for a flat roof and a north facing wall. For both orientations (horizontal and vertical) the vapour drive in Boston, Minneapolis and Seattle points into the building enclosure in January and out of the enclosure into the living space in July. While the vapour pressure differences in January do not vary a lot with the different locations or orientations, there is a comparatively high vapour pressure difference and hence drying potential for the flat roof in July.

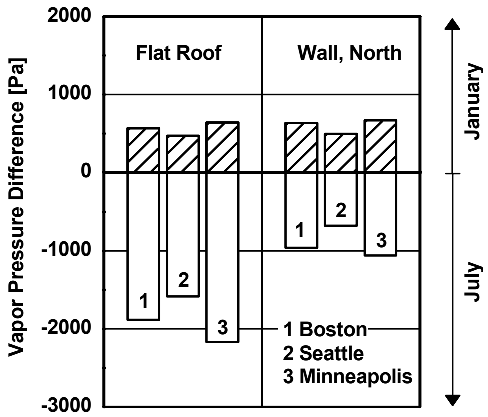

Figure 5. Mean vapor pressure difference between the moist OSB sheathing of the building enclosure and the indoor air in January and July for 3 U.S. cities. Positive vapor pressure differences indicate condensation conditions and negative values quantify the drying potential.

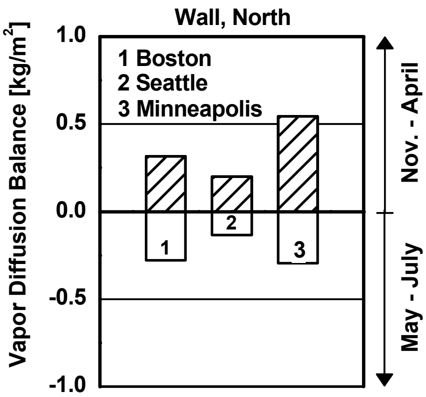

Figure 6. Calculated vapour diffusion balance of the north facing wall with a moderate retarder (s_d = 3 m) showing the maximum moisture uptake by condensation from November to April (hatched area) and the amount of moisture that dries out from May to July.

Compared to the flat roofs the drying potential of the north facing walls is much lower. For the locations considered here, the north facing wall receives less solar radiation than any other orientation which means it also has the lowest drying potential. Therefore it is necessary to examine the hygrothermal conditions more closely. For Boston, Seattle and Minneapolis the moisture accumulation by condensation from November to April (positive values) and the amount of moisture drying out during the period from May to July (negative values) are shown in Figure 6. In all cases, the moisture that permeates through the moderate retarder (s_d = 3 m) into the wall assembly from November to April does not dry out completely until the end of July. Even though the underlying boundary conditions leading to this unfavourable situation are rather severe, they are not unrealistic. Therefore, installing a moderate retarder in this wall assembly cannot be recommended for climate zones comparable to the ones investigated here.

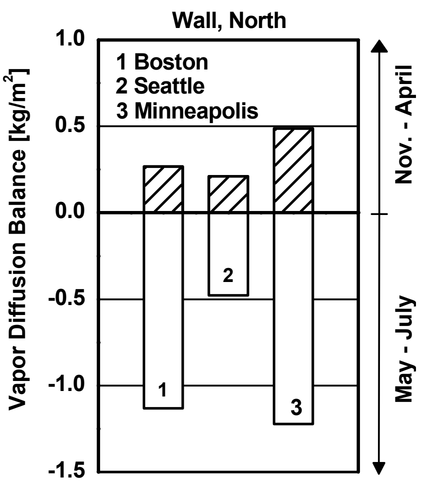

Figure 7. Vapour diffusion balance of the north facing wall with PA-film showing the max. condensation (hatched area) and the amount of moisture, drying out from May to July.

If the moderate retarder (s_d = 3 m) in the reference case of a north facing wall is replaced by the 50 µm thick PA-film the drying potential of the assembly should be significantly increased. Figure 7 shows the results for the same situation as in Fig. 6 only with a different retarder. During the heating period from November to April the amount of interstitial condensation is only slightly reduced by the PA-film because its diffusion resistance under winter conditions is not much higher than 3 m. In all cases the amount of condensate stays below the safety limit of 0.5 kg/m² for interstitial condensation formulated in the German standard on moisture protection (DIN 4108-3) in order to prevent durability problems.

In spring and early summer (May till July) the amount of moisture drying out through the polyamide film into the living space is 2.5 times (Seattle, Minneapolis) res. 4 times (Boston) higher than the amount of interstitial condensation. Because the OSB is initially rather wet (30% by mass) the amount of moisture drying out can be higher than the amount of condensing vapour. That means if the only moisture present in the building assembly results from interstitial condensation the required time for the drying process is shorter. In Boston the dry state will be reached in the middle of June, in Minneapolis end of June and in Seattle in the middle of July. This shows that the climate of Seattle is the least favorable for the building enclosure among the location examined here.

4 CONCLUSIONS

There are a number of ways moisture can enter the building enclosure and interstitial condensation due to vapor diffusion is hardly the most important one. Therefore an adequate drying potential also to the interior is the best way to prevent moisture damage or durability problems. In many cases the use of vapour retarders with a moderate diffusion resistance (sd = 3 m) offers sufficient control of interstitial condensation during the heating period without completely blocking the way out for any unintended moisture in the construction. The laboratory and field tests as well as the numerical simulations have shown that vapour controlling membranes that enhance the drying potential of the building envelope towards the interior are superior to conventional vapor retarders for the durability of insulated structures under Central European and Northern U.S. climate conditions. If the drying potential of a moderate retarder is too small for the design case or additional safety against moisture problems is desired, the installation of special vapour control membranes like the PA-film or the "Hygrodiode" should be considered.

In cases with an extremely high interior moisture load, for example in swimming halls, impermeable vapour barriers should be installed. A polyethylene film (sd ~ 30 m) is hardly ever appropriate because it is not tight enough for an extreme interior moisture load and too tight for most dynamic load conditions (very low drying potential when vapour pressure gradient inverses in summer).

5 REFERENCES

Hens, H. et al. 1996: Heat, Air and Moisture Transfer in Insulated Envelope Parts – Modelling and Common Exercises. IEA Annex 24 final report vol. 1, KU Leuven.

Kalamees, T. & Vinha, J. 2003: Hygrothermal calculations and laboratory tests on timber-framed wall structures. Building and Environment 38 (2003), pp. 689-697.

Karagiozis, A.N., Künzel, H.M., Holm, A. & Desjarlais, A.O. 2001: An Educational Hygrothermal Model WUFI-ORNL/IBP. Proceedings BETEC "Sustainable Build. III" Santa Fe, Oct. 13-18.

Korsgaard, V. & Pedersen, C.R. 1992: Laboratory and Practical Experience with a Novel Water-Permeable Vapor Retarder. Proceedings Thermal Envelopes V, ASHRAE 1992, pp. 480-490.

Künzel, H.M. 1995: Simultaneous Heat and Moisture Transport in Building Components. IRB-Verlag Stuttgart, http://www.hoki.ibp.fhg.de/ibp/publikationen/dissertationen/hk_dissertation_e.pdf.

Künzel. H.M. 1999: Flexible Vapor Control Solves Moisture Problems of Building Assemblies – Smart Retarder to Replace the Conventional PE-Film. Journal of Thermal Envelope & Building Science, Vol. 23, July 1999, pp. 95-102.

Künzel, H.M. 1996: Humidity controlled vapour retarders reduce risk of moisture damages. Proceedings 4th Sym. of Building Phys. in Nordic Countries, Espoo 9/96, Vol. 2, pp. 447-454.

Rose, W.B. 2003: The rise of the diffusion paradigm in the US. Research in Building Physics, A.A. Balkema Publishers, Lisse, NL 2003, pp. 327-334.

Straube, J.F. & Schumacher, C.J. 2003: Hygrothermal enclosure models: comparison with field data. Proceedings 2nd International Conference on Building Physics, Leuven, pp.319-325.

Trechsel, H.R. 2001: Moisture Analysis and Condensation Control in Build. Env.. ASTM MNL40.

Influence of temperature and relative humidity on the durability of mineral wool in ETICS

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ABSTRACT

When an exterior thermal insulation composite system (ETICS) is applied to an outer wall containing build in moisture, the insulating material can be subjected to an increased moisture strain during the drying-out phase. Short-term increases in moisture levels may also arise as a result of driving rain. If high temperatures occur at the same time then they may affect the durability of the insulating material.

This paper aims to predict the moisture and temperature strains who occour in an ETICS under natural climatic conditions at three different locations in northern, middle and southern Europe using computational simulations. By comparing these results with a field test it is possible to determine the maximum hygrothermal loads which arise in the insulating material of an ETICS under European climate conditions. These maximum loads can serve as a basis to review the boundary conditions for durability tests of insulating materials in the laboratory.

KEYWORDS

Durability, ETICS, Mineral wool, pull-off strength

1. PROBLEM

To check the influence of moisture and temperature on the long-term behaviour and pull-off strength of mineral wool in Exterior Thermal Insulation Composite Systems (ETICS) the European Organisation for Technical Approvals (EOTA) proposes two different (alternative) laboratory tests. For both the mineral wool has to be exposed to a high temperature and relative humidity before the pull-off test. In the first one the material has to be exposed 7 days to climate conditions of 70 °C and 95 % relative humidity (RH) and 7 further days at 23 °C and 50 % RH (Northern Test). For the second one the mineral wool stays 5 days above hot water of 60 °C and will then be stored for 7 or 28 days in a vapour tight cup (Water Bath Test). Subject of this paper is, to determine the temperature and RH within the mineral wool layer of an ETICS under different European climatic conditions in Espoo/Finland (northern Europe), Holzkirchen/Germany (central Europe) and Lisboa/Portugal (southern Europe). Furthermore the pull-off strengths corresponding to the laboratory conditions are compared to the measured strengths of mineral wool exposed to the real climate of Holzkirchen in field tests.

2. INVESTIGATIONS

With the help of the well verified simulation tool for the transient heat and moisture transfer WUFI® [1], the hygrothermal behaviour of an ETICS based on mineral wool at the three different locations is simulated. The calculations are done for the west facing façade due to the maximum rain load at this orientation. The build up of the construction is as following: 20 mm exterior mineral render (lime cement plaster), 100 mm mineral wool, 240 mm lime silica brick and 12.5 mm gypsum board at the interior surface. The main focus is to determine the maximum moisture and temperature conditions within the insulation layer. For the exterior lime cement render a water absorption coefficient (A-value) of 1 kg/m2√h is assumed. The corresponding moisture transport coefficients are approximated according to [2]. The other material data are already included to the material database of WUFI. The basic material properties are listed in Table 1.

Construction material	Lime cement render	Lime silica brick	Mineral wool
Bulk density [kg/m3]	1900	1900	60
Heat capacity [kJ/kgK]	0,85	0,85	0,85
Heat conductivity [W/mK]	0,8	1,0	0,04
Porosity [vol. %]	24	29	95
Free saturation [vol. %]	21	25	--
Vapour diffusion resistance factor [-]	25	28	1,3
A-value[kg/m2√h]	1,0	2,7	--
Equilibrium water content at 80 % RH [vol. %]	4,5	2,5	--

Table 1: Basic hygrothermal material properties as used for the simulation

The initial moisture content in the lime silica brick amounts 10 Vol. -% (build in moisture); in the other materials an equilibrium moisture content at 80 % RH is assumed. The heat transfer coefficients are set to 17 W/m2K at the exterior, and 8 W/m2K at the interior surface. The short-wave absorption coefficient of the external plaster is 0.4. For the exterior climate hourly climate data of Holzkirchen, Espoo and Lisboa are used. The room climate varies as a sine curve between 20 °C, 40 % RH in winter and 22 °C and 60 % RH in summer. These values correspond to a normal use as dwelling house. The calculations are carried out over a time period of five years, beginning in October.

3. RESULTS

For validation the results are compared for the location Holzkirchen with measured data of a field test of ETICS in Holzkirchen 1998. In Figure 1 the calculated and measured combinations of temperature and relative humidity 1 cm beneath the exterior surface of the mineral wool (position of the measuring sensor) are plotted. The variation of the calculated values is larger than the measured data – probably due to only hourly steps for the calculation and approximated (not measured) liquid transport coefficients for the external render. Apart from this the correlation between calculation and measurement is satisfactory so that the calculations can be extrapolated to longer periods.

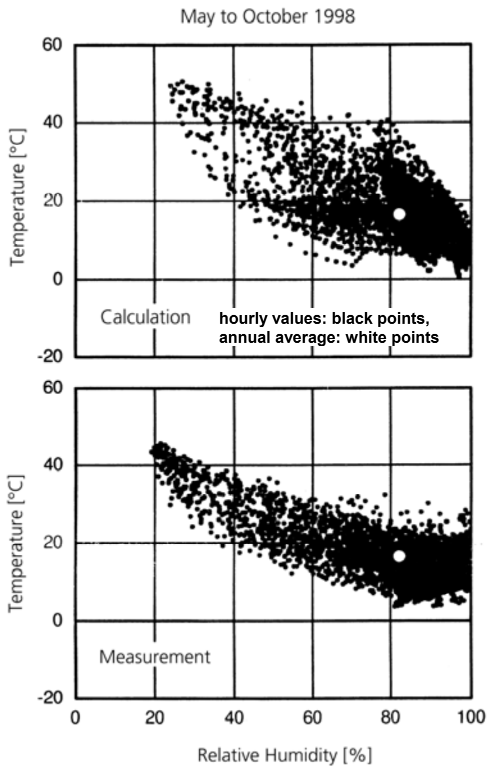

Figure 1: West facing façade at the location Holzkirchen (Germany)
Calculated (top) and measured (bottom) combinations of temperature and relative humidity in the mineral wool 1 cm beneath the exterior surface of the insulation.

Figure 2 shows the variation range and average profiles of temperature and relative humidity in the building component at the three different locations for the first and fifth year of the simulation.

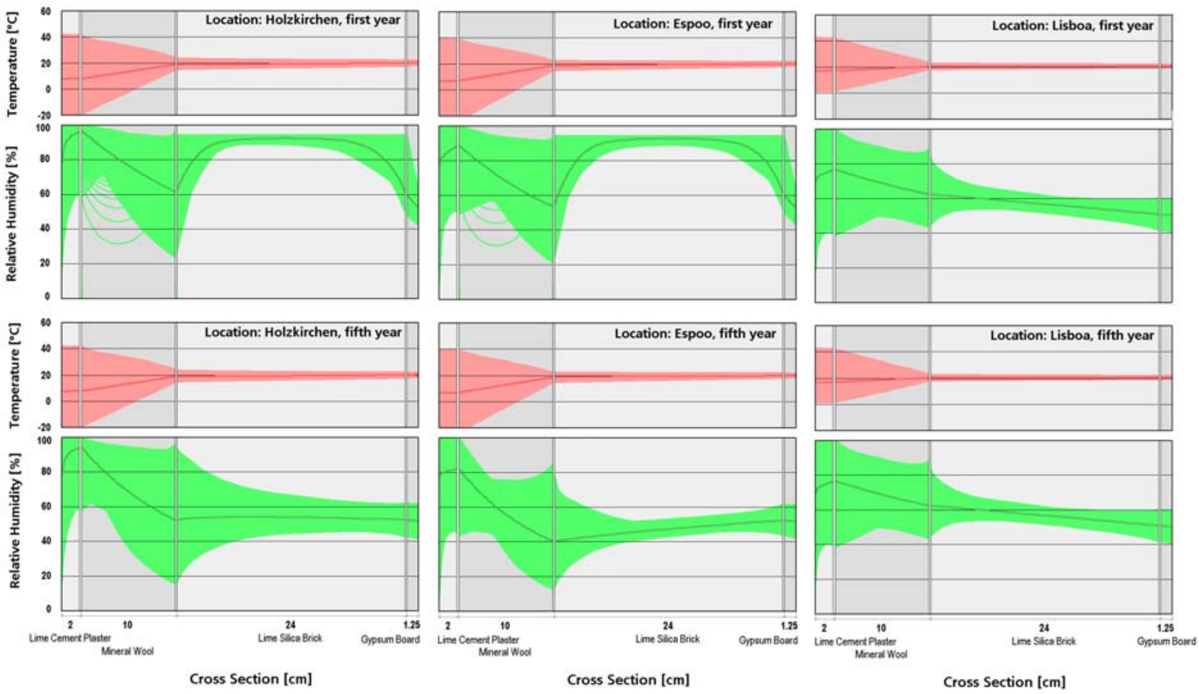

Figure 2: West facing façade at the three different locations Holzkirchen (left), Espoo (middle) and Lisboa (right). Variation range and average profile of temperature (red) and relative humidity (green) in the facade during the first (top) and the fifth (bottom) year of the simulation.

In the first year the dry out process of the lime silica brick can be seen. In the fifth year for every location the dry out is finished and the dynamic equilibrium is reached. At Holzkirchen the relative humidity within the mineral wool is in a range between 25 and 100 % RH for the first, and between 15 and 100 % in the last year. Nearly the same bandwith occurs in Espoo while the level of the relative humidity in Lisboa is higher: between 40 and 100 % RH. For all cases and all locations the highest and lowest values for relative humidity and temperature in the insulation layer arise at the interfaces between the mineral wool and the exterior render and between the mineral wool and the lime silica brick. So the combinations of relative humidity and temperature are plotted for these two positions for the first year (drying out process) and the fifth year (dynamic equilibrium) at every location.

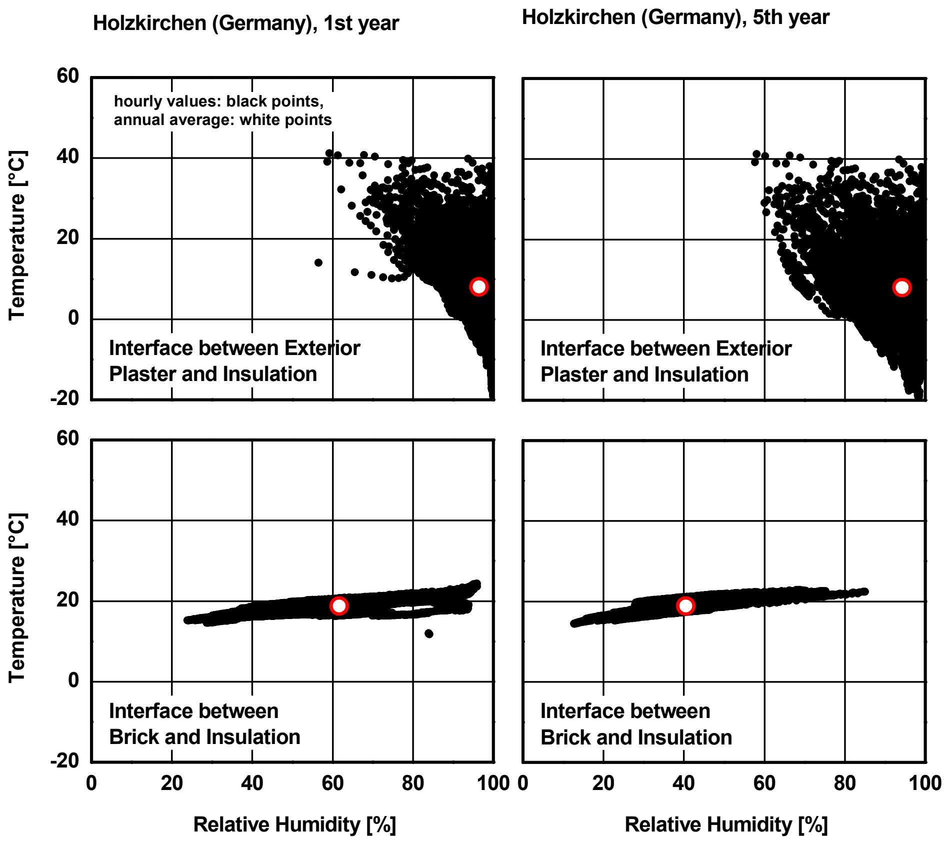

Figure 3: West facing façade at the location Holzkirchen (Germany).
Calculated combinations of temperature and relative humidity in the first (left) and fifth (right) year at two positions in the mineral wool: at the interface between exterior render and insulation (top) and at the interface between insulation and brick (bottom).

Figure 3 (left) shows these combinations for Holzkirchen. In the first year the range of the relative humidity is a little bit higher than in the fifth year while the temperature is nearly the same. The average relative humidity at the exterior of the mineral wool is in the first year at 96 % RH and in the fifth year at 94 % in both cases at a temperature of 8 °C. At the interior the temperature is about 19 °C and the relative humidity in the first year at 60 % RH and in the fifth year at about 40 %.

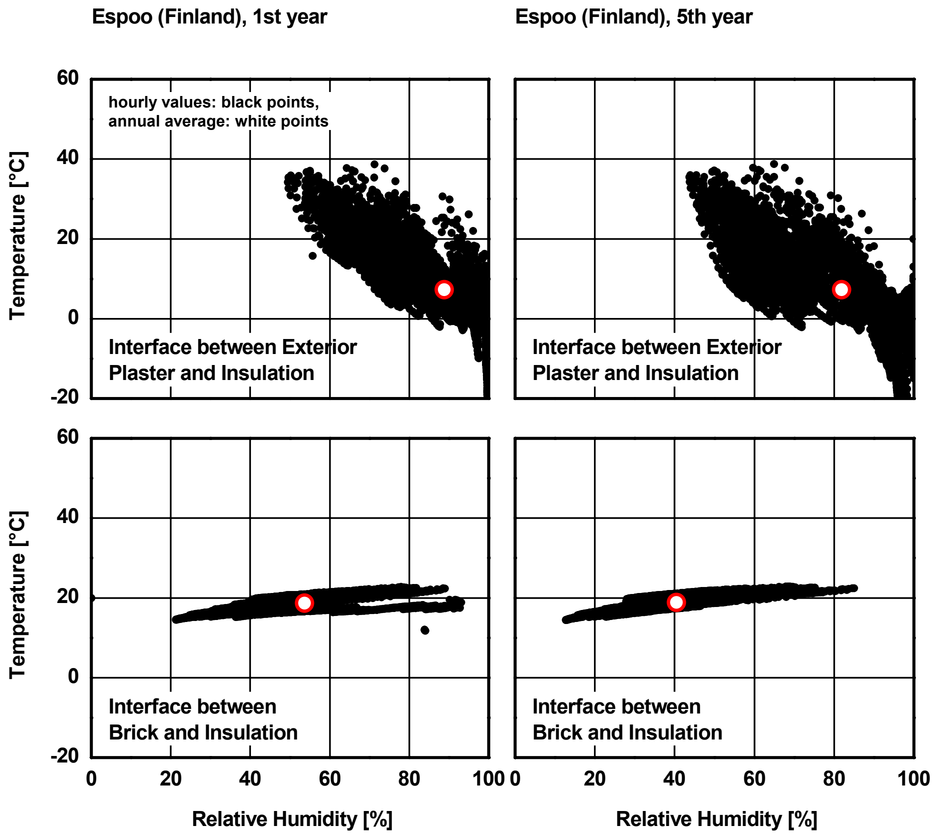

Figure 4: West facing façade at the location Espoo (Finland).
Calculated combinations of temperature and relative humidity in the first (left) and fifth (right) year at two positions in the mineral wool: at the interface between exterior render and insulation (top) and at the interface between insulation and brick (bottom).

In Figure 4 the combinations of RH and temperature are plotted for Espoo. Also here the relative humidity is higher in the first than in the fifth year but on a lower level. At the exterior of the insulation the average value falls from 89 to 82 % RH at about 7 °C, at the interior from 54 to 40 % RH at a temperature of 19 °C like in Holzkirchen. In Lisboa (Figure 5) the average relative humidity at the exterior of the insulation in the first year is even lower at 82 % and at 76 % in the last year. The temperature is at about 17 °C. At the interior the average temperature is at about 20 °C and the average relative humidity falls from 75 to 62 %, this is about 20 % higher than in Holzkirchen and Espoo. Dependent on the room climate conditions the temperature at the interior of the insulation layer at all locations is always in a range of 17 to 23 °C. Relative humidities over 90 % hardly occur. At the exterior interface higher temperatures as well as higher relative humidities are possible but at higher temperatures the relative humidity becomes tendentious lower. So combinations of RH higher than 90 % and temperature higher than 30 °C occur very rarely, temperatures over 40 °C at the same range of relative humidity (between 90 and 100 %) cannot be observed at no location neither in the dynamic equilibrium nor in the drying out phase. Comparing the three different locations the most critical conditions with the highest relative humidity can be observed at Holzkirchen while the drying out phase of the lime silica brick in the first year. So also the field test results for the progression of the pull-off strenght of mineral wool in Holzkirchen can be deemed to be representative for Europe.

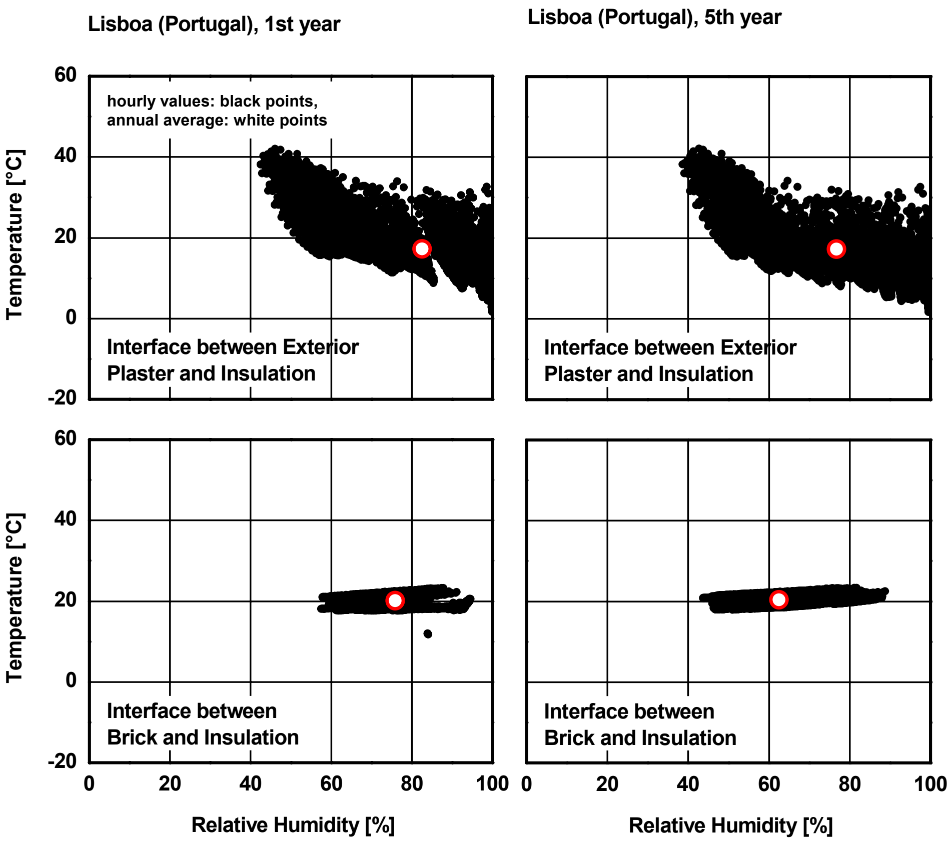

Figure 5: West facing façade at the location Lisboa (Portugal). Calculated combinations of temperature and relative humidity in the first (left) and fifth (right) year at two positions in the mineral wool: at the interface between exterior render and insulation (top) and at the interface between insulation and brick (bottom).

Table 2 shows the pull-off strength of mineral wool after the two EOTA laboratory tests and after 1, 4, 11, 18 and 30 months of natural weathering in a field test at Holzkirchen compared with the properties of the new material. The results are very different for mineral wool standard boards and lamella boards. The standard boards have a fibre direction parallel to the wall surface while the lamella board fibres are perpendicualar to the surface. So the pull-off strength of the standard baord is more influenced by the properites of the binder and the strength of the lamella by the strenght of the fibres itself. Due to this the strength of the lamella board is with a mean value of 89 kN/m² for the new material much higher than the strength of the standard board with only 12.7 kN/m².

At the standard board the pull-off strength is reduced to 37 % after the Northern Test and 44 % after the Water Bath Test. Compared to this the strength after the field test stays clearly higher with a range of 51 % and 79 % of the initial strength. The lowest values occur after 4 months with 51 %, afterwards the strenghts is rising again up to 79 % after 30 months. The strength of the lamella board falls after the laboratory tests similar to the standard board to 35 % (northern test) and 43 % (wather bath test). In the field test the strength seems to be only little reduced with values between 80 % after 4 months and 97 % or 100 % in the further measurements. So the reduction seems to occur due to the strains during the construction process and the drying out phase when a certain level of relative humidity and temperature can be exceeded. With the gradually drying out of the construction the strength seems to increase again. At least for both boards after four months no further reduction of the pull-off strength can be observed.

Mineral wool type	new material	Northern Test	Water Bath Test	after field test [months]				
				1	4	11	18	30
board	100 %	37 %	44 %	67 %	51 %	69 %	61 %	79 %
lamella	100 %	35 %	43 %	-	80 %	100 %	97 %	100 %

Table 2: Pull-off strength of mineral wool after two different EOTA laboratory tests and 1, 4, 11, 18 and 30 months of field test at Holzkirchen (west facing façade, starting in April 1998) compared to the strength of the new material (100 %).

4. DISCUSSION OF THE RESULTS AND CONCLUSION

The results of these investigations show that strongly varying moisture strains can occur in the mineral wool insulating layer of an ETICS. The highest values of relative humidity and temperature can be observed at the interface between the mineral wool and the exterior render – independent on the three examined climate locations. The most critical conditions occur in Holzkirchen. But even here high temperatures (> 30 °C) and high moisture levels (> 95 % RH) at the same time only occur for very short periods, as a high absolute humidity immediately causes a strong diffusion transport into areas with lower absolute humidity. So critical hygrothermal strains in the mineral wool are rapidly diminished due to its high permeability to water vapour.

Stability tests conducted under extreme moisture conditions and temperatures of over 50 °C are not representative for the conditions which can occur in the insulation layer of an ETICS under European climate conditions and lead to significant lower pull-off strengths than under real conditions. The field test results show that the reduction of the pull-off strength occurs in the first month after construction, in the further month no more decrease can be observed. That means that the argument that the laboratory tests represent accelerated weathering needs more profound analysis and the unrealistic high temperature and relative humidity of the tests should probably be replaced by conditions which are more conform to practice. In future the application of hygrothermal simulations could help to specifiy the conditions of accelerated weathering tests in the laboratory.

5. REFERENCES

[1] Künzel, H.M. 1995: Simultaneous Heat and Moisture Transport in Building Components. IRB-Verlag Stuttgart, (http://www.hoki.ibp.fhg.de/ibp/publikationen/dissertationen/hk_dissertation_e.pdf)

[2] Krus, M., Holm, A., Schmidt, Th.: Ermittlung des Kapillartransportkoeffizienten mineralischer Baustoffe aus dem w-Wert. Internationale Zeitschrift für Bauinstandsetzen 3 (1997), H. 3, S. 219-234.

Visual Evaluation of Building Stone Finishing on the View Point of Aging Effect

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ABSTRACT

In this research, the effect of surface properties on the visual evaluation of building stone finishing was investigated. In order to grasp the factors which influence the impression of a stone, a sensory test was performed on specimens of natural stone of various types and surface finishing. For comparison, surface properties such as roughness, 60°gloss and surface colors were measured. The relationship between these surface properties and the evaluated sensory values were analyzed.Factors affecting the visual evaluation, such as roughness, gloss, brightness, naturalness and comfort of a natural stone surface, were analyzed by performing a sensory test using natural stone specimens which varied in stone type, surface finishing method and surface pattern.

Stone specimens were cut out of actual stone. Seven kinds of stone types generally used as stone finishes for the interior or exterior of buildings in Japan were selected. The surface finishing of these stones was changed. The sensory test was performed on 15 stone specimens by 22 subjects, all of whom were architectural students with normal eyesight. The laboratory was arranged such that the stone specimens were illuminated with 500 lx fluorescent light (5000 K). The subjects were asked to evaluate six items, and the results were quantified using the Scaling of Successive Categories method.

Physical values of surface roughness, gloss and surface color were measured and the relationship between them and the results of the sensory tests were considered. It was shown that the visual evaluation of stone specimens is affected by various factors, visual surface roughness is not related to the physical surface roughness, visual gloss is related to the physical value of gloss, visual brightness is related to the value of the surface color, and comfort depends on the stone types, surface roughness, color and pattern.

KEYWORDS

stone finishing, aging effect, visual evaluation, surface properties, sensory test

1 INTRODUCTION

Durability of buildings is an important issue of social demand. For this demand it is necessary to maintain not only structural durability but also visual performance, which pertains to the pleasant appearance of building materials. This is a performance relates to the aging effect for the visual materials deeply.However, currently there is no common evaluation method for the visual performance of building materials, so it is still difficult to find a general standard for selecting finishing materials. Due to its durability, the use of stone has increased as finishing material in recent years. On the other hand, little research has been done on the visual performance of building stone finishing. It is thought that the stone has the aging characteristic.

Kitamura and Isoda (1998) evaluated the appearance of building materials by visual sensation and tactile sensation. Nakayama (1998) examined the deterioration and discoloration of natural stones used for building. However, there does not seem to be a full examination of stone finishing only. In addition to the many kinds of stone types, building stone has some traditional surface finishing, and the visual impression changes with the finish. Moreover, there are other factors which affect the impression of stone, such as colors, patterns and surface forms. These factors influence each other. The purpose of this study is to extract the features and determine the factors affecting the visual evaluation of various stones.

2 SENSORY TEST ON THE IMPRESSION OF STONE

2.1 Outline

In order to grasp the factors which influence the impression of a stone, a sensory test was performed on specimens of natural stone of various types and surface finishing. For comparison, surface properties such as roughness, 60°gloss and surface colors were measured. The relationship between these surface properties and the evaluated sensory values were analyzed.

2.2 Procedure of sensory test

Stone specimens of size 300×300×(20□30) mm were cut out of actual stone. Seven kinds of stone types generally used as stone finishes for the interior or exterior of buildings in Japan were selected. The surface finishing of these stones was changed (Table 1, Table 2 and Table 3). The sensory test was performed on 15 stone specimens by 22 subjects, all of whom were architectural students with normal eyesight (15 men & 7 women who were about 20□25 years old). The laboratory was arranged such that the stone specimens were illuminated with 500 lx fluorescent light (5000 K). The test was conducted under the controlled lighting condition.The subjects were asked to evaluate six items (Table 4), and the results were quantified using the Scaling of Successive Categories method.

Stone	granite1	granite2	granite3	sandstone	limestone	marble1	marble2
Source	Spain	China	South Africa	India	Portugal	Italy	Spain
Included minerals	feldspar quartz biotite muscovite hematite	feldspar quartz biotite muscovite amphibole	feldspar pyroxene biotite amphibole	quartz feldspar mica	stalactite	calcite ($CaCO_3$)	calcite ($CaCO_3$)
Feature	pink spotted with black,gray and brown	whitish gray spotted with black	black	brownish pink	whitish ocher	white with gray band	black with white band
Mark	G1 ○	G2 △	G3 □	S ◇	L ⊟	M1 ◪	M2 ⊞
Variety of rock	igneous rock			sedimentary rock		metamorphic rock	

Table 1. Stone types

Finish	polished	honed	burned	split face	rubbed
Mark	P	H	B	S	R

Table 2. Surface finishing

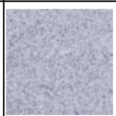
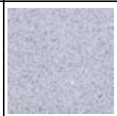
Name	G1□P	G1□H	G1□B	G2□P	G2□H	G3□P	G3□H	
Pattern	spot	spot	spot	spot	spot	even	even	
Munsell HVC	2.5YR7/2	2.5YR8/3	2.5YR7/2	N7.5	N7.5	N1	N4	
Gloss (%)	73.33	2.09	2.13	73.42	2.3	92.67	2.21	
Roughness Ra (□m)	8.6	14.3	252.3	9.1	13.1	6.1	9.5	
Image								
Name	L□P	L□H	S□S	S□R	M1□P	M1□H	M2□P	M2□H
Pattern	even	even	even	even	band	band	band	band
Munsell HVC	2.5Y8.5/1.5	2.5Y8.7/1.5	10YR7.5/1.5	5YR7/2	N8	N8	N1	N4
Gloss (%)	31.96	2.06	0.76	1.26	95.14	11.97	83.13	2.34
Roughness Ra (□m)	8.6	16.9	444.0	24.5	7.0	11.2	7.9	18.8
Image								

Table 3. Stone images and measurement results

Unevenness	even□uneven
Roughness	fine□rough
Gloss	glossy□matte
Brightness	dark□bright
Naturalness	artificial□natural
Comfort	uncomfortable□comfortable

Table 4. Evaluation items

2.3 Measurement of surface properties

Surface roughness was measured by a laser sensor (measurement capacity□1μ□)at 200mm of measurement length and 10 roughness profile was measured 5 times each at intervals for 50mm of lengthwise and crosswise on 200×200mm area of all specimens. The measurements were averaged and the Ra (μm) was calculated.

Surface gloss was measured at 25 points on the surface by a 60°gloss checker and averaged. Surface color was measured at 10 points by image processing (Akanuma et al. 1996) and averaged.

3 RESULTS AND CONSIDERRATIONS

3.1 Evaluation value of stones

Figure 1 shows the results of the evaluation of the stone specimens.

(1) Unevenness

There is a tendency to estimate a polished finish as having little unevenness. For granite1, whose pattern includes minerals of many colors, there was a low feeling of unevenness of the honed finish, and the burned finish had a high feeling of unevenness. The polished finish and honed finish were also estimated to have evenness for granite3, which was different from the two other kinds of granite because its mineral patterns were not clearly recognizable. Therefore, surface pattern was considered to affect the evaluation of the feeling of unevenness.

The feeling of unevenness of the split face of the sandstone was the highest of all specimens. For limestone, the feeling of evenness of both the polished and honed finish was lower than that for granite1 and granite2. This was considered to be the influence of the low gloss of the specimen. For marble, it was estimated that the polished finish of white marble1 did not have unevenness, and the honed finish had a little unevenness. For black marble2, it was similarly estimated that the polished finish did not have unevenness and the honed finish had a little unevenness.

(2) Roughness

All three specimens of granite1 were estimated to be rough. The order from least to most rough was polished, honed, then the burned finish. For granite2, which had a pattern of gray spotted with black mineral, the polished finish was fine and the honed finish was estimated as a little rough. Although the same tendency was seen also for granite3, the honed finish had a feeling of roughness higher than that for granite2. For sandstone, the feeling of roughness of the split face was as high as the burned granite1, which had the pattern of many colors.

(3) Gloss

In the evaluation of the feeling of gloss for all specimens, there was a tendency to estimate that the polished finish with a measured high gloss was glossy. For granite1, it was estimated that the honed finish and burned finish were matte, and the polished finish was glossy. For granite2, the polished finish was estimated to be glossy. Moreover, the highest estimations of the feeling of gloss were for the polished finish of granite 2 and the polished finish of marble1. For sandstone, it was estimated that the split face and rubbed finish were matte; this kind of stone could not easily attain a glossy finish. Although it has also been said that limestone cannot have a glossy finish compared with granite or marble, it was estimated that the polished finish had a little gloss and the honed finish was matte. As stated above, the polished finish for white marble1, which had the highest measured gloss, matched the high evaluation of the feeling of the gloss of the polished finish for granite2.

(4) Brightness

There was a tendency to estimate that stone of high luminosity was bright. Granite1 and granite2 had high luminosity and, for the feeling of brightness, were estimated to be bright. It was estimated that the sandstone was dark, which could be influenced by the low chroma of the two specimens. Moreover, the split face was estimated to be darker than the rubbed finish. For marble1, the two specimens were estimated to be bright. It was estimated that the polished finish was the brightest of all specimens, and the honed finish was a lower brightness, like that of granite and limestone. It is estimated that marble2 was darker than granite3, whose luminosity is of the same grade, and it was thought that surface patterns instead of brightness, and so on, had influenced the feeling of brightness.

(5) Naturalness

The tendency to estimate a polished finish as artificial was seen in all of the stone specimens. Granite1 and granite2 were estimated to be artificial, and the polished finish was estimated to be more artificial than the honed finish. It was estimated that the burned finish was a little natural, and it was thought that surface unevenness was an influence. It was estimated that the polished finish of granite3 was the most artificial of all specimens.

The honed finish was also estimated to be a little artificial, and it was thought that the even black surface color of this specimen was an influence. It was estimated that the split face of sandstone was the most natural of all specimens, and it was thought that the rolling surface of this specimen was an influence. The rubbed finish of sandstone was a little artificial. For the limestone, it was estimated that the honed finish was natural, but the polished finish was a little artificial. The polished finish of limestone, which could not easily have a glossy finish, was estimated as not artificial compared with the polished finishes of granite and marble, which were also considered glossy. For white marble1, the polished finish was artificial and it was estimated that the honed finish was a little natural. The polished finish and honed finish were estimated to be artificial for marble2, even with the black surface.

(6) Comfort

The evaluation of comfort was high for all seven types of stones. However, for granite1, the polished finish was estimated to be a little uncomfortable. It was thought that, because the granite1 surface had a pattern consisting of many colors and the polished finish was flat, smooth and glossy, the evaluation of comfort was low.

Although both kinds of methods of finishing were estimated to be comfortable for granite2, the evaluation of the honed finish was higher than that of the polished finish. It was thought that the flat and smooth matte finish would make a comfortable evaluation for granite2, which was colorless (gray) spotted with black mineral. For granite3, a different tendency from the other stone was shown, and the difference in the evaluation due to finish was clear. It was estimated that the polished finish was the most comfortable compared to all specimens. Moreover, it was estimated that the honed finish was the most uncomfortable of all specimens. For sandstone, the evaluation of the comfort of the split face was higher than that of the rubbed finish. For limestone, the evaluation of the comfort of the polished finish was higher than the honed finish. For marble1, unlike sandstone and limestone, it was estimated that the honed finish was more comfortable than the polished finish. The evaluation of comfort was high for marble1, as also seen for granite2, because the surface color was colorless and the polish had a high feeling of gloss and high luminosity. For marble2, the same tendency as sandstone and a limestone was seen, and the polished finish was estimated to be more comfortable than the honed finish. Since the tendency was different in granite3, which was the same color as granite2, the influence of factors other than surface color were considered in the evaluation of comfort.

3.2 Relationship with each evaluation item

3.2.1 Relationship between roughness and each item

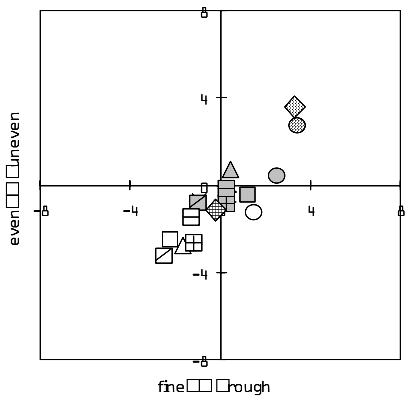
Fig. 2-1 shows the relationship between roughness and each item. There is a correlation between the feeling of unevenness and roughness. Four specimens were estimated as rough and uneven: G1-B, S-S, G1-H, and G2-H. The polished finish was fine and even. The tendency of the estimate to be rough due to the matte surface finish was seen. However, brightness was estimated when the surface was white, and the M1-H with matte surface was fine, and a little dark. Although there was a tendency of the estimate to be fine and glossy for a polished finish, G1-P was evaluated as rough and glossy; this is thought to be due to the influence of the stone's surface pattern.

The relationship between roughness and brightness was indefinite, but a difference in the evaluation by stone type and finish was shown. White M1-P and M1-H are estimated to be fine and bright, and the evaluations of both specimens were considered to be influenced by surface color.

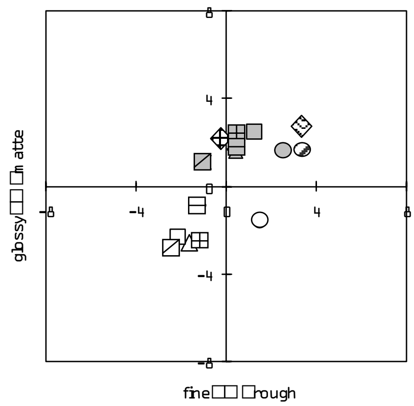
Although a variety of stone types was shown to have naturalness, there was a corresponding relationship between roughness and naturalness. There was a tendency for the evaluation to be artificial if the surface was fine, and it was natural if the surface was rough. The polished finish was fine and artificial. G1-P and G1-H were rough and artificial.Both of these specimens had a spotted surface pattern and were flat and smooth. G1-B and S-S were rough and natural, and the surface unevenness of these specimens was clearly recognized. Although the relationship between roughness and comfort was not clear, a difference in the evaluation by finish was shown. G1-H, G1-B, and S-S were rough and comfortable. On the other hand, G1-P and G3-H were rough and uncomfortable. It turned out that for granite1 with its surface pattern, the evaluation of the honed finish and burned finish became comfortable, but the polished finish became uncomfortable.

3.2.2 Relationship between comfort and each item

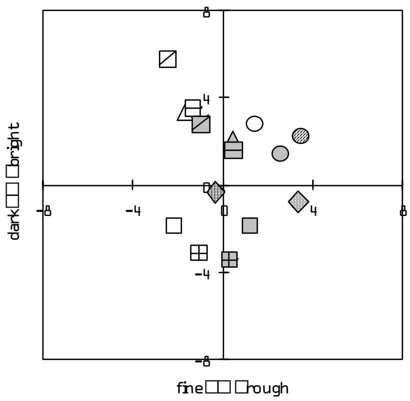
Fig. 2-2 shows the relationship between comfort and each item. The tendency for comfort to increase was seen, so that it was estimated to be due to unevenness for granite1. G1-P with a pattern ,G3-H with no pattern and flat were even and uncomfortable. When considering the stone types, it could be that the evaluations of comfort of polished finish and honed finish differ. Although the relationship between comfort and gloss was not clear, the polished finish was estimated as glossy and there was a tendency to estimate the other finish as matte. Some stones were estimated to be comfortable without the recognition of gloss. So, the combination of stone types and finish is important in evaluating the comfort of stones. The influence of brightness on the evaluation of comfort was not clear. Although the two granite3 specimens were estimated to be dark, their evaluations of comfort differed. For stones other than granite3, it was shown that naturalness corresponds with comfort. As a notable example, the polished finish of granite1 was artificial, and the evaluation of naturalness and comfort was higher for the honed finish, followed by the burned finish. Although the sandstone split face was the most natural and comfortable, the rubbed finish was artificial and only a little comfortable.



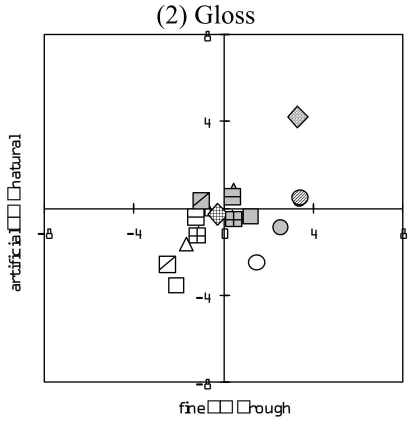
(1) Unevenness



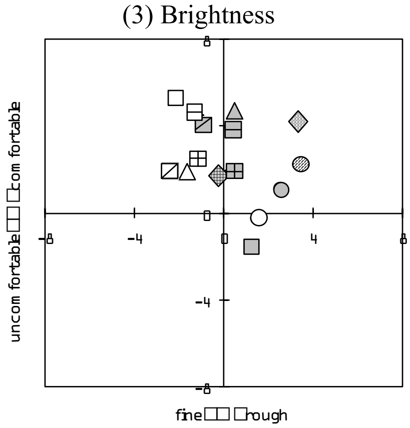
(2) Gloss



(3) Brightness



(4) Naturalness



(5) Comfort

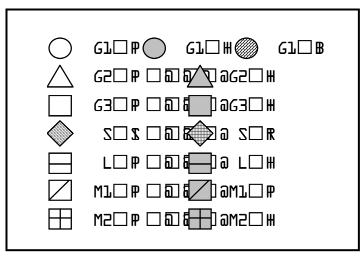
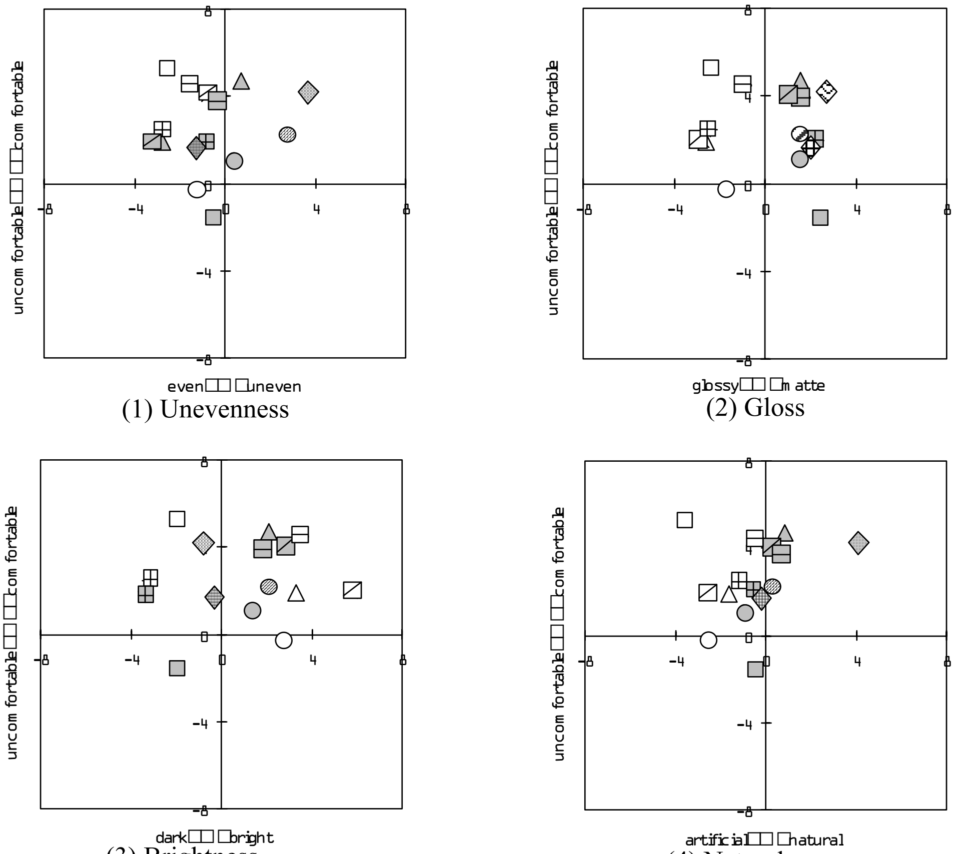


Fig.2-1 Relationship between roughness and each evaluation items



(1) Unevenness (2) Gloss

(3) Brightness (4) Naturalness

Fig. 2-2 Relationship between comfort and each evaluation items

3.3 Correspondence of physics and feelings

Fig. 3 shows the relationship between the surface roughness Ra and the evaluation of the feeling of roughness. The relationship between the evaluation of the feeling of roughness and the measured surface roughness Ra was not clear, because the result was biased for unevenness and flat and smooth specimens. Although the G1-B and S-S evaluation of the feeling of roughness was the same, their Ra values were different.

The Ra value was concentrated on 20 μm or less of the flat and smooth section of the specimen, but the evaluations of the feeling of roughness differed from each other.

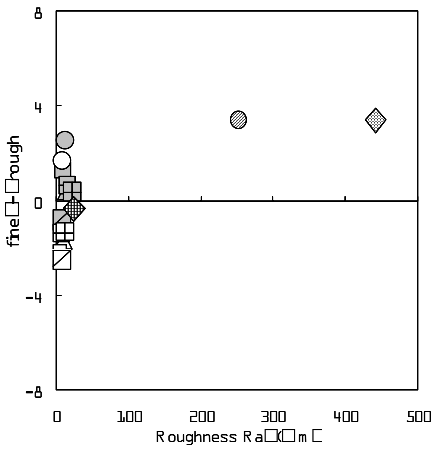


Fig. 3 Relationship between surface roughnesses Ra and evaluation of the roughness

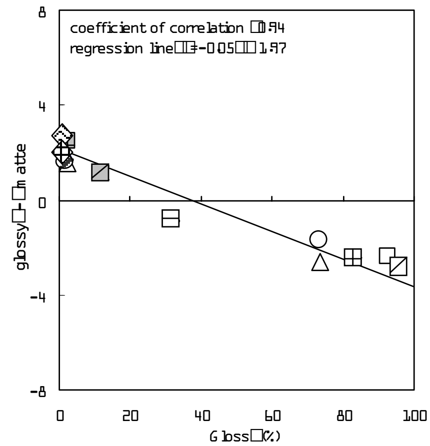


Fig. 4 Relationship between Gloss and the evaluation of gloss

Fig. 4 shows the relationship between gloss and the feeling of gloss. Although the measured gloss of a matte surface specimen was low, there was little correspondence between the feeling of gloss and gloss. If the measured gloss was high, it was easy to be recognized as glossy. The specimen of a polished finish was high gloss, and it was recognized as glossy. The gloss of L-P, which was considered to be stone that could not easily have a glossy finish, was 31.96%, and the gloss of other specimens with a polished finish was 70% or more. Because of the correspondence of the feeling of gloss and low gloss, the feeling of gloss can be estimated as also a little glossy.

4 CONCLUSIONS

The conclusions of this study are as follows.

(1) In the subjects' impression of stones, the factors which affected their evaluation were the patterns, stone types, finishing, and surface color. In particular, surface finishing was a major factor in the evaluation of unevenness, gloss, and naturalness.

(2) The evaluation of unevenness was influenced by the surface finishing method, and it was estimated that a polished finish had little unevenness. It is thought that not only the method of finishing affected the evaluation of the finish for unevenness, but also the stone type, pattern and color were influences. There was little correlation between the surface roughness Ra and the evaluation of the unevenness.

(3) The evaluation of gloss showed a correlation with gloss. The results show a close relationship between the evaluation of brightness and luminosity, and the surface pattern also had an effect on the evaluation of brightness.

(4) Comfort was considered to be influenced by various factors. Some stones had unevenness with a comfortable finish and others were flat and glossy with a comfortable finish.

5 REFERENCES

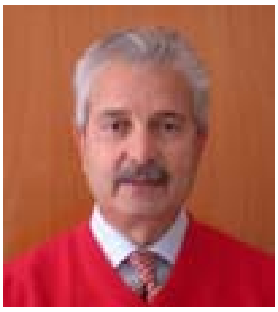
Okuda, S. & Honda, S.1968, 'A study on gloss of building finishing materials,' Summaries of Technical Papers of Annual Meeting AIJ, Japan, October.1968, pp.185-186.

Kitamura, K., Isoda, N.& Yanase T.1998, "Interpretation of the evaluation scales for appearance, and a quantitative analysis of appearance using simple texture-Evaluation of the appearance of building materials by visual sensation and tactile sensation (Rept.1)," J.Archit.Plann.Environ.Eng.,AIJ,Japan,September.1998, No.511, pp69-74.

Nakayama, M. 1998, "Investigation of deterioration associated with natural inherent in stones□ Consideration of deterioration which occurs stone surface by onsite surveys□," J.Struct.Constr. Eng., AIJ, August.1998, No.510, pp7-14.

Akanuma, R., et al. 1996, "Measurement of color distribution for building facades by using image processor," Summaries of Technical Papers of Annual Meeting AIJ, Japan, Sptember.1996, pp.1215-1216.

The Durability Evaluation for Sandwich panels: First Experimental Results



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ABSTRACT

The assessment of durability of building components and products is the first step towards the knowledge of the service life of building. In the circle of a national research (P. R. I. N.) on the durability coordinated by Prof. P. N. Maggi which involves different Italian Universities, the Research Unit of Palermo, are carrying out a study on discontinuous roofings sub-class.
This research consists of a theoretical part and an experimental part which follows the procedure of the standard ISO 15686. At the moment, the first theoretical phase regarding the functional analysis and the evaluation of reliability of the sub-class is being completed. A technical solutions repertoire of discontinuous roofings has been created.
In this circle, a study is being carried out on the behaviour during time of the sandwich panel, a component used as an element of roofing and of internal or external partition. This component in recent years has been the object of an intense normative activity and of research for the assessment of durability.
The research is conducted on panels consisting of two metal skin faces and polyurethane core, commonly used in residential and industrial buildings.
Two types in different colours have been considered (the outside face of the panel): one light and one dark. This will investigate the different capacities of absorption and how they may influence the behaviour of the component.
The actual phase of the research is developing the following activities:

- the characterization of samples through laboratory tests which serves to evaluate their mechanical and energetic characteristics;
- the collection of data on the climatic context of Palermo;
- the external exposure of the above-mentioned samples;
- the monitoring of thermo-hygrometric parameters of the outside samples.

Therefore, as can be seen, the research is being developed according to guidelines of the chosen methodology and in this paper the results so far will be shown.

KEYWORDS

Durability, Reliability, Sandwich panel, Laboratory evaluation

1 INTRODUCTION

The ISO 15686[1] which presents a methodology for the assessment of durability, predicts the functional analysis of technical element in exams; the characterisation of materials or components in relation to functional characteristics for investigation, the identification of agents and actions that can cause degradation and performance decay; the collection of data to reconstruct the climatic context in which the components are found during their life; the assessment of the logic of aging and the modality of failure through accelerated aging tests; the comparison of long term exposure tests based on the real conditions of the stress.

The assessment of durability in the building is complex for the different terms of technological aging of parts which form the system. For this reason the study is conducted by:

- class of technical elements;
- technological performances significant of the class;
- reference to environmental conditions and use of components.

The fundamental parameters for the assessment of the durability are reliability and natural durability. The first is assessed by a theoretical method, which has worked out by BEST of Polytechnic of Milan based on the examination of the component project, through a functional and technical analysis, which consents to estimate the tendency of the reliability of the natural durability in qualitative and relative terms. The propensity of reliability is attributed by comparing elements of a repertoire of technical solutions of the same class, considered out of system and with no intended use. The natural durability is assessed by means of laboratory and outdoor aging tests.

2 THE CLASS OF DISCONTINUOUS ROOFINGS

The principal function of the roofings is to contribute to the realisation, of the inside of building, of conditions which occur in human activity through light, acoustic and air flow control. The morphological classification separates the roofing on the basis of continuity or discontinuity of the water-tight element, determining two classes which present differences in formal, functional and technological characteristics.

The object of our research is the class of the discontinuous roofings. In general, characterised by a slope which consents the downflow of water, excluding the possibility of infiltration in spite of the discontinuation on the watertight element, which can be achieved with elements of two types:

- tiles and plates in small elements ;
- tiles and plates in large elements ;

The roofing is presented as a complex system of more functional elements, each one of these has specific performances. The sum of these functions determines the behaviour of the roofing in regard to the performance which it should supply (watertight, thermal insulation, mechanical resistance, etc.). Today manufacturers tend to achieve packets of stratified roofing which manage to absolve different functions, as well as for sandwich panels and multilayer sheets.

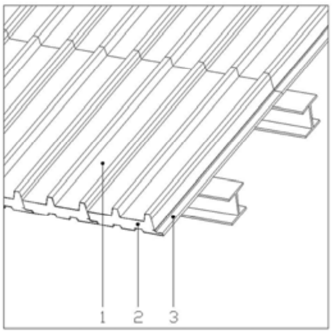
A repertoire of technical solutions has been constructed in the basis of the following criteria: heterogeneous solutions, also representative of local uses (for use, nature or materials used, etc.); presence of “innovative” solutions to study the characters in relation to “traditional” solutions.

The repertoire is composed of twenty-four technical solutions [Alaimo 2003], individualised by:

- typology of watertight element ;
- nature of load bearing structure ;
- presence of insulation and ventilation layers.

In the repertoire, for the experimental phase to evaluate the natural durability, we have considered sandwich panel which is the seal in the technical solutions DR21 “Sandwich panel on discontinuous loadbearing structure“ and DR22 “Sandwich panel on prestressed concrete loadbearing structure“ ‘Fig.1’.

[1] Sent by completion of work by the committee TC 59/SC14 from the International Standardization Organization.



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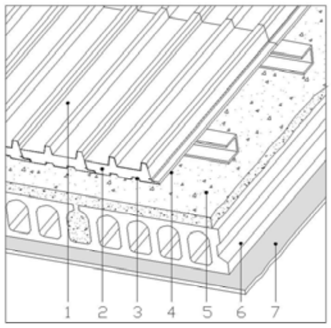


DR 22

Figure 1. Technical solutions DR21 "Sandwich panel on discontinuous loadbearing structure" and DR22 "Sandwich panel on prestressed concrete loadbearing structure".

3 ASSESSMENT OF RELIABILITY

The method for the assessment of reliability [Rejna 1995] is based on analysis of four components:

- the functional reliability which through functional analysis characterizes the distribution of functions inside of the solution and thus the degree of fatigue of the element throughout it's functioning ;
- the inherent reliability which concerns the possible dimensional variations, which may afflict the component as a result of context stress ;
- the executive reliability, which through objective analysis of technical solution verifies the degree of precision achieved with respect to predictions of the project ;
- the critical reliability which concerns the possible physical-chemical incompatibility between adjacent materials of a different nature from the technical solution.

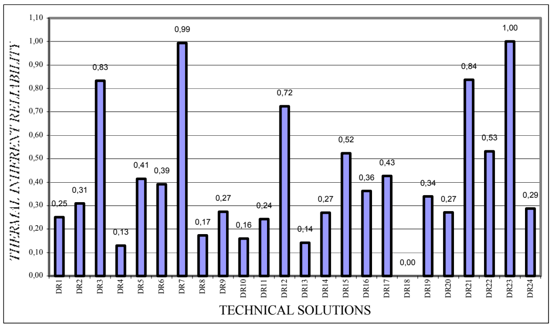
The relative results of the study of reliability have already been presented [Alaimo 2003].
In these seats the results of the study concerning of inherent reliability are explained.
For each technical solution every single functional element was analysed, in the directions X (size), Z (length) and Y (thickness).
For every functional element the following were identified:

- the number, the technological characteristics and dimension of constituent elements;
- the technological characteristics of joints and interfaces;
- the values of thermal and moisture dilatation coefficients of all constituents elements;
- the values of the moment of inertia of the rectangular section of all the interfacing elements.

Therefore, the propensity of the inherent thermal reliability (A_T) and the inherent moisture reliability (A_U) of the correspondent repertoire solutions were calculated.
The propensity of the inherent global reliability (A_i) was calculated as an arithmetical mean of the two previously calculated reliabilities.
For every solution a summary was given, which reports:

- the score of the inherent reliability predicted for the solution;
- the mean score of the inherent reliability of the repertoire;
- the parameters of assessment and the relative values for the layers of the solutions in directions X, Z and Y.

Three histograms have been created to compare the propensity of the thermal inherent reliability and moisture, which allow the comparison of the propensity reliability of each solution in absolute terms 'Fig. 2'.



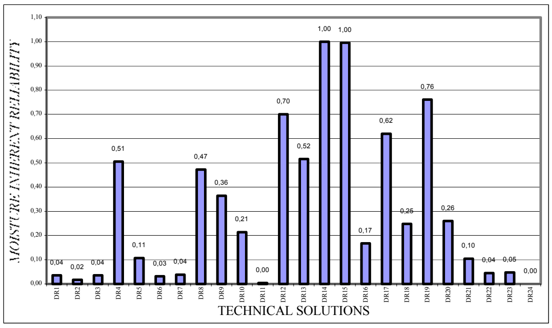


Figure 2. Propensity inherent thermal and moisture reliability of technical solutions.

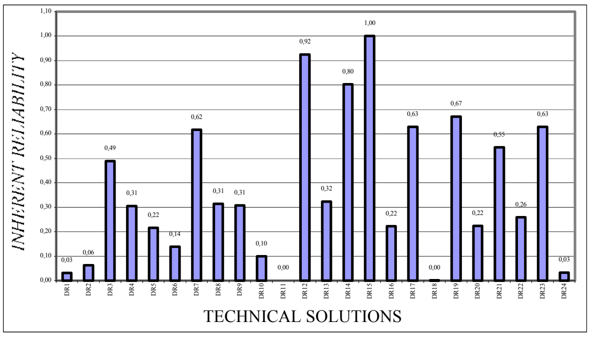


Figure 3. Propensity inherent reliability of technical solutions.

The last histogram shows the values of the propensity of inherent global reliability of each technical solution ‘Fig. 3’.

- in general, the solutions of the repertoire present a value of mean thermal reliability (0.41) greater than value of mean moisture reliability (0.30);
- the propensity of the inherent thermal reliability has little dependence on the type of load bearing structure;
- the propensity of the inherent moisture reliability is favoured in the presence of efficient ventilation layers (DR12, DR14, DR15, DR17, DR19);
- the solutions with discontinuous load bearing structure (DR12, DR14, DR15, DR17, DR19, DR21, DR23) present an elevated inherent reliability, with respect to the average (0.37) to the repertoire;
- the maximum inherent reliability propensity (1.00) is to the solution DR15 “Roofing tiles on discontinuous load bearing structure, ventilated by discontinuous inferior insulation”, thanks to the elevated scores of the inherent thermal reliability in thickness (direction Y) and inherent moisture reliability in all three directions;
- The solutions DR21 (0.55) and DR22 (0.26) show different values for the different type of load bearing structure (discontinuous or continuous).

4 THE SANDWICH PANEL

For the assessment of natural durability we can study the behaviour of sandwich panel, seal of some technical solutions of the repertoire. It’s made of two prepainted galvanized steel skins (thickness of 0,45 mm), connected by a polyurethane foam core (thickness of 40 mm) which carry out the functions of thermal insulation. This component has been largely used in residential and commercial buildings,

leisure and industry services, especially for roofings[2]. It's widespread use is mainly owned to its lightness, self-supported ability, cheapness and good insulation ability.
In recent years, the study of the durability of this building component has resulted in research arguments of great interest, which involve Universities, manufacturers, certification bodies, users etc.

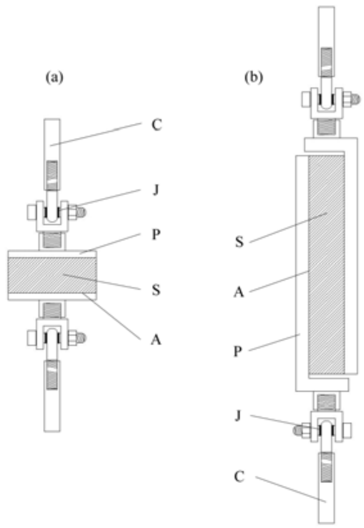
5 CHARACTERISATION OF SAMPLES

Before aging, characterisation tests were made with samples of the panel differing in dimension according to the type of test. The samples were directly supplied by the producers[3], and have dimensions of: 100 x 100 mm, 250 x 100 mm, 1000 x 100 mm and are two colours: light and dark. Tests are destructive and non-destructive. Destructive are tests to determine the resistance to tensile stress, shearing stress and bending stress; non-destructive are tests to analyse the superficial aspects and colour, the loss of mass, the conductivity measure and thermal resistance.

5.1 Tests of mechanical characterisation

Three types of tests are planned on three samples for each type, to evaluate the resistance to:
- tensile stress of samples with a dimension of 100 x 100 mm (according to the standards ETAG 016-1:2003; UNI EN 1607:1999)
- shearing stress of samples with a dimension of 250 x 100 mm (according to the standard UNI 8070:1980)
- bending stress of samples with a dimension of 1000 x 100 mm (according to the project of the standard PrEN 14509)

From the results obtained by the singular measurements, the mean value was calculated which is going to be compared with the results obtained from similar tests of aged samples to assess performance decay.
The tests were carried out in the laboratory of the department of Structural Engineering[4] at the University of Palermo. To carry out the tensile and shearing tests it was necessary to achieve the specific systems of loading comprehensive of the metallic plates to which samples have been glued[5]. The realised devices are shown in figure 4.



Key :

A = Adhesive
S = Specimen
P = Plate of steel
C = Coupling box
J = Ball joint

Figure 4. Devices used for tensile test (a) and shearing test (b).

[2] Italy is the biggest European producer and exports around 30% of this production.
[3] The samples were supplied by AIPPEG, Associazione Italiana Produttori Pannelli Grecati, for which we thank.
[4] With the assistance of Prof. A. Failla.
[5] The adhesive used to glue the samples to the plates is "Sikadur 30", which was kindly supplied by SIKA, and is a paste for structural glueing.

After the labelling of samples the protective film was removed from the surfaces. The adhesive product was applied by a fine spatula to obtain a uniform thickness of around 3 mm for each part. Thus proceeded the gluing of the supports.

Tensile test: a mean value of resistance to tensile stress (of 1.56 Kg/cm^2 for light samples, 1.31 Kg/cm^2 for dark samples) was relieved. The failure of samples, in general, come from the core and sometimes also as a results of the detachment of this from one of the skins 'Fig. 5'.
Shearing test: a mean value of resistance to shearing stress (of 1.72 Kg/cm^2 for light samples, 1.38 Kg/cm^2 for dark samples) was relieved. The applied force caused the failure of shearing of the polyurethane foam 'Fig. 6'.
In both cases the values of resistance obtained are in line with results (in waiting). The shearing module will be determined on the basis of bending tests on four points for which a relative loading system will be prearranged.

Figures 5 and 6. Mechanical characterisation tests: modality of failure of samples.

6 DISPLAY OF EXTERNAL SAMPLES

Since March 2004, some external batteries of different colour and size samples have been exposed to weather. These are placed on two metal racks and have a variable inclination of 0° to 45° appropriately realized 'Fig. 7'. Both racks allow the display of up to twelve samples of different types.

Figure 7. Apparatus for the display of samples.

The samples are displayed with facings oriented to the South where there is maximum sunshine. Before display the samples were protected at the edges[6] with the aim of protecting the core of attack of external agents.

7 REGISTRATION OF THERMOIGROMETRIC PARAMETERS

Since March 2004, monitoring of exposed surface temperature of samples in the shade has been carried out by means of micro-recorder of temperature[7]. Such instruments, which operate in the range of -40°C +85°C, were programmed to reveal temperature values every hour 'Fig. 8'.

Figure 8. Samples with sensors for temperature registration.

From the monitoring the following data was revealed:
- maximum daily temperature;
- minimum daily temperature;
- maximum daily thermal excursion.

The collected data was elaborated, distinguishing between the values of exposed facings and those for the facings in the shade.
The data concerns the period from March to August 2004, and provides a first indication of real trends of temperature in stress conditions of the climatic context of Palermo.
In the light of what, we can see in the graphs it is possible to make the following observations:
- the maximum temperature relieved in the exposed face is 63°C on the days of 6th July, 7th and 20th August 'Fig. 9'; on the face in the shade the maximum temperature is 45°C on 8th July and 20th August;
- the maximum temperature in the exposed face is higher in the dark samples, with a difference of colour as 10°C in respect to the light samples;
- the maximum temperature in the face in the shade does not vary in colour of the panel;
- the minimum temperature in the exposed face does not differ significantly in variation of colour and the minimum value registered is 4°C in 13th March.

Since March 2004, the following climatic data were supplied by the climatic station of DEAF:
- temperature;
- relative moisture;
- radiance;
- wind speed;
- rainy conditions.

The objective is to be revealed by such data, at least until March 2005, in a way to have a complete picture of temperature trend in a year of sunshine. That is, with the aim of comparing the decay effects on external samples with real climatic conditions and to calibrate appropriately one or more test cycles of which samples will be subjected to.

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[6] The products used were kindly supplied by SIKA Italy with reference to " Sikagard 552W Aquaprimer ", a varnish with a syntetic base and " SIKA 550 W Elastic " a varnish with an acrylic base.
[7] A type of " i-button " produced by T-MEX, applied to samples by a silicone thermo-conductive paste.

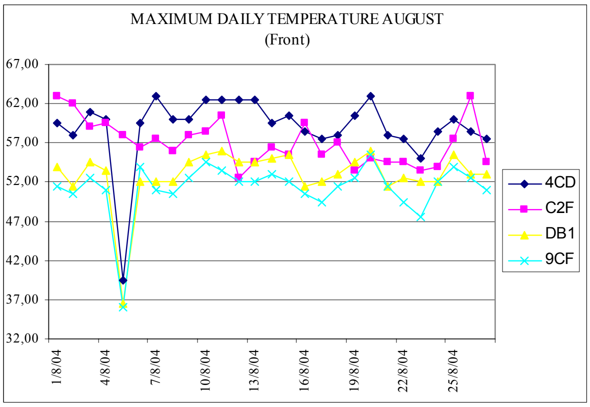

Figure 9. Trend of maximum daily temperature in the month of August 2004.

8 CONCLUSIONS

The samples were expose on 7th March 2004. With regard to aspect it can be observed that the panels have lost their initial superficial brightness and display spots as a result of dust deposits. On the other hand, the edges have remained intact which suggest the efficiency of the protective paint.
Therefore the research will proceed in different directions as foreseen, with the aim of to reach to the assessment of durability on sandwich panels, with reference to climatic context in southern Europe.
The next steps are:

- the completion of the theoretical study of reliability;
- the measurement of mechanical and thermal characteristics on the samples after 1 year of natural aging in the open air;
- to set a cycle of accelerated aging tests;
- the execution of accelerated aging tests;
- the assessment of durability.

9 REFERENCES

Alaimo, G. 2004, *L'affidabilità funzionale delle coperture discontinue*, Editecnica, Palermo.

Alaimo, G. 2003, Methodological development and experimental evaluation of discontinuous roofing durability, in Acts of conference "Management of durability in the building process", Milan, 25-26 June 2003.

Alaimo, G., Accurso, F. 2003, Il degrado delle coperture con pannelli sandwich nel contesto climatico di Palermo, in Atti del convegno "Involucri quali messaggi di architettura", Napoli, 9-11 Ottobre 2003.

Daniotti, B., Maggi, P.N. 1993, *Valutazione della qualità tecnologica dei prodotti complessi per l'edilizia*, Quaderno DISET, vol. 2, Esculapio, Bologna.

Maggi, P.N., Gottfried, A., Morra, L. 1992, *Qualità tecnologica dei prodotti complessi per l'edilizia*, Quaderno DISET, vol. 1, Esculapio, Bologna.

Maggi, P.N. (a cura di) 2000, *La qualità tecnologica dei componenti edilizi. La durabilità*, Epitesto, Milano.

Maggi, P.N. (a cura di) 2002, *La qualità tecnologica dei componenti edilizi. La valutazione della durabilità. Risultati della prima fase sperimentale*, Epitesto, Milano.

Rejna, M. 1995, *Valutazione della qualità tecnologica utile dei prodotti complessi per l'edilizia*, Quaderno DISET, vol. 4, Esculapio, Bologna.

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Service life testing of PCM based components in buildings

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ABSTRACT

In the CRAFT project C-TIDE (Changeable Thermal Inertia Dry Enclosures) the possibility of changing the thermal inertia of lightweight buildings with PCM, Phase Change Material, is explored. The project is performed in collaboration with Italian and Swedish partners representing both the industry and research. Lightweight buildings represent a well-established technology in Sweden. In Italy this technology is entering the market.

A problem is the overheating of the building during the hot season, especially in warm climate but also in Nordic climate during summer. This project deals mainly with this problem.

A crucial issue of the use of these materials is the performance over time data of the phase change material that are used in the building. An extensive program is set up to perform long time testing of the thermal properties of the materials that were used in the project. The long time testing programme of the materials was established to correspond with the governing procedure set up by ISO 15686-2 "Buildings and consructed assets– Service life planning – Part 2: Service life prediction procedures".

This paper describes the framework of the testing procedure, the set-up of the testing equipment and preliminary results of the tests.

KEYWORDS: Service life, phase change materials.

1 INTRODUCTION

1.1 Building model

In the CRAFT project C-TIDE (Changeable Thermal Inertia Dry Enclosures) the possibility of use of PCM to change the thermal inertia of lightweight buildings is explored. Thermal inertia and thermal protection is the area where the PCM will achieve a high penetration in the market (Zalba 2003).

Energy is stored/released by sensitive heat in the liquid phase and in the solid phase. When the material melts or solidifies the energy is stored/released by latent heat. The energy that is stored over a temperature span where the material goes from solid to liquid is the sum of the sensitive and latent heat. The phase change takes place over a small temperature span; thereby large amounts of energy can be stored by small temperature change in the PCM (Zalba 2003 p.266).

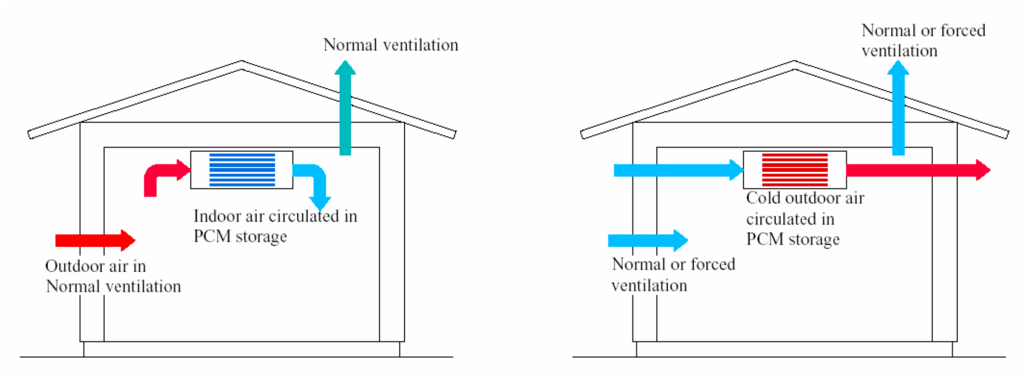
The project is performed in collaboration with Italian and Swedish partners representing both the industry and research. Two different approaches of the integration of the PCM are taken in Italy and Sweden. In the Italian project PCM is integrated in the façade of a building. The Swedish group investigates the possibility to place the PCM inside a building and actively, with fans, exchange the stored energy. These considerations are taken due to the differences in the outdoor climate in the two regions of Europe.

Regardless of type of integration of the material, the long time performance of the PCM is of a crucial importance for the performance of the systems. This issue is addressed in the European Construction Product Directive, which states that essential requirements of all building materials must be fulfilled throughout its service life.

The work by the Swedish group will deal with two issues:

Issue 1; long time performance of the PCM material. The heat capacity and chemical stability is tested where the PCM is exposed to temperature cycling. The tests are carried out using a water bath calorimeter where the sample weight is about 1.5 kg.

Issue 2; a PCM night cooling system is investigated. Cool night-air is used to cool the PCM and the building interior; Figure 1.The aim of the use of PCM is to reduce the energy needed for cooling of building where there is overproduction of heat. The driving force of the cooling system is the cold night air, which crystallizes the PCM during nighttime. During daytime the PCM melts and cools the indoor air.



a) **b)**

Figure 1. Principal function of PCM night cooling system. a) is the day case where the PCM is melting. b) is the night case where the PCM is crystallized. The PCM cooling system works independent of the normal ventilation system of the building.

2 FRAMEWORK FOR TESTING PROCEDURE

2.1 Construction Product Directive

Materials that are going to be used in buildings must fulfil the essential requirements of the Construction Product Directive, (CPD1988). Six essential requirements must be fulfilled during the service life of the product.

"The products must be suitable for construction works which (as a whole and in their separate parts) are fit for their intended use, account being taken of economy, and in this connection satisfy the following essential requirements where the works are subject to regulations containing such requirements. Such requirements must, subject to normal maintenance, be satisfied for an economically reasonable working life. The requirements generally concern actions which are foreseeable" (CPD1988)

The six essential requirements are 1) Mechanical resistance and stability, 2) Safety in case of fire 3) Hygiene health and stability, 4) Safety in use, 5) Protection against noise and 6) Energy economy and heat retention

In this case requirement 6 "Energy economy and heat retention" sets the target of the tests of the material. Further requirement 6 states:

"The construction works and its heating, cooling and ventilation installations must be designed and built in such a way that the amount of energy required in use shall be low, having regard to the climatic conditions of the location and the occupants."

The idea of using PCM to adjust the thermal inertia is to lower both annual energy consumption and the maximum power consumption to cool the building.

The CPD also gives prerequisites for a CE marking of a building product. With the CE marking a product a can be marketed the EU area as a single market. The use of the product is regulated by the national building code. (Sjöström, Lair 2003) The CE marking can be based on harmonized national standards, which applies for common building products. For less frequent product the CE marking should be based on European technical approvals, ETA. The latter applies for the PCM product family.

A product, which complies with the following definition must be CE marked (CPD 1988): "a product which is produced for incorporation in a permanent manner in construction works, including both buildings and civil engineering works, and which is placed on the European internal market."

This project is not intended to create a technical approval for the product but can serve as a guidance to establish standardized testing methods for this type of material in this specific application.

2.2 ISO 15686-2

The international standard ISO 15686-2, Buildings and constructed assets – Service life planning, part 2, Service life prediction principles gives a generic methodology on how to set-up the testing procedure, Figure 2.

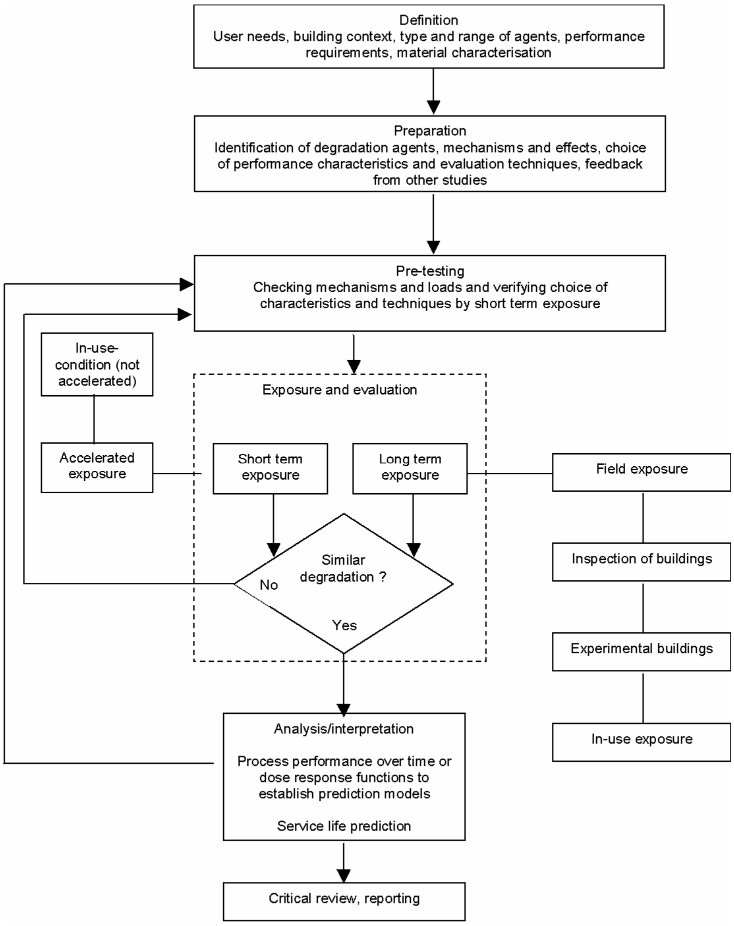


Figure 2. Systematic methodology according to ISO 15686-2 "Buildings and consructed assets– Service life planning – Part 2: Service life prediction procedures".

2.2.1 Definition, user needs, building context, type and range of agents.

The phase change material that will be tested is an industrial product packed in aluminium pouches. The product is based on Glauber´s Salt ($Na_2SO_4\,10H_2O$) where the melting temperature of the main agent is 32.4 ºC (Zalba 2003). The product is a mix of agents in order to overcome known problems, in temperature cycling, like super cooling, incongruent melting and phase separation (Carlsson 1978) and (Lou 1983). The testing procedure is not intended to develop the composition of the actual product but is focused on the performance of the supplied product. The procedure is general for similar products in this type of application.

The critical properties of the material, which could be expected to change over time, are the specific heat, latent heat and thermal conductivity. The testing procedure is focused on specific heat and the latent heat.

Other critical properties concerning the enclosure of the product are: mechanical stability due to handling, use of material in application; Mechanical stability of due to expansion and contraction of salt hydrate; Chemical (electrochemical) stability of enclosure material; Corrosion from salt hydrate on enclosure material; Corrosion problem in contact with ambient materials.

The product is to fit into a building to adjust the thermal inertia of a building. The temperature range, which the PCM should work, is limited to the operative temperatures of the ambient air. The PCM is crystallized during the night. This will give the lower limit of the temperature range, 18 ˚C. The highest acceptable indoor air temperature gives the upper temperature range, 28 ˚C. The manufacturer gives the following data, Table 1.

Phase change temperature	24	ºC
Latent heat	108	kJ/kg
Specific heat (sensitive heat)	3.6	kJ/kg˚C
Heat storage capacity 15 – 45 °C (Latent+sensitive)	216	kJ/kg
Specific gravity:	1480	Kg/m^3
Thermal conductivity	0.5 – 0.7	W/m˚C.

Table 1. Material properties.

The PCM is used to limit high temperatures in buildings during the summer period. A crystallization and melting cycle is expected to take place every 24 hours during a half year. This sets the number of test cycles for each sample to 200 cycles.

2.2.2 Preparation

The agent that is expected to affect the material most is the 24 hour temperature cycling. Each day a crystallisation and melting will take place. Studies of a similar material from the same manufacturer were carried out in 1981 (Heteny 1981). The heat storage capacity (Latent+sensitive) was determined in a temperature interval between 15 and 35 ºC. The thermal properties were obtained by using DTA (differential thermal analysis). The weight of the samples was about 400 mg.

The product is intended for use in large quantities in buildings. Therefore an approach of testing large samples of the product is taken. To overcome errors in the measurements due to heat transfer between ambient medium and testing sample it was decided that large samples of the material should be tested in a water calorimeter. The sample size was decided to approximately 1.5 kg.

2.2.3 Accelerated exposure

The reasons why water bath calorimeter is used are: It is possible to accelerate the tests; the cycling time in the water bath calorimeter is about 3 hours. The equipment is inexpensive and gives the possibility to test large samples.

The following testing program was set up

Number of samples: 5

Number of cycles for each sample: 200.

The product will be tested in the following temperature intervals:
15 - 45 ºC
18 - 28 ºC
21 - 27 ºC

Differential scanning calorimetry is a common method to establish the thermal properties. The testing is carried out on small samples. Problems with the method are that the result of the measurements varies with the scanning rate (He 2004). Other testing methods for large samples with the temperature history method are described by (Yinping 1999) and (Marin 2003).

2.2.4 Field exposure/Experimental building

A semi full scale test will be carried out coming summer where the application of the product will be tested in real conditions see Figure 1. The design of the equipment is based on the data given by the supplier of the material.

3 PRELIMINARY RESULTS

At the time of writing this paper there are only preliminary results available. The temperature in the PCM starts at 15 ˚C (solid) is put in a water bath with 45 ˚C. The material goes from solid to liquid. Figure 3 shows how the temperature in the middle of the sample changes with time. Figure 4 shows the accumulated energy absorbed by the sample over the temperature interval. The preliminary results include one sample.

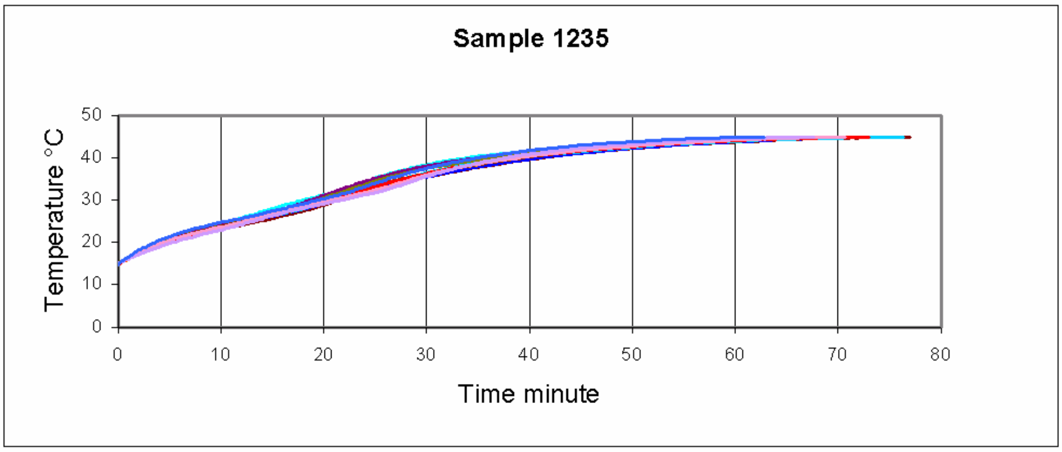


Figure 3. Preliminary results of water bath calorimeter measurements. Temperature in the middle of sample.

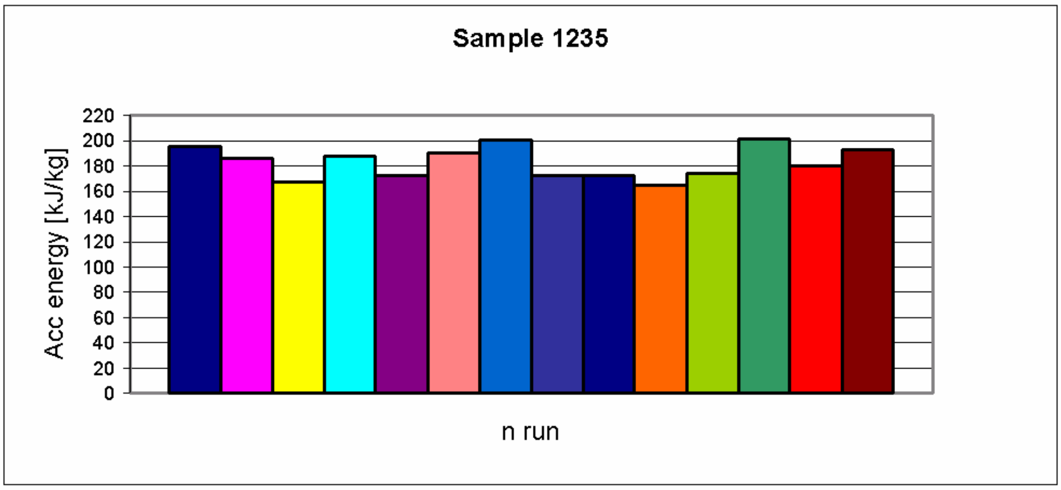


Figure 4. Preliminary results of water bath calorimeter measurements. Accumulated energy.

4 DISCUSSION

Products that are installed in buildings need to have long service life. The service life is counted in 10:th of years. EOTA guidance document 2 states that a short service life is 10 years and a long is 100 years. (EOTA Guidance Document 002, 1999). This implies the importance for a manufacturer or a supplier of a product to inform his customers of the service life of a product. With a CE marking of a building product a product can be marketed in the whole European market. The product must fulfil essential requirements for its intended use throughout its working life. Therefore it is important to develop testing methods that resemble the actual use.

In this project focus of the intended use is seen as an important issue. As guiding document for development of the methods serves the ISO 15686-2 "Buildings and consructed assets– Service life planning – Part 2: Service life prediction procedures". In developing a service life testing procedure user needs, actual degradation agents, mechanisms of degradation, comprasison of short and long term testing results are taken into consideration. The long time performance of a phase change material, PCM, is tested and these considerations have been taken into account when choosing testing methods. This is done for example by testing the PCM where it is packed in a similar way at it is expected to be done in an actual use situation. It will be packed in flat aluminum pouches with ta thickness ranging from 10 to 30 mm. The area of use is to change the thermal inertia of a building therefore large quantities of the material must be used.

The operative temperature interval is also of great importance in the testing method. The actual use is a semi passive method where no other cooling equipment is involved. It means that the operative temperature is around the indoor climate of a room. The product will be tested in different temperature spans 15-45, 18-28 and 21-27 ºC The tested product is a mixed product where the melting temperature according to the supplier of the material is 24 ºC. The main agent though, Glauber´s Salt Na_2SO_4 $10H_2O$ has a melting temperature of 32 ºC, which is above the actual temperature span. Glauber´s Salt has in earlier studies shown problems like incongruent melting and phase separation. It is to overcome these problems that the actual product is a mixture. The composition of the mixture is not known to the testing team, the product is seen from the "outside", as it will be packed and delivered to customers.

The shape of the curve in Figure 3 indicates that there is not a clear phase change temperature in the tested temperature span, 15 to 45 ºC. Figure 4 shows the thermal storage capacity in the investigated temperature span. Mean value is 180 kJ/kg. Over a 30 ºC temperature span the average storage capacity is 180/30=6 kJ/kgºC. This can be compared with the heat capacity of water, 4.2 kJ/kgºC. This indicates that there is a phase change taking place in the temperature span. In the earlier test of a similar material (Johansson 2001), (Heteny 1981) different temperature spans are investigated. The preliminary results indicate similar results.

The operative temperature span for the product is 24±3-4ºC. In further testing these temperature spans will be investigated. A full report of the testing results will be presented in a later report.

5 CONCLUSION

In the CRAFT project C-TIDE the use of phase change materials, PCM, to change the thermal inertia of lightweight buildings is investigated. It is a joint project with Italian and Swedish partners representing both industry and research.

A building material that is CE marked can be marketed on the whole European marked. But to CE mark a product its service life must be declared. Therefore an extensive program is set up to develop and perform long term testing of the material. The testing procedure is set up according to ISO standards concerning long time testing.

The tests are focused on the change over time of the heat storage capacity, the sum of sensitive and latent heat, over different temperature spans measured with water bath calorimeter on large samples. Preliminary results from the tests are presented in the paper.

6 REFERENCES

Carlsson Bo, Stymne Hans, Wettermark Gunnar, 1978. *Storage of low-temperature heat in salt-hydrate melts – calcium chloride hexahydrate.* Swedish Council for Building Research. Document D12:1978. ISBN 91-540-2889-2.

CPD 1988, *COUNCIL DIRECTIVE of 21 December 1988 on the approximation of laws, regulations and administrative provisions of the Member States relating to construction prod*ucts. (EG:s direktiv rörande byggprodukter) ISBN 91-85548-38-3.

EOTA Guidance Document 002, 1999. *Assumption of working life of Construction Products in Guidelines for European Technical Approval, European Technical Approvals and Harmonised Standards.* EOTA Guidance Documents 002/ Edition December 1999

Heteny, Janos, Lagerkvist Knut-Olof, 1981. *Långtidstest genom cycling av ett saltsmältlager med glaubersalt som energilagringsmaterial*. (Long-term testing by cycling a phase-change heat store with glauber salts as an energy storage material). Teknisk rapport SP-RAPP 1981:05. Borås, Sweden 1981.

He, Bo, 2004: *High-Capacity Cool Thermal Energy Storage for Peak Shaving*. Doctoral Theseis, Stockholm, Sweden 2004. TRITA-KET R195. ISSN 1104-3466. ISBN 91-7283-751-9.

ISO 15686-2, *Buildings and constructed assets – Service life planning, part 2, Service life prediction principles*. First edition 2001-03-01. International Organization for Standardization

Johansson Ulrika, 2001. *DCN analysis of PCM*. SP Report F1 16387

Lou D.Y. (1983). 'Solidification process in a Glaubersalt mixture'. *Solar Energy. vol 30, no 2,* pp 115–121.

Marin, J.M.; Zalba, B.; Cabeza, L.F.; Mehling, H. 'Determination of enthalpy-temperature curves of phase change materials with the T-history method - improvement to temperature dependent properties'. *Measurement Science and Technology*, 14, 184-189, 2003. (2003).

Sjöström, Lair (2003). 'Performance Based Buildings – Some implications on Construction Materials and Components', *Proceeding of the 2nd International Symposium Integrated Life-time Engineering of Buildings and Civil Infrastructure*, Kuopio, Finland, 2003. ISSN 0353-9403, ISBN 951-758-436-9.

Yinping Z, Yi J, Yi J, (1999). 'A simple method, the T-history method, of determining the heat of fusion, specific heat and thermal conductivity of phase-change materials'. *Measurement Science. Technology. volume 10*, pp 201–205.

Zalba Belén, Marin José Mª, Cabeza Luisa F., Mehling Harald. 2002. 'Review on thermal storage with phase change materials, heat transfer analysis and applications'. *Applied Therma Engineering 23 (2003)* 251-283

Atmospheric corrosion of metals in Iceland – characterisation of the environment and five-year results

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ABSTRACT

In Iceland many kinds of building materials have to be imported because of limited natural resources. The building market is small and far away from other markets, so transports costs easily run high. In this context, and generally speaking, it is important to obtain the best economical longevity of materials as possible, e.g. by limiting corrosion of metals. It is therefore of interest to consider what the corrosion speed and corrosive agents are so as to be able to estimate material degradation and describe what corrosion protection is needed. Mapping to show the atmospheric corrosion rate is essential to be able to follow changes over time and also gives a much-needed information to the domestic market about how the conditions relate to other known parts of the world. The climate in Iceland is characterised by a wet and windy southwestern part and a colder-dryer northern part. The air is mostly clean, due to the windy climate and rather minimal local pollution, and so the corrosion environment ranges from a wet, chloride-rich area in the south, to a drier and cooler area in the north. The corrosive environment is now being mapped by weathering of small test pieces; mild steel, zinc and aluminium, at 16 different locations.
Measurement results for corrosion over a total of five years have now been gathered and these show a fairly good correlation with what is expected from the mapping of climate in accordance with ISO 9223, the corrosion classes being in the range 3-4.

Measured corrosion differs much between places; Corrosion of zinc (1-year results) varies between 11.3 and 28.3 g/m^2, similar figures for steel are 12.7 and 297.5. Chloride is considered as an important variable in the corrosive environment on Iceland, and measurements on both chlorides and SO_2 in air have started recently at some of the test locations. The variability in chloride content is great and it is evident that a model of chloride content based on weather parameters and location relative to the sea, will be necessary before dose-response functions for corrosion can be determined. The paper discusses project design and presents 1-, 3- and 5-year results of corrosion on mild steel, aluminium and zinc.

KEYWORDS

corrosion, metals, atmospheric corrosivity, climate.

1 INTRODUCTION

Iceland is an island in the North Atlantic Ocean. The middle is uninhabitable highlands, and the built environment is, thus, mainly along the coast. The climate is marine-influenced and geographic location is in the route of frequent low-pressure systems that move over the ocean. Accompanying the low pressure systems are prevailing wind directions of south-west to south-east and this, in combination with mountains and a few glaciers, results in wide climate variation within the country.

The climate is windy and moist, and precipitation is very common on the south coast. The highlands, especially north of the glaciers, are very dry, with the area north of the largest glacier, being the biggest desert in Europe. The energy use for heating and electricity is environmentally friendly, as it involves geothermal energy and electricity from hydro power stations.

Heavy industry is on a small scale (three factories so far are located near Reykjavik). The number of cars per capita in Iceland is very high and pollution from them is noticeable in the capital city, Reykjavik, on the few calm days each year. In periods of gentle south-easterly winds, polluted air from Europe comes to Iceland. However, the air is mostly clean, due to the windy climate and rather minimal local pollution. Information on air pollution in Iceland is limited and measurements are mainly done to assess the pollution from traffic in the capital.

Almost all building materials, except cement and rock materials, have to be imported to Iceland, and this obviously applies especially to all metals. The annual import of metals for the building industry is approximately 310 kg/capita, and other annual imports (transport vehicles, etc) about 185 kg/capita. Of the total annual import approximately 370 kg/capita of metals end up in a corrosive environment of some kind. Metals are recycled to some extent or about 0.2 kg/capita.

Iceland's market is small and far away from other markets, so transports costs easily run high. In this context, and generally speaking, it is important to obtain the best longevity of materials as possible, e.g. by limiting corrosion of metals. Therefore, corrosion speed and corrosive agents must be evaluated. Mapping to show the background atmospheric corrosion rate is likewise essential to be able to follow changes over time, but also to be able to inform the market about how the conditions relate to other known parts of the world.

2 CORROSION AND WEATHERING TEST PROGRAM

Corrosiveness of the environment can be estimated using two different methods: direct measure-ments (here as loss of weight, see ASTM G1-90), or evaluation of atmospheric agents in accordance with ISO 9223. In the current project both methods are used to evaluate the corrosive environment in Iceland. The project has been presented at earlier stages by Marteinsson and Sigurjónsson [2002] and Marteinsson *et al.* [2004], where comparison between mapping of climate according to ISO 9223 and the test results showed a fairly good correlation. Based on measurements the corrosion classes for steel and zinc are in the ranges 2-3, and 3-4 wheras based on the climatic classification the classes are in the range 3-4. This paper presents results from measurements of first-, third and fifth-years corrosion and discusses the differences in corrosive climate in Iceland.

Samples sized 100x150x3 mm were mounted in fasteners of polymer materials on a backing panel of plywood, such that the distance from the backside of the sample to the plywood sheet is 17 mm with 15 mm between different sample materials. The samples are placed at 15 test locations, each rack with a mounting board of size 530x860 mm at a height of 3 - 4m above ground, oriented to face south at an angle of 45° in accordance with EN ISO 8565. The specimens are exposed in unsheltered conditions and are of technically pure, low carbon steel (C=0.05%), zinc (> 99,9% pure), aluminium of types 1050A and AlMg3 (3% Magnesium) and also different types of painted steel samples. Corrosion is a function of time (see later), to register this and generally as our main interest is the effect of corrosivity on degradation of material, most of the samples are exposed for many years. Plans are to demount samples for evaluation after 1, 3, 5 and 10 years. The measurements consist of recording the corrosion of the metals and evaluating the condition of paints. The project started in 1999, and results to date are therefore limited to five year's corrosion.

The test locations are shown in Figure 1, and numbering of them is listed in Table 1.
The results for corrosion rate of steel, zinc and aluminium after one, three and five years are shown in Table 1.

From the start, it was clear that some spot measurements of at least SO_2 and salinity would be needed, but it was decided to wait for initial results on corrosion to see where to concentrate the main effort. These spot measurements started late summer 2004 at 6 of the 16 test locations, the stations being no. 3, 6, 8, 13, 14 and 16. Salinity is measured as concentration in rainwater and SO_2 as concentration in air, both measured as monthly values.

3 THE CLIMATE AND CORROSIVE AGENTS IN ICELAND

Atmospheric corrosion of metals is discussed at length in the literature, the standard ISO 9223 specifies the key factors and Haagenrud [1997] gives a very good overview. The climatic factors used in the project are measured at, or nearby, most of the test stations by the Icelandic Metrological Station. These measurements have been made for years, with temperature, humidity, wind-speed and direction logged automatically each 1 or 3 hours and in some instances precipitation is also available. SO_2 is usually considered one of the major corrosive agents of steel and zinc. Iceland has little SO_2 pollution, as almost all house heating is done either by geothermal energy or electricity from hydro power stations. Due to increased industrialization and increased use of thermal energy the pollution may though be increasing, at least locally. The island is very sparsely populated and the average wind speed is high. Thus, problems concerning air pollution are so far uncommon and only limited measurements are made of these agents.

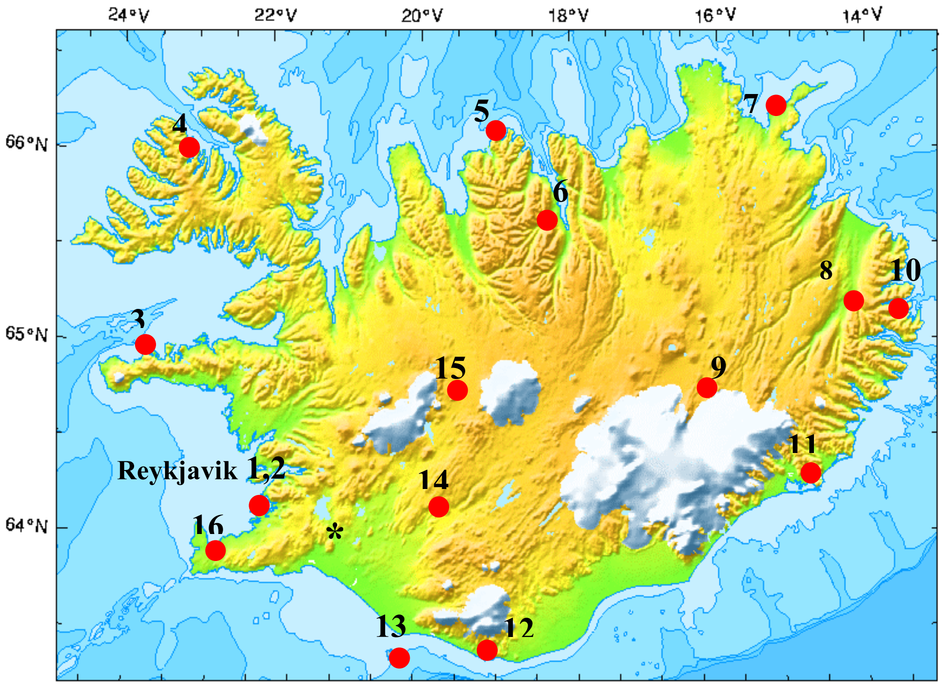


Figure 1. Iceland ; exposure test locations (numbers explained in Table 1; * Irafoss, see text)

Table 1 Corrosion in unshealtered locations for carbon steel, zinc and aluminium (g/m2)

	Location	Carbon steel	Zinc	Al 1050
1	Reykjavík			
	1 year	170.0	17.0	0.5
	3 years	263.7	22.8	0.0
	5 years	214.8	32.3	0.2
2	Reykjavík M1			
	1 year	153.3	20.0	
	3 years	264.7 *	28.7 *	0.0
	5 years	181.0 *	44.3 *	0.3 *
3	Ólafsvík			
	1 year	178.3	19.8	
	3 years	295.3 *	38.7 *	0.3 *
	5 years	181.7 *	70.7 *	0.7 *
4	Bolungarvík			
	1 year	125.2	13.8	0.0
	3 years	288.8	16.0	0.0
	5 years	126.8	23.5	0.2
5	Siglufjörður			
	1 year	108.8	10.7	
	3 years	211.0 *	27.3 *	0.0
	5 years	157.3 *	25.3 *	0.0 *
6	Akureyri			
	1 year	46.0	11.5	0.0
	3 years	135.3	11.2	0.2
	5 years	80.5		0.0
7	Þórshöfn			
	1 year	142.2	12.0	
	3 years	296.0 *	24.3 *	0.0 *
	5 years	127.0 *	23.0 *	0.0 *
8	Egilsstaðir			
	1 year	30.0	18.2	
	3 years	92.7 *	15.3 *	0.0 *
	5 years	68.0 *	30.3 *	0.0 *
9	Kverkfjöll			
	1 year	12.7	11.3	
	3 years			0.0
	5 years			
10	Neskaupsstaður			
	1 year	114.0	28.3	0.2
	3 years	290.8	38.0	0.3
	5 years	234.3	56.7	0.3
11	Höfn í Hornafirði			
	1 year	209.0	12.2	
	3 years	329.0 *	40.7 *	0.0 *
	5 years	170.0 *	42.0 *	0.0 *
12	Vík í Mýrdal			
	1 year	157.2	17.0	0.3
	3 years	277.3	34.0	0.0
	5 years	177.8	34.8	0.0
13	Vestmannaeyjar			
	1 year	297.5	15.8	
	3 years	515.0 *	47.3 *	0.3 *
	5 years	431.3 *	53.7 *	0.3 *
14	Búrfell í Þjórsárdal			
	1 year	184.0	19.7	
	3 years	268.0 *	23.7 *	0.0 *
	5 years	143.0 *	44.0 *	0.0 *
15	Hveravellir			
	1 year	37.2	17.3	
	3 years	90.3	21.0	0.0
	5 years	70.7	34.7	0.0
16	Svartsengi			
	1 year	287.8	17.2	
	3 years	436.0 *	44.3 *	0.0 *
	5 years	322.3 *	61.0 *	0.0 *

* One specimen (in other cases two)

It is known that salinity greatly affects the speed of corrosion, and even in highly populated areas this effect can be more than that from SO_2, especially as the content of SO_2 is decreasing in many places, as discussed by Haagenrud [1997] and reported in '*Hot Dip Galvanizing*' [2001]. The chloride deposited has until recently only been measured regularly at two places in Iceland: in the capital Reykjavik, and at Irafoss, which is inland from the south coast. These measurements show great difference in concentration between places, and the newly started measurements in the discussed project indicate an even greater variation.

For the exposure locations, the climatic factors of importance in this context can be described as in Table 2 (with reference to some of the locations). A somewhat more detailed discussion is found in Marteinsson and Sigurjonsson [2002].

4 CLIMATE AND MEASURED CORROSION

The measured weather parameters are not that different between stations when the average yearly values are considered, this holds especially true for temperature and humidity. Monthly variations in the weather variables are though considerable and it should be of interest to consider how these factors combine to form the corrosive environment that results in the cumulative corrosion attack.

Table 2. Description of the climatic variables at exposure locations

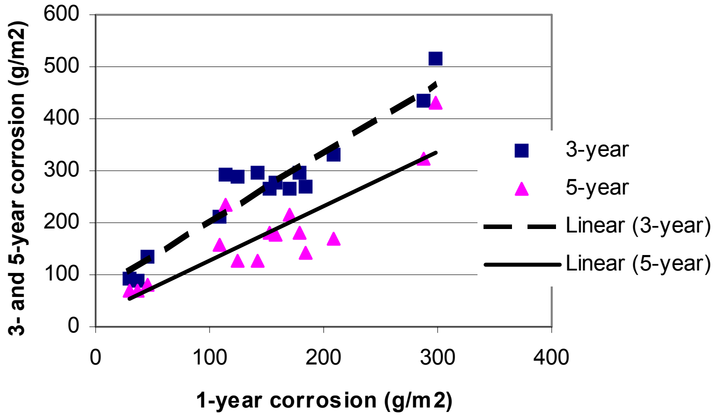
Temperature (°C):	The yearly average temperature varies locally from 0.6 (no. 15) to 6.4 °C (no. 12).
Humidity (%RH) :	The yearly average value of relative humidity (% RH) is in the interval 78 - 83 % RH at the coasts and slightly higher in the highlands.
Time of wetness (TOW)[1]:	The yearly value is in the interval 1449 hrs. (no.10) to 6324 hrs. (no. 13).
Rain (mm):	The yearly precipitation is from 860 mm (no. 15) to 3375 mm (no. 12)
Wind (m/s):	The climate is windy all year round. The average yearly wind speed at the locations is measured in the range of 3 - 10 m/s, with one location at each extreme, but usually the value for average wind speed is between 4.7 and 7.1 m/s.
SO_2 ($\mu g/m^3$):	The yearly mean, near a heavily travelled street, measured in 1991 as 3.2 $\mu g/m^3$.
Salinity (mg/m^2,d)[2]:	The yearly mean values are for Reykjavik for the years 1992-2000 in the intervall 19.6 – 50.0 and at Irafoss in the year 1985 the mean value was 10.0 The resently started measurements show monthly average values between 1.3 and 121.4.

1) TOW according to the definition of ISO 9223; number of hours with T>0 ºC and RH>80 %
2) Wet deposition

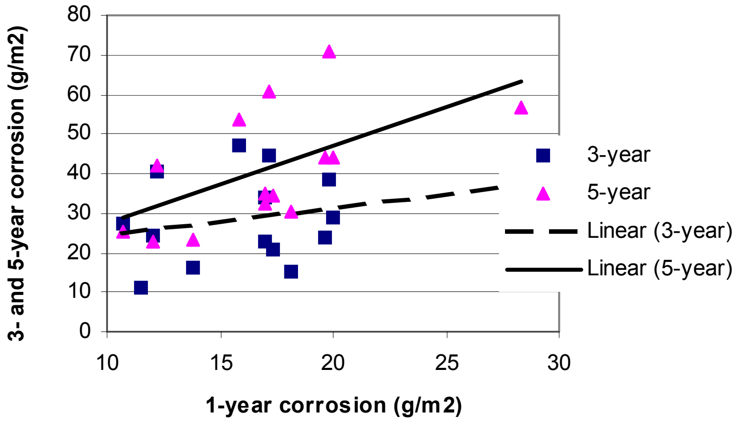
This work has only started and as yet has not given any useful results that can be implemented in the modelling of the dose-response functions, which will be done later in the project. The analysing of information regarding climate during the exposure time and especially the measurements on chloride concentration is not finished and the discussion is therefore mainly limited to the measured results of corrosion.

Just as could be expected, and in harmony with common knowledge, carbon steel corrodes faster than zinc and aluminium corrodes very slowly over the five year period considered, see table 1. The measured corrosion (g/m^2) of steel shows great fluctuations and inconsistency over time, as the measured results for the three-year period are often higher than those measured after five-year (Figure 2a). Measurements on both zinc and aluminium do not show similar irregularity, for zinc see Figure 2b, and it has therefore to await for the next measurement (the 7-years observation) to see if this inconsistency can be explained. It is of interest to note than when comparing either three-year or five-year measurements with one-year measurements, the results for steel show less distribution than similar results for zinc. It is clear that the effect of corrosion with time is very different between stations (given that the climate is similar between time periods), probably because of a rather complex function between degradation agents and the resulting degradation.

Dose-response functions link the dose of agents (climatic parameters or pollution) to the rate of material corrosion. Several dose-response functions have been published over the years; Haagenrud (1997) and the results from the UN/ ECE project ICP-Materials give a good overview. The dose-response functions for e.g. carbon steel or weathering steel found in the literature are though often targeted on the effect of acidifying pollutants and in the ICP-materials project sites with elevated level of chloride deposition have been deliberately excluded from the program, see Tidblad and Kucera [2003].



2a Corrosion of steel



2b Cossosion of zinc

Figure 2 Corrosion of steel and zinc
Comparison of three- and five years corrosion to first-years corrosion

In the ICP-materials program the dose response functions are preferably of the type shown in Eq. 1

$$K= dry(T, Rh, [SO_2],[NO_2],[O_3],t) + wet(Rain[H^+],t) \quad [1]$$

where
T temperature (°C)
Rh relative humidity (%)
[ ] concentration of SO_2, NO_2, O_3 (µg/m3)
Rain precipitation (mm)
[H+] acidity of precipitation (mg/l)
others as earlier

The functions are generally based on yearly values (averages or totals) of the agents in question. Many of the agents (parameters) used in the dose response functions found in the literature are not known for the exposure locations in Iceland. When the dose response functions were tested on the data from our project, with different values for the unknown factors (S, SO_2, H+, Cl- etc.) based on an intelligent guess, it was apparent that the functions are poor estimators for corrosion speed in Iceland. The reason for this is probably that the SO_2 content is usually low, and thus not a main factor in the corrosion environment but then salinity is high at times. It is of interest to define what parameters are of importance for the measured corrosion and by use of dose-response function make it possible to interpolate between known, measured, values.

Based on the general knowledge on climate in Iceland, as discussed above, the agents that primarily affect atmospheric corrosion in Iceland are assumed to be rain, humidity, chlorides and temperature. Until measurements show otherwise the effects of SO_2, NO_2 and O_3 are thought to be small. The parameters that show biggest variation between locations are precipitation, time of wetness and salinity, but humidity and temperature show less variation. It is known that salinity concentration is highest at the south-west parts of the country and lowest in the north-east parts (south to south-westerly winds prevails in Iceland). Salinity is carried by wind over great distances and in southern storms, saline deposit is observed on structures on the north coast, despite highlands in the middle of the country that, depending on direction of wind from coast to coast has an average height of 400 - 600 m. Concentration of salinity in air due to wind must depend on many factors, including topography, wind speed and direction, as well as distance from location to sea in line with the wind direction. Cole et al. [1999] reported a study of salinity, which takes into account how salinity transfers from sea to air and the effect of this on the corrosiveness of the climate. In these studied wind speed was found to be a very important variable.

Based on the first year results for corrosion, and the measured climatic parameters for moisture, temperature, wind-speed and direction, attempts were made [Marteinsson et al. 2004] to model dose-response functions where chloride content was modelled based on the discussion by Cole [1999]. These models gave strongest dependency between time of wetness and corrosion, but with more information on the chloride content in air this may change. The model-results give an acceptable correlation with measured corrosion (1-year), a model that takes the time factor into consideration has though not been found yet and more test results are probably needed for this.

Measurements on the mentioned climatic variables will of course be continued, and information on them may be expected to be accessible in the future for many places in Iceland. On the other hand it is doubtful if extended measurements on chlorides will be continued in Iceland and the need for modelling the concentration of chlorides, based on measurements, is therefore great. As the chloride concentration shows great fluctuations with time it will also be of interest to see if the effect of time series of weather parameters and chlorides on corrosion may be modelled better based on monthly measurements of corrosion for some of the stations.

5 CONCLUSIONS

Corrosion of metals, especially steel, in Iceland varies widely for different locations, with the main corrosive factor seemingly being airborne salinity in addition to the humid environment. The measurements give information about the atmospheric corrosion in Iceland, which makes comparison with other countries much easier. The corrosion rate has been measured over five years and the project will continue for a total of ten years. Dose response functions in the literature do not seem to be appropriate for Icelandic conditions which shows that the impact of corrosive agents in different environments can be very different. Attempts to model own dose response functions have not given good results so far as the measured corrosion varies greatly in time and concentration of salinity is not yet sufficiently mapped. The content of salinity and SO_2 concentration is measured at few places to obtain knowledge about what values to expect. These measurements will probably be made periodically, and over limited time. A model for estimating salinity in air needs to be worked out for use in future estimates of corrosion. Further data processing of the effect of climate on corrosivity of metals in Iceland will continue when more results from the pollution measurements have been gathered. In the continuation of the project some test panels will be exposed for one-month periods to get a better information on the effect of the time series of weather on corrosion.

6 ACKNOWLEDGMENTS

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7 REFERENCES

ASTM G1-90 G1-90(1999)e1 Standard Practice for Preparing, Cleaning, and Evaluating Corrosion Test Specimens, ASTM International

Cole, I.S., King, G.A., Trinidad, G.S., Chan, W.Y. & Paterson, D.A. 1999, 'An Australian-Wide Map of Corrosivity: a GIS Approach', *Proc. Durability of Building Materials and Components 8. Edited by M.A. Lacasse and D.J. Vanier, Institute for Research in Construction, Ottawa ON. K1A 0R6, Canada,* 30. May - 3. June 1999, pp. 901-911.

EN ISO 8565: Metals and alloys - Atmospheric corrosion testing - General requirements for field tests.

Haagenrud, S.E. 1997. *Environmental Characterisation including Equipment for Monitoring*, CIB W80/RILEM; 140-PSL SubGroup 2 Report, Norwegian Institute for Air Research (NILU) OR:27/97.

Hot Dip Galvanizing Volume II . Number 2 June 2001: The millennium Map Project

ISO 9223:1992 Corrosion of metals and alloys - Corrosivity of atmospheres – Classification.

Marteinsson, B. & Sigurjónsson, J. 2002, 'Corrosion of metals- mapping of the environment in Iceland', *Proc. The 9th International Conference on Durability of Building Materials and Components 9DBMC, Brisbane, Australia*, 17. - 21. March 2002, Paper 29

Marteinsson, B., Sigurjónsson, J., Stefansson, P. &. Sigurdsson, P. 2004, 'Atmospheric corrosion of metals in Iceland – three year results', *Proc. 13. Scandinavian Corrosion Congress, Reykjavik, Iceland*, 18. - 20. April 2004

Tidblad, J. & Kucera, V. 2003, *Mapping areas in Nordic countries with elevated risk of atmospheric corrosion*, KI Report 2003:4E, Korrosionsinstitutet, Stockholm, Sweden

UN ECE/ ICP Materials, Results and Dose/response functions, http://www.corr-institute.se/ICP-Materials/html/results.html, accessed 2004.03.03

Evaluating the Service Life of External Walls: a Comparison between Long-Term and Short-Term Exposure

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ABSTRACT

The goal of the paper is to present the latest results of an ongoing research programme conducted in the last decade by the Durability of Building Components' Group (DBCg) of BEST – Polytechnic of Milan, concerning the correlation over time between the degradation of building envelope components and the outdoor climate. The experimental program (conducted in cooperation between BEST and University of Applied Sciences of Southern Switzerland) requires on one hand the development of short–term laboratory accelerated aging test, and on the other hand a long–term outdoor exposure in order to verify the possibility of a time re-scaling. Research has been focused on the most external side of a typical Italian masonry, exposed in two towns (Milano–I and Lugano–CH) and covered with two different kinds of painting protection (acrylic and vinylversatic paints). During outdoor exposure, surface degradation has been checked by photos. This allowed us to gain information on different degradation mechanisms characterising the protective layers. Evolution of degradation has been also analysed comparing visible damage over time. Moreover, examination in the laboratory of thin sections, by means of optical microscopy, allowed us to correlate the increasing degradation level of samples under outdoor exposure conditions with the degradation of laboratory accelerated aging test. After the re-scaling activity, which is useful for creating a time correspondence between the accelerated aging cycles and the natural outdoor exposure in the two towns, it has been very important for us trying to evaluate the service life of samples. Estabilishing the end of the service life means knowing if the protective film is still able to provide the performance it has been designed for; as the main requirement of the paints is the waterproofing, disruptive tests have been carried out in order to highlight the fulfilment's degree of such a requirement. To date, analyses carried out from the results of the laboratory test, as well as the results on natural ageing exposure, have provided us the following information: a) Acrylic paints have shown a better degree of waterproofing, as regards those ones based on vinylversatic resins; b) The higher the resin ratio is, the better protection degree is; c) Time-rescaling for VH samples has been estabilished; in particular: 2 years ←→ 75 cycles and 4 years ←→ 150 cycles; d) Distruptive tests have confirmed that after 4 years of natural exposure (1999 – 2003) vinylversatic paints have reached the end of service life.

KEYWORDS

Degradation, re-scaling, service life, long – term exposure, short – term exposure.

1 INTRODUCTION

This paper presents the latest results of an ongoing research programme conducted in the last decade (see Daniotti *et al.* [1998], Maggi *et al.* [1999] Daniotti [2002, 2003]) by the Durability of Building Components' Group (DBCg) of BEST – Polytechnic of Milan, concerning the correlation over time between the degradation of building envelope components and the outdoor climate. This study is part of a wider experimental program (conducted in synergy between BEST -Polytechnic of Milan and University of Applied Sciences of Southern Switzerland) that needs on one hand the development of short – term accelerated laboratory ageing tests, and on the other hand a long – term outdoor exposure in order to verify the possibility of a time re-scaling (see Rigamonti [2002]). This term means the process through which it is possible to correlate laboratory weathering cycles with the "real" degradation times appeared from the monitoring on samples under natural outdoor ageing.

2 METHOD

The methodology is developed following the guidelines of ISO 15686-2:2001 "Building and constructed assets – Service life planning: Service life prediction procedures". According with the general approach specified by the Guide, comparison between long – term and short – term exposure results can be used in the assessment of the predicted service life of building components, once it has been verified that there is the same degradation level in each one of the samples. The complete method is fully described in Maggi *et al.* [1999]. This paper reports about the monitoring results, related to outdoor samples.

2.1 Brief description of the partial samples

The results of this experimental phase are related to partial samples composed of (from the inside): perforated brickwork coated with render and a protective layer of paint (see below for the different kinds of protections).

2.2 Variables

Several variables have been adopted, in order to gain as more information as possible. Particularly:

- *Different kind of painting protection (Acrylic paints, Vinylversatic paints)*: the aim is to study both the degree of waterproofing and the degradation's mechanisms;
- *Different degree of protection: resin ratio (PVC 40, PVC 60)*: Powder Volume Concentration, for its own definition, measures the powders percentage inside the paint's layer, so the resin ratio in the paint is given by the complement to 100 of the PVC-value;
- *Double exposure (Milano – I, Lugano – CH)*, to evaluate effects due to the air and rain pollution, through the behaviour of the samples in environments with similar climate conditions;
- *Different slope (90°, 45°)*: the 45° configuration should amplify the degradation and make much clearer the effects on the masonry.

So we are dealing with 8 different kind of samples for each esposure site (Milano and Lugano); in the following pages: **AH90** stays for "**A**crylic paint with **H**igh resin ratio (PVC40) on **90**-degrees sample". Similarly, we refer to other samples as AL90, VH90, VL90, AH45, AL45, VH45, VL45.

3 MASS INCREASE

No relevant information has been obtained in the last year of monitoring. The updating of the mass increase data over time has confirmed the behaviour of the samples: the monitoring of the mass increase highlights how 45-degrees slope represents a condition for an accelerated degradation just in the first period, whereas over time the different slope becomes a less important parameter in fulfilling the protective functions by the paints. More information is available in Daniotti, Iacono [2003].

4 SURFACE PHOTOGHRAPHS

During outdoor exposure, surface degradation has been assessed by photos. This allowed us to gain information on the different degradation mechanism characterising the protective layers (clause 4.1). Moreover, evolution of degradation has been analysed comparing visible damage over time (clause 4.2). As to be expected, the greatest degradation has been found on the 45° samples. The following figures will show the natural ageing process as regards to the considered paints.

4.1 Degradation mechanisms

4.1.1 AH – Acrylic paint with High resin ratio (PVC40)
Acrylic paints have the strongest resistence to environmental agents. Fig. 1 shows how the protective film on a AH90 sample is still regularly spread and lacking of blisters after almost 4 years of natural outdoor exposure. On the surface are visible small cavities, whose presence is due to air blisters formed inside the paint during its application.

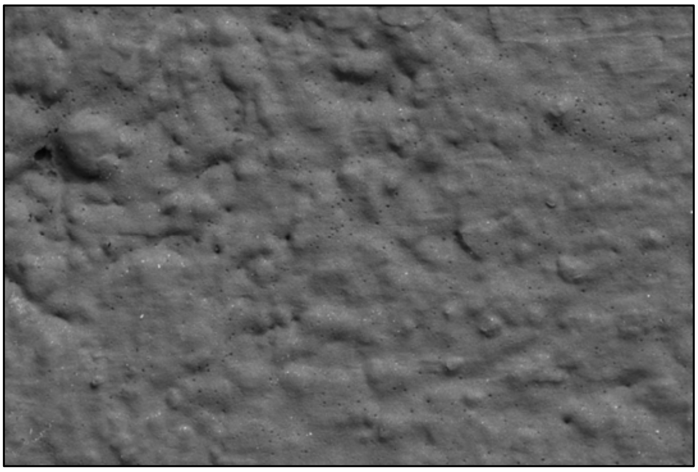

Fig. 1. Sample covered with acrylic paint (AH90) – Milano, April 9th, 2003

4.1.2 AL – Acrylic paint with Low resin ratio (PVC60)
A lower resin ratio makes the protective layer more subjected to crackings (see Fig. 2); nevertheless, these phenomena are still localized, and the rest of the film appears undamaged, which is in accord with lab results after a large number of cycles (Teruzzi and Jornet [2002]).

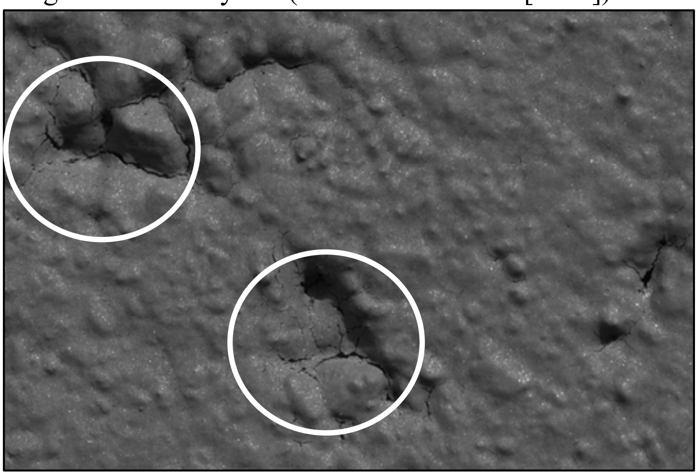

Fig. 2. Sample covered with acrylic paint (AL90) – Milano, April 9th, 2003

4.1.3 VH – Vinylversatic paint with High resin ratio (PVC40)

The degradation mechanism of vinylversatic paints appears as quite different.

Fig. 3 shows the degree of the protection layer in the sample covered with vinylversatic paint (PVC40) on April 2003; surface blistering has already appeared, but no cracks are present and the protective film is still unbroken.

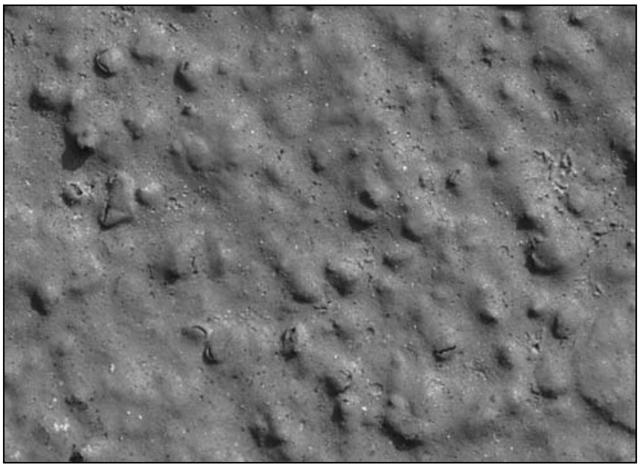

Fig. 3. Sample covered with vinylversatic paint (VH45) – Milano, April 9th, 2003

4.1.4 VL– Vinylversatic paint with Low resin ratio (PVC60)

The use of vinylversatic paints with high PVC leads to a different kind of degradation (Fig. 4): micro-cracks appear placed along the whole surface, evolving into a close net, and consistent with the presence of blisters.

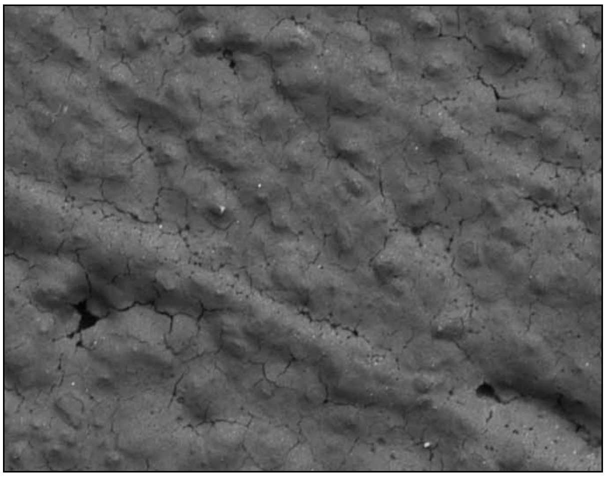

Fig. 4. Sample covered with vinylversatic paint (VL90) – Milano, April 9th, 2003

Exposure on two geographical sites (Milano and Lugano) has validated the different behaviour models of paints; moreover, accelerated aging test had also predicted two different degradation mechanisms, as one can see from figures 5 and 6 (see also Daniotti [2002]).

The most evident degradation has been found in the samples covered with vinylversatc paint, whereas acrylic films have shown to resist quite well environmental loads, even in the 45-degrees configuration.

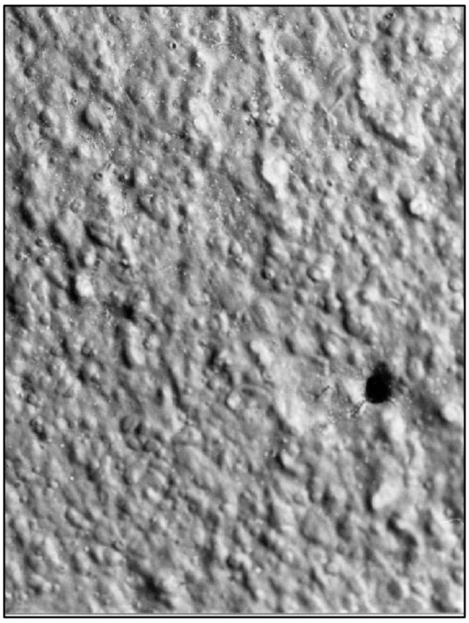

Fig. 5. VH sample – 150 cycles

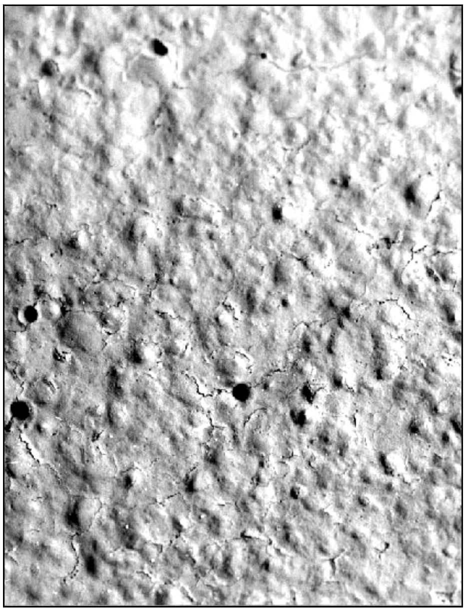

Fig. 6. VL sample – 150 cycles

Summarizing (Tab. 7):

DEGRADATION MECHANISMS			
ACRYLIC PAINT		*VINYLVERSATIC PAINT*	
AH (PVC40)	*AL (PVC60)*	*VH (PVC40)*	*VL (PVC60)*
Slight presence of blisters	Localized cracks	Broken blisters, lacerated bumps	Close net of micro-cracks

Tab. 7. Degradation mechanisms of the paints

4.2 Evolution of the degradation

Monitoring over time, besides providing information on the different degradation mechanisms, allowed us to focus the attention on evolution of the degradation, as shown by the following figures (Bazzi [2004]). In Fig. 8 one can see the situation of a VL90 sample in April 9th, 2003 (Milano).

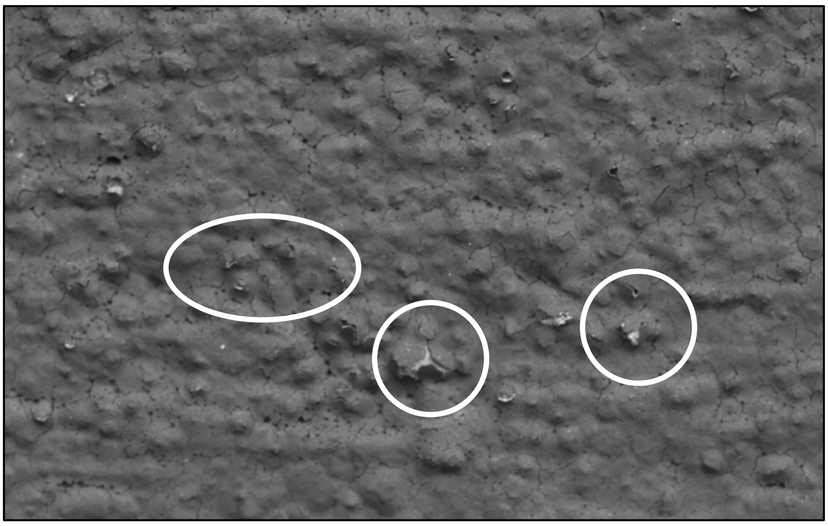

Fig. 8. Sample covered with vinylversatic paint (VL90) – Milano, April 9th, 2003

One year later, the sample shows rather high losses of the protective layer, with further openings of blisters, through which the plaster is clearly visible below (Fig. 9).

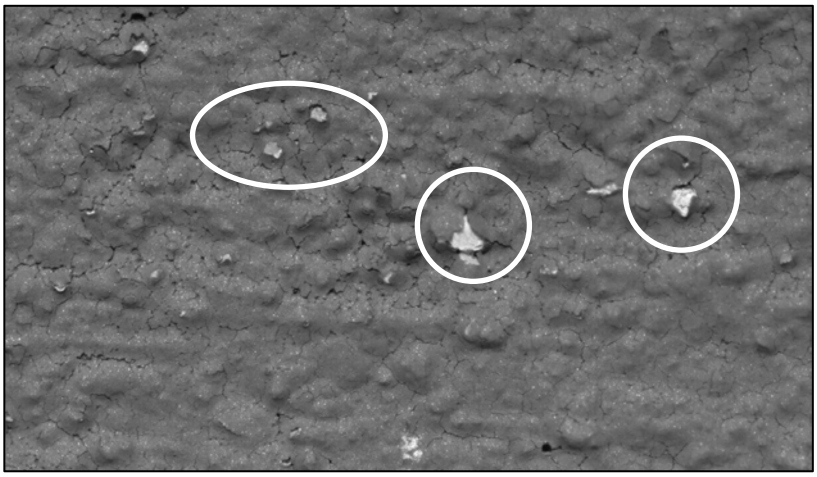

Fig. 9. Sample covered with vinylversatic paint (VL90) – Milano, March 29th, 2004

4.3 Re-scaling

Examination in the laboratory of thin sections, by means of optical microscopy, allowed us also to correlate the increasing degradation level of the samples put under outdoor exposure conditions with the ones in the laboratory accelerated ageing test.
The analyses have been conducted after 0, 75, 150 and 325 cycles; an increasing degradation level of the external plaster's surface (as the number of cycles increases) has been observed on a VH sample. Further visual inspections of the samples showed that, after 75 cycles (see Fig. 10) bumps and blisters had started appearing.
These are accompanied by detachments and ruptures after 150 cycles (see Fig. 11).

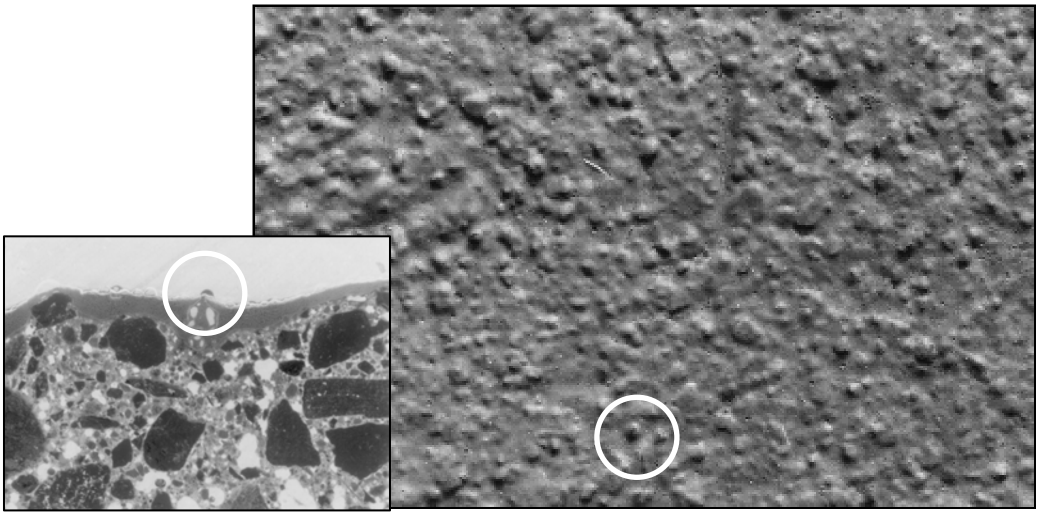

Fig. 10. Comparison between VH sample at 75 cycles and VH45 after 2 years (June 1999 – May 2001): blisters start appearing

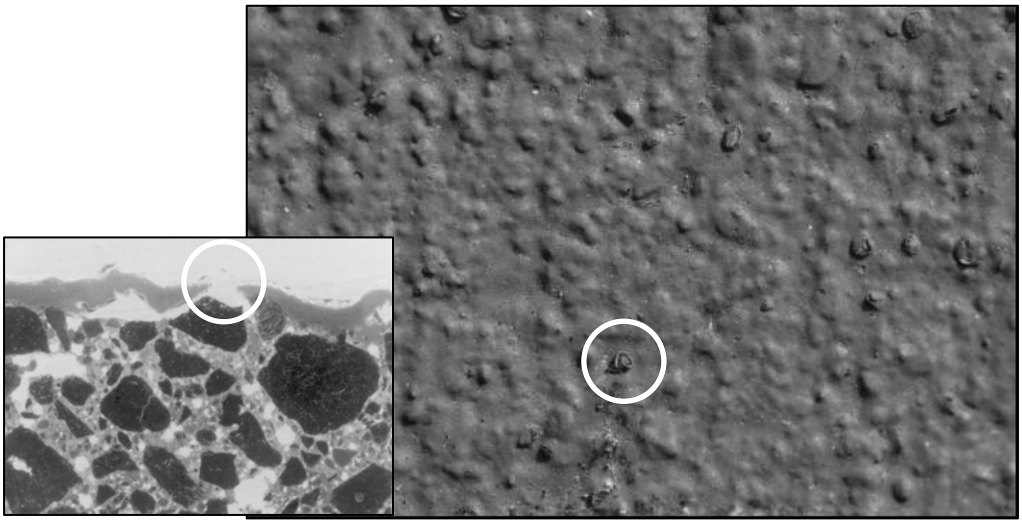

Fig. 11. Comparison between VH sample at 150 cycles and VH45 after 4 years (June 1999 – April 2003): lacerated blisters and ruptures

5 EVALUATION OF THE SERVICE LIFE

After the re-scaling activity, which is useful for creating a time correspondence between the accelerated aging cycles and the natural outdoor exposure in the two towns, it has been very important for us trying to evaluate the service life of samples. Estabilishing the end of the service life means knowing if the protective film is still able to provide the performance which it has been designed for; as the main requirement of the paint is waterproofing, disruptive test have been carried out in order to highlight the fulfilment's degree of such a requirement.
The histogram of Fig. 12 shows the outcomes related to the water absorption coefficient for 45-degrees samples, exposed in Lugano.

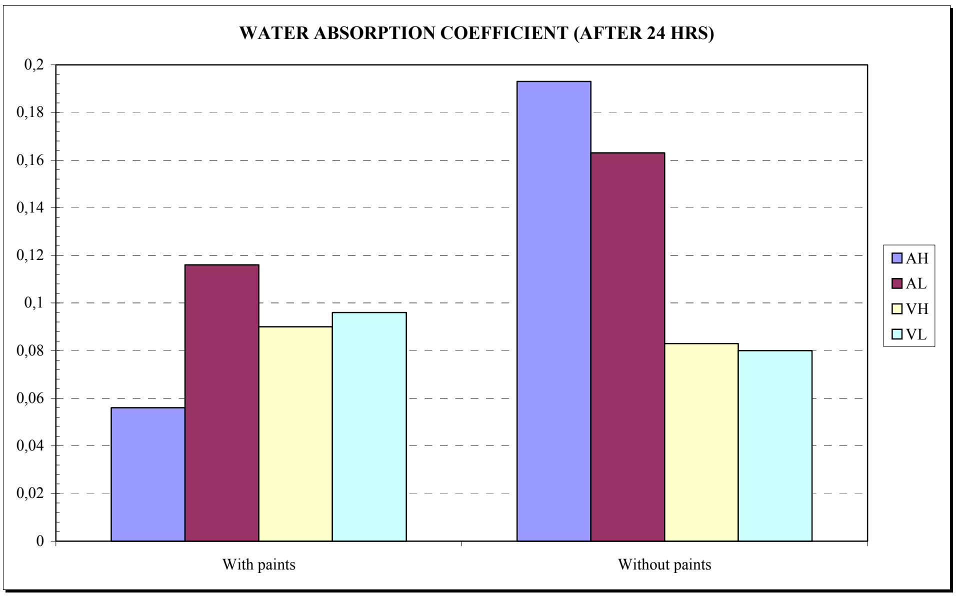

Fig. 12. Water absorpsion coefficients (45° samples)

First, test have been carried out on original samples; then, the same test have been repeated (on the same samples) once removed the surface layer, necessary to avoid the entrance of water into the technical solution. As one can see, samples covered with acrylic paints clearly show worse behaviour without the protective layer; this indicates that the waterproofing function is still being performed in a significant way by the external skin.
On the contrary, in the case of samples covered with vinylversatic paints, moving between the two conditions highlights that the end of service life has already been reached by paints, and that they are unable to provide a right water protection.

6 CONCLUSIONS

The latest results of an experimental program carried out since May 1999 at the BEST Department – Polytechnic of Milan have been described.
The analysis carried out from the results of the laboratory tests, as well as the results on natural ageing exposure, provided us the following information:

- Acrylic paints have shown a better degree of waterproofing, as regards those ones based on vinylversatic resins:
- The higher the resin ratio is, the better protection degree is;
- A time-rescaling for VH samples has been estabilished; in particular:
  - 2 years ←→ 75 cycles;
  - 4 years ←→ 150 cycles;
- Distruptive test have confirmed that after 4 years of natural exposure (1999 – 2003) vinylversatic paints have reached the end of service life, evaluated by laboratory and outdoor aging .

7 REFERENCES

ISO 15686-1:2000, *Building and constructed assets – Service life planning: General principles*.

ISO 15686-2:2001, *Building and constructed assets – Service life planning: Service life prediction procedures*.

Daniotti, B., Re Cecconi, F., Rigamonti, G., Poli, T., 1998. *Methodology and experimental programme to evaluate building components service life*, Proceedings of CIB World Building Congress 98, Construction and the environment, Gävle, Sweden.

Maggi P.N., Rejna M.G., Daniotti B., Re Cecconi F., Poli T., Rigamonti B., Jornet A., Teruzzi T. 1999, *Experimental program to evaluate building elements service life: first results on brickwork*, in Proceedings of 8th "Durability of Building Materials & Components" Conference, Vancouver.

Daniotti, B. 2002, *Performance characteristics' degradation and paints' protective degree for wall components subject to durability test*, in Proceedings of 9th "Durability of Building Materials & Components, Conference, Brisbane.

Rigamonti, G. 2002, *The systematic approach of an experimental to support service life prediction: a 4 years experience on external wall building components*, in Proceedings of 9th "Durability of Building Materials & Components" Conference, Brisbane.

Teruzzi, T. Jornet, A. 2002, *Artificial weathering of building components: experimental results on external walls*, in Proceedings of 9th "Durability of Building Materials & Components" Conference, Brisbane.

Daniotti, B. Iacono, P. 2003, *Long–Term Degradation under Outdoor Exposure Conditions*, in Proceedings of "Management of Durability in the Building Process" Workshop, Milano.

Bazzi, C. 2004, *La valutazione sperimentale della durabilità dei componenti edilizi*, degree thesis, Polytechnic of Milan, BEST Department.

IMPORTANCE OF THERMOGRAPHY IN THE STUDY OF ETICS FINISHING COATINGS DEGRADATION DUE TO ALGAE AND MILDEW GROWTH

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ABSTRACT

External thermal insulation composite systems (ETICS) for walls - expanded polystyrene insulation faced with a thin rendering - have been used in various countries since the mid twentieth century. In Portugal, these systems were first applied in the nineties. Unfortunately, the results weren't always as expected. In recent years, stains have frequently appeared on façades covered with ETICS as a result of algae and mildew growth. These stains compromise the façades' aspect and have been greatly contested by property owners. The thermal advantages of ETICS are, therefore, undervalued by owners due to the conspicuous degradation. This problem is intensified by poor building maintenance habits in Portugal.

It is essential to study the causes of algae and mildew growth in order to understand the hygrothermal behaviour of façades covered with ETICS. Dynamic heat exchanges between the environment and building components and temperature fluctuations of walls surfaces caused by radiation are two important aspects to be considered. During the night, energy exchanges occur between the ETICS' exterior layer and the atmosphere by long-wave emission. This causes surface cooling, condensation and higher surface moisture, therefore promoting the development of algae and mildew.

Infrared thermography can be used to determine the superficial temperature of objects. Detectors collect infrared radiation, transform it into electrical signals and create a thermal image based on the superficial temperature distribution. In this process, each colour represents a certain temperature range. Therefore, thermography may be a useful tool to evaluate surface condensation on ETICS.

To assess the use of thermography, some simple experiments were carried out at the Building Physics Laboratory (LFC) of the Engineering Faculty of Porto University (FEUP). A sensibility study was performed with LFC's equipment to evaluate the influence of emissivity on the measurements. By using this technology, it was also possible to study the wetting and drying process of building materials, since water evaporation is an endothermic reaction inducing local surface cooling. Theses experimental results are presented in this paper.

KEYWORDS
Infrared thermography, Hygrothermal behaviour, Building materials, ETICS.

1 INTRODUCTION

Studying hygrothermal behaviour is essential for evaluating the performance of building envelop, especially when new materials or components are used. These studies lead to improved technical solutions and regulations to ensure a building's durability and to guarantee user comfort and satisfaction.

In recent years, Portugal has seen a considerable increase in the use of external thermal insulation composite systems (ETICS) for walls - expanded polystyrene insulation faced with a thin rendering - although not always successfully. Most of the respective buildings, including the most recent ones, exhibit stained façades caused by algae and mildew growth. These stains compromise the façades' aspect and have been greatly contested by property owners. The thermal advantages of ETICS are, therefore, undervalued by owners due to the conspicuous degradation. This problem is intensified by poor building maintenance habits in Portugal.

Algae and mildew grow because of the hygrothermal behaviour of the ETICS' surface. During the night, heat exchanges occur between the ETICS' exterior layer and the atmosphere, by long-wave radiation. This causes surface cooling, condensation and higher surface moisture, therefore promoting the development of algae and mildew [Zillig *et al.* 2003].

Since thermography is a means of measuring superficial temperatures, it may be a helpful tool for understanding the causes of superficial condensation on ETICS. Cameras collect infrared radiation emitted by the surface, convert it into electrical signals and create a thermal image showing the body's superficial temperature distribution. In this process, each colour represents a specific temperature range (Fig. 1).

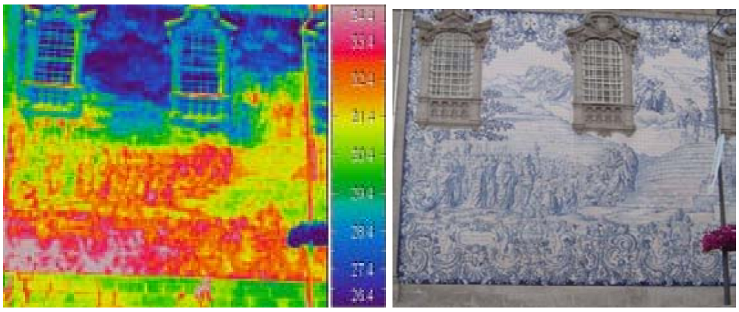


Figure 1. Thermogram and visible image of Carmo Church's ceramic façade in Porto.

The main objective of this work was to evaluate the applicability of thermography in the study of hygrothermal behaviour of building materials, especially of ETICS. To meet this goal, some simple experiments were carried out at the Building Physics Laboratory (LFC) of the Engineering Faculty of Porto University (FEUP).

The thermography equipment was subject to a sensibility study, using a climatic chamber with steady state conditions, to evaluate the influence of emissivity in the measurements.

The study made it possible to visualise the wetting and drying process of specimens, since water evaporation is an endothermic reaction inducing local surface cooling. This experiment ensured that condensation may be detected by the thermography equipment during ETICS field research activities.

2 LFC'S THERMOGRAPHY EQUIPMENT

LFC's thermography equipment consists on a detection section, a control section and a colour monitor (Fig. 2). The detection section scans and collects infrared radiations emitted from an object and converts them into electrical signals. The control section commands the scanning system and processes thermal signals according to user-selected measurement and display modes. The 12'' colour monitor provides a more accurate display of the thermal images.

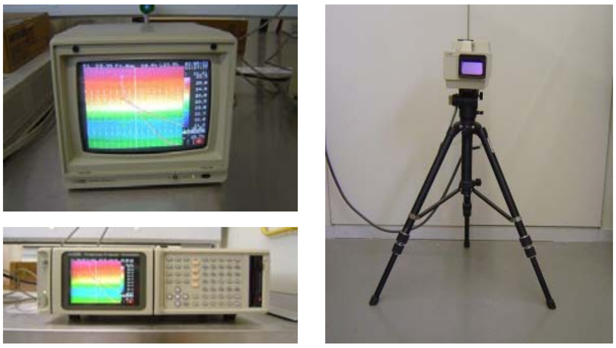


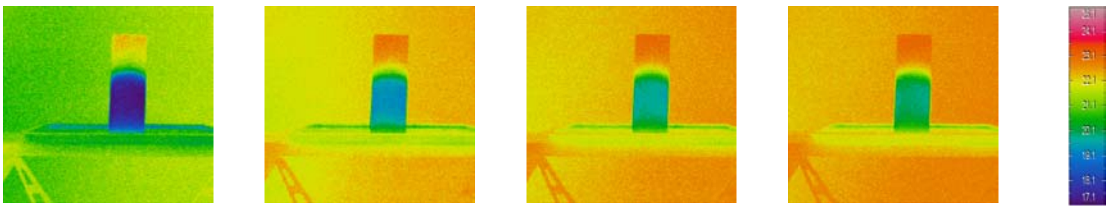
Figure 2. LFC's thermography equipment.

This equipment has an infrared detector, type Mercury – Cadmium – Tellurium (HgCdTe), cooled by liquid nitrogen. The detection section is sensitive to a wavelength ranging from 8 mm to 13 mm and measures temperatures ranging from -50º C to 2,000º C.

3 SENSIBILITY STUDIES

LFC's thermography equipment was subject to performance evaluation, both during laboratory and field experiments. In the laboratory, a cellular concrete specimen was submitted to partial immersion in water followed by a drying period under steady state conditions inside a climatic chamber. Thermal images were obtained during each test using four different values of emissivity: 0.62, 0.85, 0.91 and 0.95.

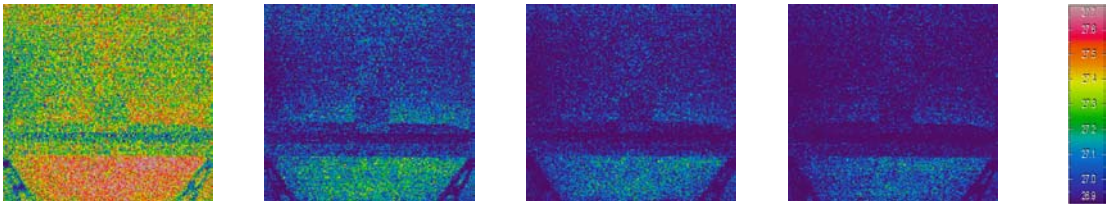
As expected [Chew 1998, Chown & Burn 1983, Hart 1991, Gaussorgues 1999], the results showed that emissivity variation induced changes in the thermal images, both during the absorption and drying periods (Fig. 3). Thermal images obtained with emissivity 0.62 were different from the other images. Differences among the other thermograms (emissivities 0.85, 0.91 and 0.95) were not very significant. However, thermal images obtained with emissivity 0.85 generally had clearer isothermals.



$\varepsilon = 0.62$ $\varepsilon = 0.85$ $\varepsilon = 0.91$ $\varepsilon = 0.95$

Figure 3. Thermograms at 168 hours (absorption).

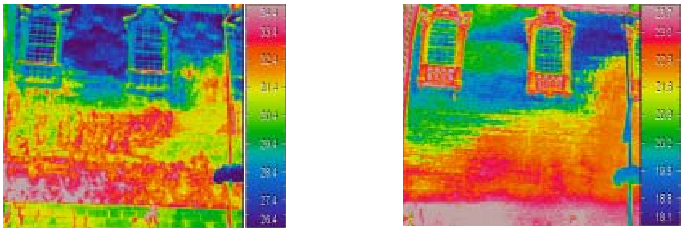
At the end of the test, the specimens' thermal image borders began to blend with the background (Fig. 4) as temperatures became similar. Only objects whose temperature varies at least 1º C from the environmental temperature can be detected using thermography. Therefore, this technology cannot be used to study objects in thermal equilibrium or in a hygroscopic domain [Hart 1991].



ε = 0.62 ε = 0.85 ε = 0.91 ε = 0.95

Figure 4. Thermograms at 792 hours (end of the drying period).

The influence of emissivity on thermographic measurements was also demonstrated when thermal images were obtained from the Carmo Church in Porto. Carmo Church’s east façade is covered with “azulejos” (hand-painted ceramic tiles). The colouring varies between white and several tones of blue (Fig. 1). The thermal images showed remarkable temperature variations caused by each colour’s different emissivity. That influence was greater when temperature differences were more pronounced. When temperature differences decrease at the end of the day (Fig. 5), the influence of emissivity became less important [Barreira & P. de Freitas 2004].



At midday (incident sun rays) At nightfall

Figure 5. Thermograms of Carmo Church in Porto, covered with “azulejos”.

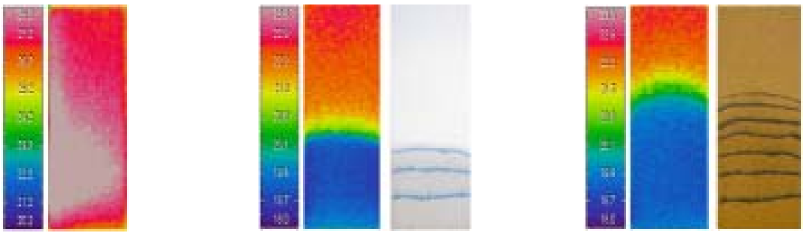
Although previous tests have shown that the influence of emissivity is relevant, if the study aims for a qualitative evaluation of the results, that is, an analysis of superficial temperature differences, the adopted value for emissivity is not very important [Hart 1991]. However, a wise selection of its value may simplify the thermal image’s interpretation.

4 CAPILLARITY ABSORPTION AND DRYING OF BUILDING MATERIALS

Capillarity water absorption and subsequent drying was viewed using a cellular concrete specimen, of 70 mm x 70 mm x 200 mm. The test was performed inside a climatic chamber, under steady state conditions of temperature and relative humidity (T = 20º C and RH = 60%). Several thermal images were obtained using emissivity 0.85.

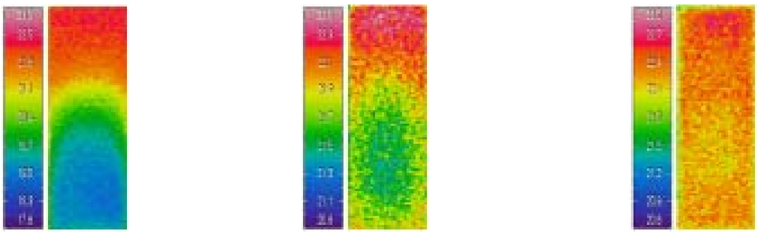
During the absorption period the water level in the specimen was visually observed and thermographically detected due to the superficial temperature variation (Fig. 6). This variation resulted from surface evaporation that, being an endothermic reaction, induced local cooling. The upper visible water level was shown as the line between the blue and green isothermals. The green and yellow isothermals showed a transition area between the “wet” and “dry” surfaces of the specimen [Laboratório Nacional de Engenharia Civil (LNEC) 2002].

The specimen was removed from the water at 407 hours and the drying period started immediately. Until the first 40 hours of drying, the thermograms still showed significant superficial temperature differences. However, during the drying period the colder area decreased as the green and yellow isothermals enlarged. This revealed faster drying at the top and vertical edges of the specimen. At the end of the test, superficial temperature was almost uniform and therefore moisture distribution was not perceptible (Fig. 7).



Dry specimen | Specimen at 34 hours | Specimen at 408 hours (end of absorption)

Figure 6. Thermograms obtained during absorption.



Specimen at 442 hours | Specimen at 578 hours | Specimen at 794 hours (end of drying)

Figure 7. Thermograms obtained during the drying period.

Thermography visually displays superficial temperature variations due to rising capillarity. During the drying period, it is also possible to detect superficial temperature variations. However, decreasing moisture induces small superficial temperature variations, thereby making it more difficult to detect these variations [Barreira & P. de Freitas 2004].

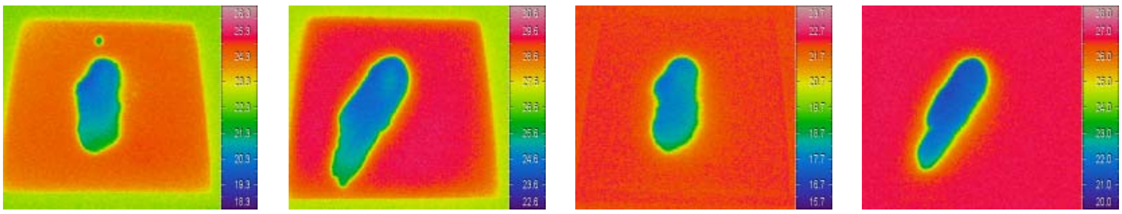
5 WETTING AND DRYING OF BUILDING MATERIALS

Wetting and drying were visualised using two cellular concrete specimens of 300 mm × 300 mm × 20 mm. Four different tests were carried out inside climatic chambers:

- Test G1: T = 20º C and HR = 60%;
- Test G2: T = 25º C and HR = 60%;
- Test G3: T = 20º C and HR = 40%;
- Test G4: T = 25º C and HR = 40%.

Each specimen was subject to water dripping for one hour, at a rate of 11 drops per minute. After the wetting, the drying period started immediately at the same environmental conditions. Thermal images were obtained during the tests using emissivity 0.85.

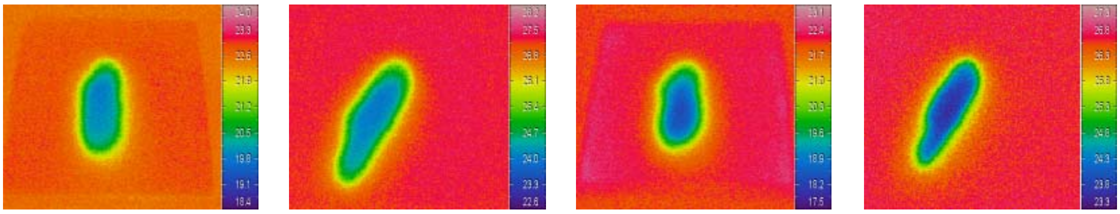
After a one-hour wetting, a stained area was visually observed and thermographically detected, due to a superficial temperature variation (Fig. 8). The green and yellow isothermals showed a transition area between the "wet" and "dry" surfaces.



Test G1 | Test G2 | Test G3 | Test G4

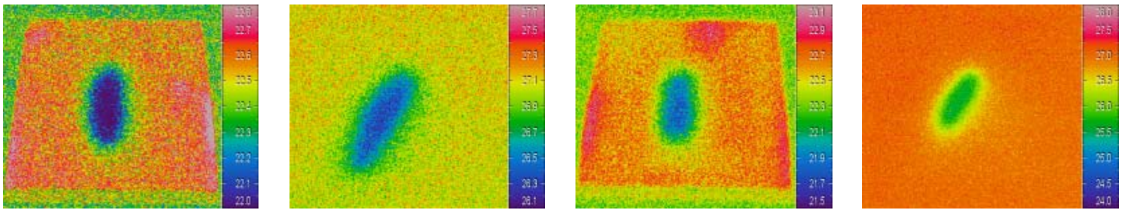
Figure 8. Thermograms after a one-hour wetting.

During the first day of drying, thermograms of the specimens still showed significant superficial temperature differences (Fig. 9). At the end of 48 hours of drying, the colder area decreased considerably as the green and yellow isothermals enlarged (Fig. 10). This revealed faster drying speed along the outer borders of the "wet" area.



Test G1 Test G2 Test G3 Test G4

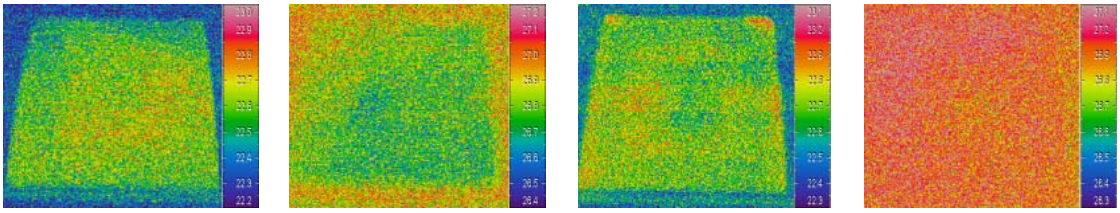
Figure 9. Thermograms after drying for 10 hours.



Test G1 Test G2 Test G3 Test G4

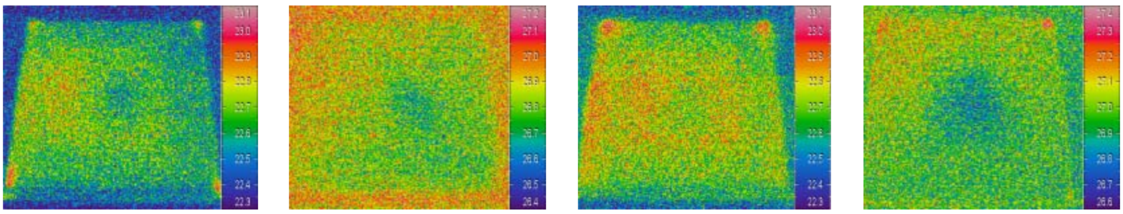
Figure 10. Thermograms after drying for 48 hours.

During the drying period, the specimen's temperature and relative humidity became similar to the environmental conditions, and moisture detection was hampered by the small superficial temperature differences. By this time the tests had been concluded (Fig. 11). However, at the end of the tests, the opposite side of the specimens still showed superficial temperature variation (Fig. 12).



Test G1 Test G2 Test G3 Test G4

Figure 11. Thermograms at the end of the tests.



Test G1 Test G2 Test G3 Test G4

Figure 12. Thermograms at the end of the tests (opposite side of the specimens).

Although thermography had made it possible to evaluate the specimens' approximate drying times, it could only detected superficial moisture [Barreira & P. de Freitas 2004].

6 STUDY OF ETICS' HYGROTHERMAL BEHAVIOUR USING THERMOGRAPHY

The laboratory test results showed that thermography may be useful to evaluate superficial temperature variations on ETICS, which cause condensation and higher surface moisture, thus promoting the development of algae and mildew.

Field research is now beginning. Thermal images were obtained from different façades covered with ETICS, some exhibiting algae and mildew, others without visible problems (Fig. 13). The thermal images indicate that areas with algae and mildew have higher superficial temperature variations. On the contrary, areas with no visible problems seem to have a more uniform superficial temperature.

Area exhibiting algae and mildew — Area without visible problems

Figure 13. Thermograms and visible images of a northern façade covered with ETICS.

In this first research it was also possible to assess defects detected on ETICS using thermography. This technology made it possible to view joints between the thermal insulation and the degradation of the exterior layers caused by mechanical impacts (Fig. 14). This is an important aspect to be considered, since those defects may mask superficial temperature variations responsible for condensation.

Joints between the thermal insulation — Degradation caused by mechanical impacts

Figure 14. Thermograms of a façade covered with ETICS.

These are only the preliminary conclusions. Many more thermal images of different buildings must be obtained to establish a relation between condensation and superficial temperature variations on the ETICS' exterior layer. We are collecting a series of 24-hour thermograms from façades facing different directions and in different months of the year, which will probably reveal critical condensation periods.

7 CONCLUSIONS

Research carried out at the Building Physics Laboratory (LFC) revealed that emissivity is an essential parameter, since it influences thermographic measurements and may restrict potential applications. However, if the study aims for a qualitative analysis, the adopted emissivity value is not very important. It was also possible to confirm that thermography is feasible only when there are significant temperature differences between the object and the environment. Therefore, this technique can't be applied to study objects in thermal or hygroscopic equilibrium.

It was possible to study the wetting and drying process of building materials using thermography. Temperature differences caused by surface water evaporation made it possible to recognize the "wet" outer limits. It was also possible to evaluate the approximate drying time of a material. However, thermography only detected superficial moisture.

The very satisfactory laboratory test results justified the first "in situ" thermographic tests on ETICS. These "in situ" tests led to the conclusion that evaluating the superficial water content in building components, although apparently feasible, requires more studies before being validated. The existence of some parameters that may affect the results call for precautions in interpreting the results, as moisture may be masked by the influence of exterior factors.

7 REFERENCES

Barreira, E. 2004, *Aplicação da Termografia ao Estudo do Comportamento Higrotérmico dos Edifícios*, Dissertação de Mestrado, FEUP, Porto.

Barreira, E. & P. de Freitas, V. 2004, *Infrared Thermography Applications in the Study of Building Hygrothermal Behaviour*, CIB W40 Meeting, Caledonian University, Glasgow.

Barreira, E. & P. de Freitas, V. 2004, *Realização de Termogramas à Fachada em Azulejo da Igreja do Carmo*, Relatório LFC/IC 146.2004 – LFC, FEUP, Porto.

Chew, M. Y. L. 1998, 'Assessing building façades using infra-red thermography', in *Structural Survey*, MCB University Press, Volume 16, N.º 2, pp. 81-86. http://pinkerton.emeraldinsigth.com/vl=23886247/cl=27/nw=1/rpsv/~1087/v16n2/s4/p81 (07/01/2002)

Chown, G. A. & Burn, K. N. 1983, 'Thermographic identification of buildings enclosure defects and deficiencies', in *Canadian Building Digest 229,* Canada, NRC-IRC. http://www.nrc.ca/irc/cbd/cbd229e.html (15/01/2002)

Gaussorgues, G. 1999, *La thermographie infrarouge – Principes, Technologies, Applications*, Edition TEC & DOC, Paris.

Hart, J. M. 1991, *A practical guide to infra-red thermography for building surveys*, BRE, Watford.

Jenkins, D. R., Knab, L. I. & Mathey, R. G. 1982, 'Laboratory studies of infrared thermography in roofing moisture detection', in *Moisture Migration in Buildings*, ASTM Special Technical Publication 779, ASTM, pp. 207-220.

Laboratório Nacional de Engenharia Civil (LNEC) 2002, *Aplicação laboratorial da análise termográfica para visualização da absorção capilar e da secagem de um provete de betão celular autoclavado*, Relatório 05/02 – NCCt, LNEC, Lisboa.

Laboratório Nacional de Engenharia Civil (LNEC) 2003, *Aplicação laboratorial de técnicas termográficas e de ultra-sons na detecção de deslocamentos localizados dos revestimentos duma parede de alvenaria de blocos de betão de argila expandida*, Relatório 88/03 – NRI, LNEC, Lisboa.

Zillig, W., Lenz, K. & Krus, M. 2003, 'Condensation on façades – influence of construction type and orientation', Research in Building Physics, Leuven, Belgium, 14-18 September 2003, pp. 437-444.

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Damage functions for wind induced building damage based on meteorological and insurance loss data in Norway

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ABSTRACT

Wind induced damages to buildings is a complex of several causes. Both design and craftsmanship are variables that will influence on the damage ratio of buildings exposed to strong wind. In this study, models for the general durability to withstand windstorms is presented for parts of the Norwegian building stock.

The general level of safety against wind damages in the coastal area of Norway is presented based on insurance loss data, meteorological data and data on the building stock. The study is restricted to yield the exposed areas of the coast of Norway because of the complex terrain further inland.

Two simple models are constructed. One describing the relationship between the wind velocity and the building damage and another describing the cost of damages related to wind velocity. Combining these two models, it is possible to estimate the cost of wind related damages in the present climate as well in a climate in change. The data suggests that the resistance to wind induced damages of the building stock is having a 10-22% too low capacity compared to the present wind load code.

In Norway, the design relies on the Norwegian building code, estimating maximum design wind speed. The wind speed is based on an annual probability of occurrence of 0.02 %. However, various climate scenarios indicate that the number of wind events is increasing in the costal area of Norway. A doubling of the frequency of wind events is expected for large areas. The climate scenarios are not yet conclusive on any possible increase in the wind velocity of such climate change.

The effect on wind induced building damage of an increase in the frequency of storms with 1-year return period is expected to be only 2.4% increase in total cost during a 50 year period. However, application of the developed models shows that 10% increases in the wind velocity during windstorms will more than double the financial cost related to wind damages on residential buildings.

A simple analysis shows that if cost is the determinant factor, adaptation to a more harsh wind climate will consist mostly of securing the secondary constructions.

KEYWORDS

wind, damage functions, modelling

1 INTRODUCTION

Analysing the building stock response to windstorms, it is necessary to assess if the building codes have appropriate level of safety to resist climatic impact. This information will also have direct implications for insurers insuring against natural hazards. It is also imperative to quantify the building response to climatic load when developing adaptation strategies to a possible climate change. Such quantified knowledge will be the basis of any socio-economic analysis and any pre-action towards adapting a building stock to resist a climatic change.

Wind induced damages to buildings may range from a torn off roof tile to a total collapse of the building structure. The causes of wind induced damages may be poor design, poor craftsmanship, poor maintenance or a combinations of these. Liu et. al. [1990] presents an array of weak points in wood framed houses. They found that "the most common damage to wood framed buildings is roof failure" and that "failure to distribute the loads from the fasteners to a large area of the covering is the principal cause of wind damage to covering". Nagethi [1996], presents a method of calculating the wind speeds that damage buildings. Based on information on the building condition and the building damage, he found a method for estimating the wind speed causing such damage.

Khanduri and Morrow [2003] constructed vulnerability curves for wood frame buildings based on insurance loss data from the hurricanes Hugo and Andrew in the US. They also presented a method for converting the curves to be valid also for other areas. The approach was to define a "mean damage ratio", D_u at wind speed u. D_u is the ratio of the cost to repair the damage, to the estimated value of buildings at that location. The sensitivity to the data accuracy is high, both in estimating the value of the building stock, and in the estimation of the insured value to the real value.

In Norway, the design load on buildings relies on the Norwegian building code, estimating maximum design wind speed. These are based on tabulated reference values for each municipality, based on a probability of occurrence of 0.02 %. The tabulated values are corrected for roughness, influence from the terrain and a general load coefficient dependent on the building part.

The topography in Norway is often very complex. This complex terrain results in a highly variable topographic exposure. Thus, the wind velocity will change dramatically over short distance. The extreme windstorms in Norway ranges from 30 to 50 m/s with a maximum measured gust wind of 62 m/s, measured at Smøla in 1992. Most storms in Norway can be classified in category 1 and 2 in the Saffir-Simpson hurricane scale.

2. THE HURRICANE OF 1st JANARY 1992

The hurricane in the western coast of Norway on 1st of January 1992 represents the highest wind velocities recorded on main land Norway. The total financial cost of the incident was approximately 2 billion Norwegian kroner[1] (NOK) in 1992. The loss related to buildings alone was approximately 1.3 billion Nkr. More than 29000 buildings were damaged, in some municipalities as much as 33% of the building stock was damaged. Approximately 20 % of the damages were related to impact from flying debris. This large number indicates that even if one building is designed to withstand the wind forces it is still vulnerable to damages. The buildings capacity to resist wind damages is therefore largely dependent on neighbouring buildings. On this basis, it is reasonable to present the general level of safety against wind damages in an area. Figure 1 shows the area were the hurricane struck together with estimated return period of similar wind episode.

[1] 100 Nkr = 12.2 €

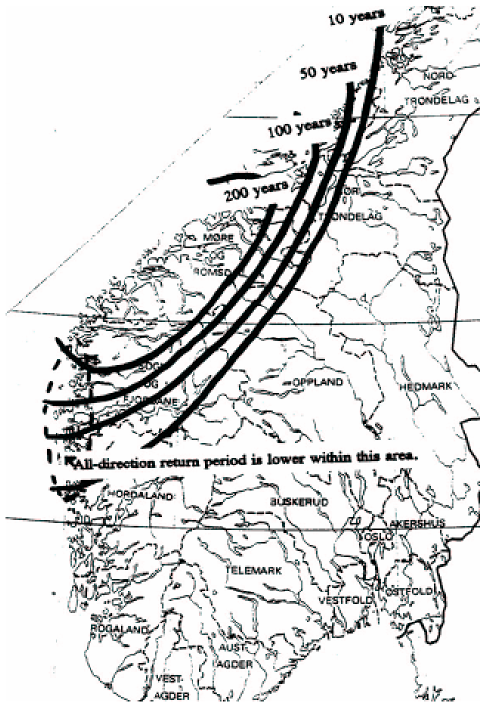

Figure 1. Geographical distribution of the gust return periods for areas exposed to wind fields from southwest to west, January 1. 1992. [Aune and Harstveit 1992]

3. NORWEGIAN WIND LOAD CODE

The Norwegian wind load code, NS 3491-4 “Design of structures - Design actions - Part 4: Wind loads”, is based on the European code ENV 1991-2-4 “Basis of design and action on structures – part 2-4: Actions on structures – wind actions”. The basis wind speed on a building site is found on the basis on a tabulated value for each municipality. This wind speed is multiplied with a number of factors to get the typical mean wind speed on the building site. For low rise buildings near the coast, the practice is to set most of the factors 1.0. The wind gust pressure is then found from eq. 1

$$q_{gust} = 0{,}5 \cdot \rho \cdot c_r^2 \cdot c_t^2 \cdot v_b^2\,[1 + 7(c_{tt} \cdot k_t / c_r \cdot c_t)] \qquad (1)$$

c_r is the roughness factor, c_t and c_{tt} are topography factors, all set to 1.0. The physical meaning of these assumptions is that the buildings are low rise and positioned close to the sea and that the buildings are not close to any steep topography. ρ is the density of air and v_b is the basis wind speed, in our case set to the tabulated wind speed. k_t is the tabulated roughness factor set to 0.17 for terrain close to the sea.

The design gust wind speed is the expected gust wind speed at the building site and it is found by taking the square root of the wind component from eq.1.

$$v_{gust} = \sqrt{v_b^{\,2} \cdot [1 + 7(c_{tt} \cdot k_t / c_r \cdot c_t)]} = 1.67 \cdot v_b \qquad (2)$$

Finding the wind speed at the building site, the wind pressure can be computed. The wind pressure is dependent on the shape of the building, thus building specific shape factors are introduced. To get the final design load, the load factor is introduced. This factor originates in the reliability code NS 3490 “Design of structures - Requirements to reliability” and ranges from 1.2 to 1.5 according to building type and part of the construction. The equivalent factors considering wind speed can be found by taking the square root of the load factor, i.e. $\sqrt{1.2}$ = 1.10 and $\sqrt{1.5}$ = 1.22 respectively.

4. DATA IN STUDY

The data in the study are collected from different sources. Meteorological data is from the Norwegian Meteorological Institute (1992), the building stock data is from the Norwegian Statistical Bureau and the damage and loss data is from the archives of the Norwegian Pool of Natural Perils. The study is restricted to yield the exposed areas of the coast of Norway. The reason for this is that the complex topography of Norway alters the wind and turbulence to a great extent. Measurements in areas away from the coast will be so much affected by the topography that it is valid only for a smaller area. In the more open costal area, the wind measurements are assumed to be valid for a larger area. The insurance data is only available on the geographical scale of a municipality, thus wind speed measurements in an inland municipality is regarded less representative than for an exposed coastal municipality.

Table 1 shows the data for 11 municipalities in the study. A total of 123714 buildings are included in the data material. 3245 of these buildings were exposed to by wind induced damages in the hurricane of 1. January 1992. The building damage ratio, $D_{m,}$ is here the ratio of damages in a municipality to the total number of residential buildings in that municipality.

Municipality	# Res. buildings	# Damages	Building damage ratio, D_m	Cost [Nkr]	Cost/ damage [Nkr]	Gust [m/s]	Mean wind [m/s]	v_b [m/s]	v_g [m/s]	Normalised gust G_n
Smøla	986	231	23.4 %	5815737	25176	62	46	30	50	1.24
Giske	2120	355	16.7 %	8493277	23925	55	36	30	50	1.10
Herøy	2825	426	15.1 %	8084050	18977	62	46	30	50	1.24
Frøya	1691	243	14.4 %	3711451	15273	55	40	30	50	1.10
Kristiansund	7342	906	12.3 %	18930114	20894	50	33	30	50	1.00
Ørland	1900	137	7.2 %	3529317	25761	49	37	30	50	0.98
Molde	8763	630	7.2 %	11947616	18964	54	-	29	48	1.12
Leka	323	3	0.9 %	19324	6441	43	33	29	48	0.89
Fedje	267	2	0.7 %	4808	2404	43	30	30	50	0.86
Bergen	94035	311	0.3 %	2062123	6631	28	-	26	43	0.65
Farsund	3462	1	0.0 %	1469	1469	29	21	28	47	0.62

Table 1. Data in study

The gust and mean wind speeds is the measured wind speed from meteorological stations on lighthouses and airports. v_b is the basis wind speed, tabulated in the wind load code and v_g is the gust wind speed derived from the equation (2). The normalised gust is the ratio of the measured gust to the gust wind speed derived from the Norwegian wind load code, v_g.

Figure 2 shows the building damage ratio plotted against the measured gust.

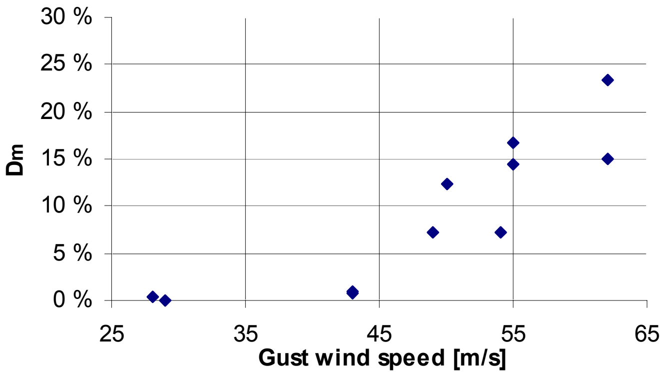

Figure 2. Building damage ratio plotted against measured wind speed.

From analysis of the data two regression models develop. This is the model relating the percentage of damaged buildings to the normalised gust, figure 3 and the model relating the cost of a single damage to the normalised gust wind speed, figure 4.

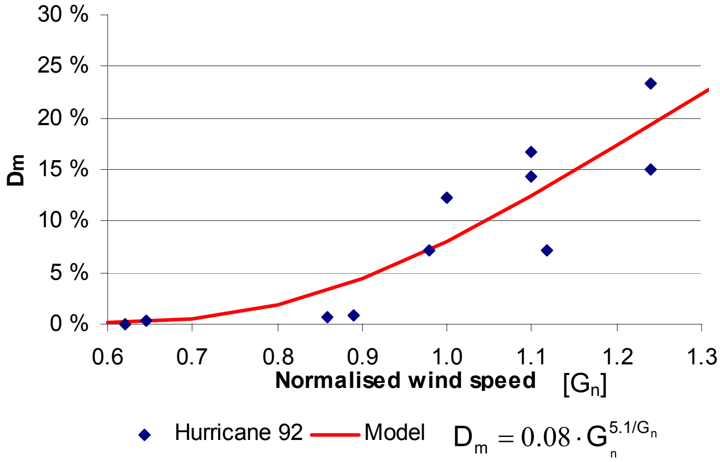

Figure 3. Building damage ratio, D_m in exposed Norwegian municipalities when exposed to wind speeds normalised with the wind speed derived from the Norwegian wind load code.

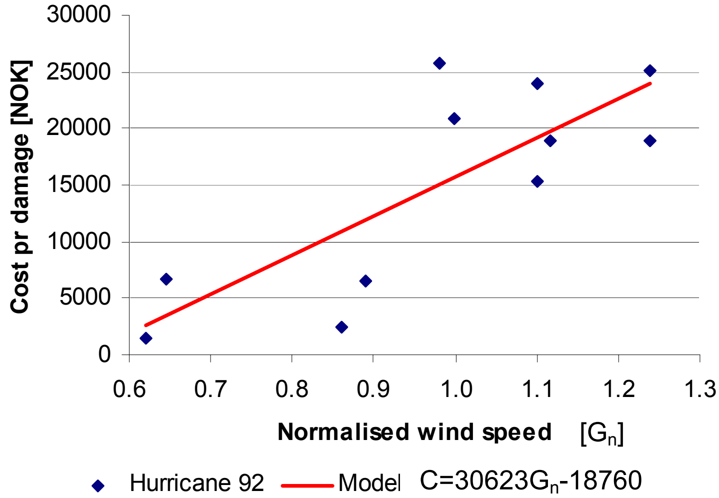

Figure 4. Cost pr. building damage in exposed Norwegian municipalities when exposed to wind speeds normalised with the wind speed derived from the Norwegian wind load code.

5. DATA AND MODEL ANALYSIS

The data in figure 2 can be interpreted as the real level of resistance to wind induced damages of the building stock. The overall wind load the building stock can resist is a gust wind speed of 44-46 m/s before the damage ratio exceeds 5 % in a municipality.

Figure 3 shows the percentage of damaged buildings plotted against the normalised gust. This can be interpreted as the level of resistance to wind induced damages of the building stock relative the resistance given in the wind load code. I.e. 95 % of the buildings in the data can resist a wind speed equal to the wind speed in the code. The data in figure 3 resemble an exponential relation. The model $D_m = 0.08 \cdot G_n^{5.1/Gn}$ fits the data with a standard error of 0.038 and a correlation coefficient of 0.89. The coefficients 0.08 and 5.1 have a standard error of 0.017 and 1.49 respectively. The model showing the relation between the damage level and the normalised gust wind velocity, G_n, shows that when G_n is approximately 1, the level of damage is below 5%. When G_n increases, the level of damage increases dramatically. A physical interpretation of this is that most of the damages in a building stock happens when the wind speed increases over the design wind speed taken from the wind load code. Relating this to the load factor in the reliability code NS3490, it is reasonable to define the capacity of the building stock in the study to resist gust wind to be approximately 1.0. In the reliability code, the load factor related to wind speed is ranging from 1.10 to 1.22 dependent on the building type and part of construction. Thus it is reasonable to describe the building stock resistance to wind induced damages as having a 10-22% too low capacity compared to the present wind load code.

A theoretical boundary value for the data is when all buildings have been damaged by the wind. This is assumed to happen at much higher wind velocities than recorded and the curve is assumed to follow the exponential shape at least until G_n is well above 2.0.

The data and model of the specific damage cost is showed in figure 4. The model is fitted by the linear function $C = 30623 \cdot G_n - 18760$. The standard error is 5963 and the correlation coefficient is 0.79. The coefficients 30623 and –18760 have a standard error of 7930 and 8947 respectively. The linear relationship indicates that the higher the wind speed, the more expensive is the damage. This seems logical in that the damage is related to the whole building and not to the specific building part. In the data a damaged roof tile count as one damage. A damaged roof tile together with a broken window, still counts as one damage. There seems to be a general trend in the data that the damages in more densely built areas are more expensive than damages in rural areas. This is, however, not reflected in the linear model. The cost of the damage is also believed to be dependent on the precipitation in the following few days after the wind incident.

Strictly, both the models are valid only for the building stock exposed to the hurricane in 1992. Since then the quality of the buildings might have changed. Also, after the repair of damaged buildings one must assume that at least some of the buildings have been restored to a state that was better than before the damage. However, the number of new buildings in a municipality is in general less than 1% pr. year, thus the models are assumed to be valid also at present.

6. APPLICATION OF THE MODELS

The models can be used to estimate the number of wind induced damages in a municipality and the total cost of such damage connected to storm events. To do this exercise, statistical data from the municipality is required. The necessary parameters are the total number of residential buildings and data on the gust wind velocity. Knowing the 50-year return period wind velocity, Eurocode 1 gives the wind velocity in years with different return period with equation 3.

$$U(p) = \sqrt{\frac{1 - K \cdot \ln(-\ln(1-p))}{1 - K \cdot \ln(-\ln(0.98))}} \cdot U_{50} \qquad (3)$$

here K is based on factors from the extreme value analysis. K=0.2 is specified in Eurocode 1 as a representative value. $U(p)$ is the wind velocity with the annual probability of exceedense p. U_{50} is the wind velocity with a return period of 50 years.

$$p = 1 - \exp(-\nu \cdot t_1) \qquad (4)$$

here ν is the expected annual number of upcrossings and t_1 is the time interval. From equation 4 the total number of expected wind events could be found.

7. CLIMATE CHANGE

Various climate scenarios indicate that the number of wind events is increasing on the coast of Norway. A doubling of the frequency of the wind events with 1-year return period is expected for large areas in the northern Norway. In the western part of the country, an increase of 50% in the frequency of such events is expected [Iversen et.al., 2003]. The climate scenarios are not yet conclusive on any possible increase in the wind velocity of such events. However, it is expected that also the wind speed in the extreme events will increase.

Using the models it is possible to estimate the cost of the climate change. The effect on wind induced building damage of an increase in the frequency of storms with 1-year return period is expected to be only 2.4% increase in total cost during a 50 year period. If, however, wind episodes with a longer return period will occur more often, the cost will increase much more. A 50% increase in the frequency of all wind events, not only the minor, will increase the total cost with approximately 50 %.

A 10% increase in gust wind velocity will result in an increase of 130% in the total cost during a 50-year period.

The cost of the tree different climate change scenarios is summarised in table 3.

	Increase of cost during 50 years (without adjustments for price growth)
50 % increase of frequency of 1 year storm	2.4%
50 % increase of frequency of all storms	50%
10% increase in gust wind velocity, same frequency of storms as today	130%

Table 2. Increase of cost of wind induced damages associated with climate change in a town in western Norway.

7. ADAPTATION TO CLIMATE CHANGE

The analysis of the damages after the hurricane in 1992 showed that 20 % of the damages was related to impact from flying debris. Secondary constructions are the part of the building with the lowest load factor, and it is assumed that such damages are relatively more common in lower wind velocities. An increase of the capacity of secondary constructions to resist wind induced damages will increase the overall safety factor. The cost induced by wind velocities with 10-year return period or lower represents 41% of the total cost during 50 years. In a wind climate with 10% higher wind velocities, this number is 51%. These are probably mainly damages to secondary constructions. Securing the secondary constructions better will reduce the damages accordingly. From this simple analysis it is evident that if cost is the determinant factor, adaptation to a more harsh wind climate will consist mostly of securing the secondary constructions.

8. CONCLUSIONS

The data in the study suggests that the resistance to wind induced damages of the building stock is having a 10-22% too low capacity compared to the present wind load code. This code was introduced in 2002.

Modelling the damage ratio and the cost related to wind induced damages, it is possible to model the cost in today's climate as well as the climate after a climate change. Modelling cost in the case of increased frequency of storms with 1-year return period suggests that the effect of such a climate change is not very extensive for a city on the coast of Norway. However, an increase of the gust wind velocity of 10% will have large effects on the building stock and repair cost.

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TT2-124, Damage functions for wind induced building damage based on meteorological and insurance loss data in Norway,T.K. Thiis, H. Einstabland and G.Krigsvoll

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10. REFERENCES

Aune, B., Harstveit, K. "The storm of january 1.1999", The Norwegian meteorological institute Report no. 23/92, Klima

Eurocode 1. (1995) "Basis of design and action on structures – part 2-4: Actions on structures – wind actions" ENV 1991-2-4

Iversen, T., Haugen, J.E., Sorteberg, A., Ødegaard, V., "Ekstremene nedjusteres" Cicerone no.5/2003

Khanduri, A.c, Morrow, G.C., 2003 "Vulnerability of buildings to windstorms and insurance loss estimation, JWEIA in press

Liu, H., Gopal, V.S., Nathegi, F., 1990 " Improving Wind resistance of wood-frame houses", Journal of Wind engineering and Industrial Aerodynamics, 36 (1990) pp.699-707

Nateghi-a, F, 1996, "Assessment of wind speeds that damage buildings", Natural hazards, 14. pp. 73-84

Standard Norway (2002) NS 3491-4 Design of structures - Design actions - Part 4: Wind loads

Standard Norway (2004) NS 3490 Design of structures - Requirements to reliability

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Influence of material quality and climate exposure on moisture condition of a wooden façade

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ABSTRACT

In the framework of the research programme *Climate 2000*, the Norwegian Building Research Institute is running field exposure tests of claddings on an experimental building in Trondheim, Norway. The eastern and western façades are divided into fourteen fields respectively, using different material qualities, painting systems and assemblies concerning building physics. The building is equipped with temperature and moisture measuring devices, continuously logging data since January 2004.
Data gathered from January 1, 2004 until July 31, 2004 are analysed and special attention is directed to the fields of the façade containing untreated wood. These fields contain fast and slow grown Norway spruce (*picea abies*) respectively, and the objective of this investigation is to study how different material qualities influence the moisture conditions of the cladding. In addition to this, the measured data were compared to data logged at the meteorological station located on the same site, in order to see if and how the moisture data mirror the climatic conditions.
At this point of time no clear conclusion can be drawn regarding the different moisture contents of the two material qualities. Though, the moisture content of the fast grown material seems to vary more than for the slow grown one. Crack formation due to greater variations in moisture content can be interpreted as a first sign of differences in the moisture condition of the two material qualities. This can lead to increased moisture absorption by and by.
Additionally, it was investigated if and to what extent the measured moisture content reflects meteorological parameters. It appears that by adding a time delay of 12 hours the goodness of fit, namely the coefficient of determination, can be improved significantly from an average of 0.5 to an average of 0.97. That means that 97 % of the measured values of moisture content can be explained by the chosen meteorological parameters.
The results provide useful information considering further investigation regarding the durability of wood materials exposed to climatic stresses.

KEYWORDS

Durability, Service Life, Wood, Material Properties, Climate Exposure

1 INTRODUCTION

As the concern and interest in sustainable materials and buildings increase, it is significant to acquire knowledge for innovative and enhanced use of wood as a renewable resource and sustainable material. A crucial issue for the application of wood as a sustainable material is the durability and service life. The standard ISO 15686 – *Buildings and constructed assets, Service life planning* [ISO 15686, 2000] contains general principles for the prediction of service life and maintenance planning. The standard contains the so-called factor method to determine the influence of different stresses on the service life of a building, material or component. The service life estimation is based on a Reference Service Life (RSL) and seven factors regarding the quality of components, the design level, the work execution level, the indoor environment, the outdoor environment, the in-use conditions and the maintenance level. Apart from determining the reference service life it is of interest to explore the factors themselves. The present study investigates the factors quality of components and outdoor climate.
At present studies carried out by Siemes [2003] or Moser [2002] focus on the further development of theoretical models presented in ISO 15686 [2000 & 2001]. Another important issue is the characterisation, mapping and modelling of the environment. Stresses and strains caused by weather as for example wind, rain and radiation are the main reasons for degradation of all kinds of building materials in outdoor use [Haagenrud [1997] and Högberg [2002]]. In terms of wood used as a building material Scheffer [1971] developed already in 1971 a map showing the risk of fungal decay for wood based on meteorological data. Newer research is directed towards the development of a prediction model for wood decay not only taking into account the environmental factors but also the design and shape of a building or object [Foliente et al. [2002]].
As a part of the ongoing NBI (Norwegian Building Research Institute) programme *Climate 2000 – Building constructions in a more severe climate* [Lisø et al. [2002]], an experimental investigation of wooden cladding design has been set up at a test building. The overall aim of the investigation is a better understanding of the connection between macro and micro climate conditions, the implications for the building stock in general and for wooden façades in particular.
The present study focuses primarily on investigations regarding the material quality of a untreated wooden façade and secondly on the characterisation of the climatic stresses it is exposed to.

2 MATERIALS AND METHODS

2.1 Test Site

The test site comprising the experimental building and a meteorological station (property of *The Norwegian Meteorological Institute* (DNMI)) is owned by the *Norwegian University of Science and Technology* (NTNU) and the *Norwegian Building Research Institute* (NBI). The test site is located in Trondheim, Norway (longitude 10°27'14'', latitude 69°24'40'', 129 m above sea level). Located in an open field, no surrounding buildings are likely to have an influence on the measurements.

The experimental building consists of a rectangular structure, the minor walls oriented towards south and north respectively so that the major walls are exposed to the east and west respectively. Each of the major walls is divided into fourteen fields, each of them 585 mm wide and 3200 mm high, each consisting of 27 horizontal boards.
Data are logged continuously since January 1, 2004, logging one value an hour. The present study considers data logged from January 1, 2004 until July 31, 2004.

2.2 Design

The present study is focused on two fields containing material left untreated i.e. without any painting, on the western side of the experimental house. Each board is 585 mm long, 142 mm high and 19 mm thick, moulded on both the upper and the lower edge to achieve a plane surface on the back of the cladding. Both inner and outer side are left rough sawn. Between the back of the cladding and the wind barrier a 23 mm gap guarantees the ventilation of the cladding boards. Furthermore, the gap serves the desiccation of the claddings' reverse side and the equalisation the pressure on the façade.

2.3 Materials

Norway Spruce (*picea abies*) in two different material qualities is used for all fields, slow grown material with an average density of 687 kg/m^3 and fast grown material with an average density of 574 kg/m^3. The following Table 1 lists the accurate values for the instrumented boards of the investigated fields.

		Density [kg/m^3]	*Moisture content [weight %]*
1	fguMt	565.3	18.6
2	fguMb	592.7	18.8
3	sguMt	631.5	18.0
4	sguMb	630.3	15.9

see Figure 1 or Table 3 for abbreviations

Table 1 Density and initial moisture content of the instrumented boards

2.4 Measurement Devices

Each field is instrumented with two temperature measuring devices and two moisture measuring devices respectively, located 550 mm from the top edge of the field, and 875 mm above the lower edge of the field, as illustrated in Figure 1.

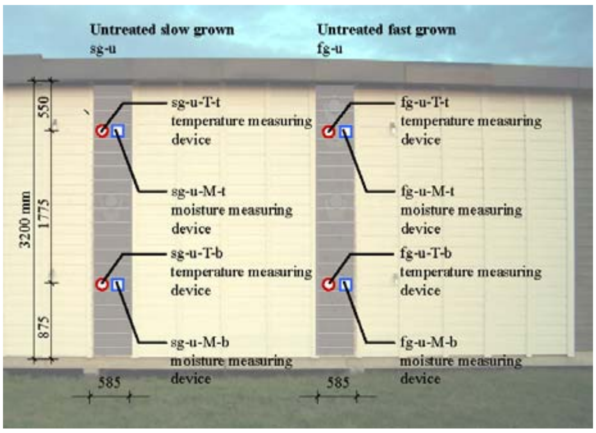

Figure 1 Instrumentation and location of measuring devices

The measuring device for the temperature consists of a type T thermocouple and a compensation cable, measuring the temperature at the back of the cladding. The maximum measuring range is -270 °C to 400 °C corresponding to a signal between 0 mV and 21 mV. The tolerance in the used area between -59 °C and 93 °C constitutes ± 1.1 °C.
The moisture content of the boards is measured using the regular impedance; the electrical resistance between two steel nails located 25 mm apart from each other. The two steel nails used for the measuring of the moisture content penetrated the whole cross section from the back ending 3 mm behind the surface of the board.

3 RESULTS AND DISCUSSION

3.1 Material quality

Data were analysed to detect differences in moisture conditions caused by material quality and the location of the measuring device in the field. The different data sets were compared and the mean monthly values and the respective standard deviation were calculated. Table 2 lists the monthly mean values and the respective standard deviations for the moisture content of the two untreated fields.

			Jan [%]	*Feb [%]*	*Mar [%]*	*Apr [%]*	*May [%]*	*Jun [%]*	*Jul [%]*	*Mean [%]*
1	fguMt	Mean	19.5	21.0	17.3	15.5	15.1	14.5	14.5	**16.8**
		StDev	1.05	1.48	2.21	2.18	2.85	2.31	1.24	**1.90**
2	fguMb	Mean	20.1	21.2	17.5	15.5	14.8	14.1	14.3	**16.8**
		StDev	1.20	1.30	2.20	1.90	2.30	1.80	1.10	**1.70**
3	sguMt	Mean	19.9	20.8	17.4	15.5	14.7	14.2	14.4	**16.7**
		StDev	1.06	1.13	1.95	1.60	1.92	1.62	0.96	**1.46**
4	sguMb	Mean	21.4	22.4	18.1	16.0	15.5	14.8	15.0	**17.6**
		StDev	1.40	1.40	2.20	1.70	2.20	1.60	1.00	**1.60**

1: **f**ast **g**rown, **u**ntreated, **t**op; 2: **f**ast **g**rown, **u**ntreated, **b**ottom; 3: **s**low **g**rown, **u**ntreated, **t**op; 4: **s**low **g**rown, **u**ntreated, **b**ottom

Table 2 Mean monthly values and respective standard deviations of the moisture content

The fast grown material (fgu) had an average moisture content of 16.8 % at both the measuring points on the top and bottom. Here, the respective slow grown material (sgu) had an average moisture content of 16.7 % and 17.6 % respectively. In this case, the standard deviation is a measure of the variation of the moisture content. However, the fast grown material is the one with greater variation in moisture content, namely 1.90 % at the top and 1.70 % at the bottom measuring point, compared to 1.46 % and 1.60 % for the slow grown material respectively.

Though, the fast grown material seems to have a certain tendency to vary more in moisture content than the slow grown one. The standard deviation of the moisture content is always greater for the fast grown material than for the slow grown material. Likewise, the increase of the variation of the standard deviation is greater for the fast grown than for the slow grown material. However, since there are only seven values available these results are not conclusive, even though this trend shows expected results.

3.2 Climate Exposure

Another interesting aspect is to investigate if and to what extent the moisture conditions of the cladding mirror the weather conditions. Therefore, the measured data were compared to the weather data logged at the meteorological station located on the same site as the experimental building.
Regression analyses were used to review how the different weather parameters affect the moisture content of the cladding material. The meteorological station logs altogether 28 different meteorological parameters; temperature, relative humidity, wind speed and direction, precipitation, air pressure, and global and long wave radiation. Some of the parameters are measured several times, for example as maximum, minimum and average value.
The first regression analysis with a set of example data revealed the most important parameters used for further investigations, listed in Table 3. Radiation, global radiation as well as longwave radiation, is omitted in the regression analyses since they correlate highly with the air temperature.

Symbol	*Meteorological parameter*	*Unit*
TTM	Temperature, average last hour	^{o}C
UUM	Relative humidity, average last hour	%
FM	Wind speed, average last hour	m/s
DM	Wind direction, belongs to FM	$^{o\,(*)}$
RT	Number of minutes precipitation last hour	0-60 min

$^{(*)}$ 0^{o} = north, 90^{o} = east, 180^{o} = south, 270^{o} = west

Table 3 Meteorological parameters, abbreviations and units

The first regression analyses of the moisture content show comparatively poor coefficients of determination, ranging from 0.04 to 0.12. To improve the fit of the regression the moisture content values were compared to weather parameters delayed by 1 to 240 hours.
For each measuring device of the cladding 240 regression analyses – that equals a time delay of 10 days - were carried out. The objective was to detect if the particular weather parameter influenced the moisture content immediately and to determine the dimension of a possible time delay. First, regression analyses have been carried out for all data together, second for the data of each month separately and third for special weather conditions (such as periods with a lot of rain or cold or warm periods).
The coefficient of determination is used to estimate how well the measured data correspond to the weather parameters. First, regression analyses were carried out for each of the parameters listed in Table 3.

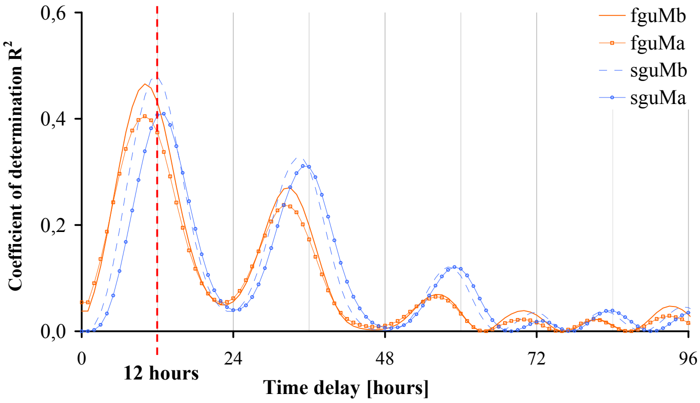

Figure 2 Coefficient of determination for the regression analyses of moisture content and relative humidity for July 2004

Figure 2 shows the connection between the coefficient of determination (R^2) and the time delay for all four measuring devices. The regression coefficient R^2 is frequently interpreted as the fraction of the variability explained by the independent variable, in this case the meteorological data. The measured moisture content for July 2004 is in this case compared to the relative humidity (*UUM*). It appears that the coefficient of determination reaches a maximum value at a time delay of approximately 12 hours. This way, R^2 can be increased from 0.06 to 0.47 for example for the measuring device *fguMb*.
All parameters show a significant increase in the values for R^2 from no time delay to a 12 hour time delay, though the maximum is not necessarily exactly at this point of time. In the case of no time delay the measured values are compared to the weather parameters logged at the same time, whereas a 12 hour time delay means that the measured values are compared to the weather parameter logged 12 hours earlier.
To investigate if and to what extent the goodness of fit increases by adding a time delay of 12 hours, multiple regression analyses where conducted with all relevant parameters, *TTM*, *UUM*, *FM*, *DM*, and *RT*.

time delay	*fguMb*	*fguMt*	*sguMb*	*sguMt*
0	0.513	0.496	0.499	0.495
12	0.975	0.937	0.984	1.000
see Table 2 for abbreviations				

Table 4 Improvement of the coefficient of determination by adding a 12 hour time delay

It appears that the regression coefficient R^2 can be increased considerably; in average an improvement from 0.50 to up to 0.97 can be achieved, see Table 4.

The next steps will be first to investigate if the revealed assumption mentioned in 3.1 persists in data measured after July 2004. Second it will be studied if the results found by regression analyses in 3.2 also apply for data measured in the autumn and winter 2004/2005 to focus on further development of the characterisation of the micro climate.

4 CONCLUSION

The first investigated influence of the material quality and location of the measuring device on the moisture content of the untreated cladding boards did not reveal any significant difference between the four measuring devices. Usually, the fast grown material is supposed to absorb more moisture than the slow grown one, but this can not be affirmed at that point of time. However, the fast grown material appears to have a tendency to vary stronger in moisture content than the slow grown one. This can lead to greater dimensional changes in the cladding boards by and by, followed by the development of cracks. More moisture can be absorbed by these cracks, leading to increased moisture content of the fast grown material. Though, this is only a possible scenario of how the materials quality can gain importance by and by, at this early stage of the experiments such ageing effects can not be revealed clearly.

Second, the effects of different weather parameters on the moisture content of the cladding boards were investigated. The following four meteorological parameters seem to have an influence on the moisture content of the cladding boards: the average temperature *TTM*, the average relative humidity *UUM*, the number of minutes precipitation *RT*, the wind speed *FM*, and the wind direction *DM*. Regression analyses were carried out to reveal if the parameters influence the moisture content with a time delay. It appears that a time delay of 12 hours results in the best regression fit. The coefficient of determination can be increased from about 0.5 to 0.97 or even 1.0 in one case. Hence, the moisture content of the cladding can mostly be explained by the prevailing weather conditions.

The observation of the moisture content of different wooden materials over the years can provide valuable insight in ageing processes of wooden materials used above ground. Likewise, the characterisation of the micro climate results in better comprehension of the influence of weather parameters on the performance of the material. The advance of understanding the processes that influence the performance of building materials is an important step towards the operability of service life prediction models.

5 ACKNOWLEDGEMENT

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REFERENCES

Foliente, G.C., Leichester, R.H. & Cole, I. 1999, 'Development of a reliability-based durability design method for timber construction', Proc. 8th International Conference on Durability of Building Materials and Components, National Research Council Canada, Ottawa, Canada.

Haagenrud, S.E. 1997, *Environmental Characterisation including Equipment for Monitoring*, CIB W80/RILEM 140-PSL, Sub Group 2 Report, Norwegian Institute for Air Research (Norsk institutt for luftforskning, NILU), Kjeller, Norway.

Högberg, A. 2002, *Microclimate Load: Transformed Weather Observations for Use in Design of Durable Buildings*, Department of Building Physics, Chalmers University of Technology, Gothenburg, Sweden.

ISO 15686-1, 2000, *Building and Construction assets – Service Life Planning; Part 1: General Principles*, The International Organization for Standardization, Geneva, Switzerland.

ISO 15686-2, 2001, *Building and Construction assets – Service Life Planning; Part 2: Service life Prediction Procedures* , The International Organization for Standardization, Geneva, Switzerland.

Lisø, K.R., et al. 2002, *Climate 2000 – Building Constructions in a more severe climate; Programme 2000-2006*, Programme Description, Norwegian Building Research Institute, Oslo, Norway

Moser, K. 2002, *Engineering Design Methods for Service Life Prediction*, CIB W80/RILEM 175, SLM: Service Life for Buildings and Components, Task Group Performance based Methods of Service Life Prediction, Swiss Federal Laboratories for Material Testing and Research, Dübendorf, Switzerland.

Siemes, T. 2003, *Prediction of service life of building materials and components, Service life methodologies*, CIB W80/RILEM 175, SLM: Service Life for Buildings and Components, Task Group Probabilistic methods, TNO Building and Construction Research, Delft, The Netherlands.

Scheffer, T.C. 1971, 'A Climate Index for Estimating Potential dor Decay in Wood Structures Above Ground', in *Forest Products Journal*, Forest Products Society, Madison, USA.
pp. 25-31

Wall Cladding System Durability Lessons Learned from the Premature Deterioration of Wood-Framed Construction Clad with Exterior Insulation and Finish Systems (EIFS) in the U.S.

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ABSTRACT

Despite well-intended laboratory testing of individual materials and components, unanticipated and severe durability failures of building wall systems can occur when the system design does not adequately integrate the various materials and components within the wall. Wall systems often fail when the design does not anticipate the deterioration and failure of individual components (such as sealant) and minimize the adverse consequences of such localized failures.

Exterior Insulation and Finish System (EIFS), a.k.a. "synthetic stucco," is an exterior-wall-cladding system that had been used on mass masonry wall construction in Europe since the 1940s and in the United States since approximately 1970. From the mid-1980s to the early 1990s, use of EIFS cladding became common on wood-framed residential construction in the U.S. Despite prior laboratory testing of the cladding system, unanticipated and spectacularly rapid deterioration problems occurred with the wood-framed walls beneath the cladding, particularly in regions with significant rainfall and humidity, such as the southeastern U.S. The EIFS cladding itself was typically intact and undamaged; however, the EIFS cladding trapped leakage from windows, sealant joints, and other wall components, resulting in the rapid deterioration of the underlying structural wood framing and/or the wood- or gypsum-based sheathing.

Extensive durability testing of individual materials and components alone is not adequate to ensure the durability of a wall system. To be durable over the long term, wall system designs should consider in detail the integration of numerous wall components (e.g., windows, cladding, sealants, etc.), anticipate the degradation and failure of individual components (such as sealant), and design the system to accommodate such localized component failures and minimize their adverse consequences.

KEYWORDS

Building, cladding, system, durability, EIFS

1 INTRODUCTION

While laboratory testing of individual building materials and components is often worthwhile and necessary, it is essential to maintain a broader view of the potential interrelationship of various building materials and components and their interrelated roles in the wall system in order to assess and provide for the durability of the overall wall.

The case study of the durability failure of wood-framed walls clad with barrier EIFS in the United States serves as a lesson and caveat for those concerned with the durability of building materials, components, and systems. From this case study, broader lessons can be learned to help designers, code officials, testing professionals, and manufacturers avoid similar wall system durability failures in the future.

2 CASE STUDY: DURABILITY FAILURE OF WOOD-FRAMED WALLS CLAD WITH BARRIER EIFS IN THE U.S.

2.1 Background information

Exterior Insulation and Finish System (EIFS), a.k.a. “synthetic stucco,” is an exterior-wall-cladding system that had been used on mass masonry walls in northern Europe since the 1940s and in the U.S. since approximately 1970 [Williams &Williams 1994]. In its most common and conventional form, “barrier EIFS,” the cladding system is a surface-sealed barrier wall designed to exclude all precipitation at the outermost cladding surface. Barrier-EIFS wall systems in the U.S. generally consist of the following components, from exterior to interior: lamina (consisting of an acrylic finish coat installed over a glass-fiber-mesh-reinforced polymer-modified cementitious basecoat) and expanded polystyrene insulation board adhesively attached to the wall substrate (masonry). Masonry’s inherent resistance to degradation from incidental moisture provides the durability of the structural masonry wall, despite any defects in the surface-sealed barrier cladding that admit moisture to the masonry. Although some problems occured with barrier EIFS installed on masonry walls in the U.S. (notably debonding failure of sealants adhered to the finish coat and impact damage to the EIFS cladding near grade), these problems did not tend to result in failure of the overall wall-cladding system, in serious damage to other wall components, or in serious damage to the concealed masonry wall. Thus, as a wall system, masonry walls clad with barrier EIFS typically can accommodate localized damage or holes in the EIFS cladding and/or degradation or failure of other wall components (e.g., sealants) without serious adverse consequences such as overall wall-system failure or structural damage.

2.2 Laboratory testing of EIFS cladding and separate laboratory testing of windows

Prior to its acceptance by various building codes in the U.S., EIFS underwent a series of tests for weathertightness and durability. For example, the code-required independent testing under the 1993 International Conference of Building Officials Accceptance Criteria for EIFS [ICBO 1993] includes the following durability tests: accelerated-weathering tests (ASTM G23-81), freeze/thaw tests, salt-spray-resistance tests (ASTM B117), structural performance testing (ASTM E330-84, Procedure B), water penetration tests (ASTM E331), and water resistance tests (ASTM D2347). Additional testing was required for other considerations, such as structural performance, fire resistance, and impact resistance. Code-compliance testing of EIFS under ICBO 1993 required the EIFS test specimen to include control joints *if* control joints were used in the design; however, the testing did not require the inclusion of other wall components (e.g., windows, doors, conduit penetrations, deck or balcony framing members, etc.) within the EIFS test panels. Because other major wall components such as windows and doors are typically subject to separate individual laboratory tests, some may contend that their inclusion in the test panel for a wall-cladding system would be superfluous and an undue complication and burden on the wall-cladding test. Nevertheless, the omission of these other wall components from the EIFS test panels resulted in laboratory tests that failed to reveal the gross incompatibility between the design concept and plane of watertightness of surface-sealed barrier wall EIFS, and the non-surface-sealed design concept and plane of watertightness of common residential windows and doors.

Further compounding this incompatibility between surface-sealed barrier EIFS and common windows and doors was the definition of leakage under the water-penetration-test requirement for windows and doors. ICBO 1993 code-compliance-testing requirements of EIFS refer to ASTM E331, *Standard Test Method for Water Penetration of Exterior Windows, Curtain Walls, and Doors by Uniform Static Air Pressure Difference* [ICBO 1993 and ASTM E331-93 1993]. ASTM E331-93 defines water pentration as "penetration of water beyond the vertical plane intersecting the innermost projection of the test specimen, not including interior trim or hardware, under the specified conditions of air pressure difference acrosss the specimen." Under this definition, water passing through the window-frame corner outboard of the nailing flange is not considered water penetration ("leakage"). While water in this zone is controlled and accomodated in claddings that include a secondary weather barrier at the sheathing level (e.g., most traditional residential claddings in the U.S.), water in this zone would prove to be a major contributor to the rapid durability failures in wood-framed buildings clad with barrier EIFS.

2.3 Durability failure of wood-framed walls clad with EIFS

From the late 1980s to the early 1990s, use of barrier-EIFS cladding became common on wood-framed residential construction in the U.S. In the mid-1990s, despite laboratory testing of the EIFS cladding, unanticipated and spectacularly rapid deterioration problems occurred on many wood-framed walls beneath the barrier-EIFS cladding, particularly in regions with significant rainfall and humidity, such as the southeastern U.S. In many cases, significant localized deterioration (rot) of the wood sheathing and, less often, the wood framing occurred within the first five years after the house was constructed. In such cases, the EIFS cladding was typically intact and undamaged; however, the undamaged, intact cladding concealed significant wood deterioration.

Figure 1. Rapid durability failure of wood-framed wall clad with barrier EIFS in the southeastern U.S. EIFS cladding is removed at lower right to reveal deterioration of wood-based oriented strand board (OSB) sheathing. The house is fewer than five years old at the time of the photo. Note the localized deterioration of the wood-based sheathing (black areas) beneath locations of leakage at window-frame corners and window-perimeter sealant joints.

Several factors contributed to the rapid deterioration (i.e., durability failure) of wood-framed walls clad with barrier EIFS. In all wall claddings, as exposed materials and components weather and degrade, numerous entry points occur where water may bypass the outer cladding surface (Figure 2). From our field investigations and water testing, it appears that barrier EIFS is typically more effective than most traditional residential claddings at excluding precipitation at the outermost surface of the cladding, particularly in the "field" of the wall (away from other wall components). However, it is inevitable that some water will bypass the cladding, particularly at its juncture with other wall components such as doors, windows, roofs, vent and conduit penetrations, etc.

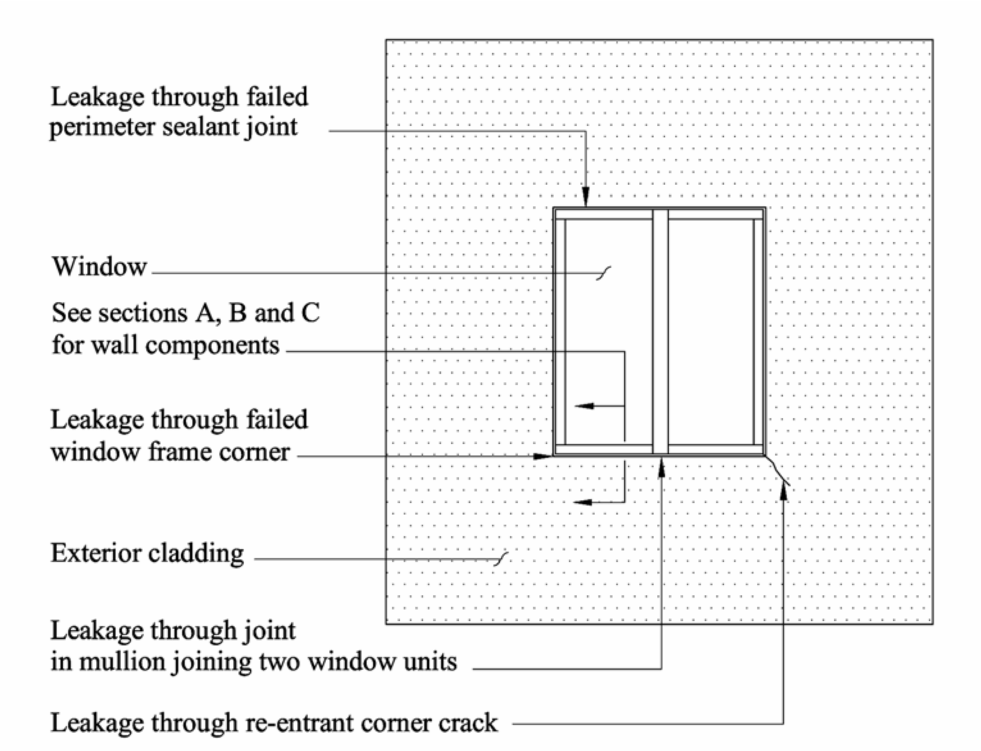

Figure 2. Several common entry points for water behind exterior cladding.

While all wall systems will admit some precipitation past the outermost cladding surface, the fundamental difference between traditional residential claddings and barrier-EIFS-clad walls is in their respective abilities to control precipitation that inevitably bypasses the cladding and in the resulting consequences of this incidental moisture ingress. Common, traditional residential claddings such as brick, stucco, wood clapboard, and wood shingles typically include a secondary waterproofing layer ("building paper") installed over the sheathing, while barrier EIFS does not (Figure 3). As a result, in claddings that include a secondary waterproofing layer, incidental moisture ingress (through joints or defects in the cladding, failed sealant joints, etc.) is controlled by the secondary waterproofing and flashings, so the wood-framed wall structure is protected from moisture, and the serious adverse consequences of incidental moisture ingress are minimized.

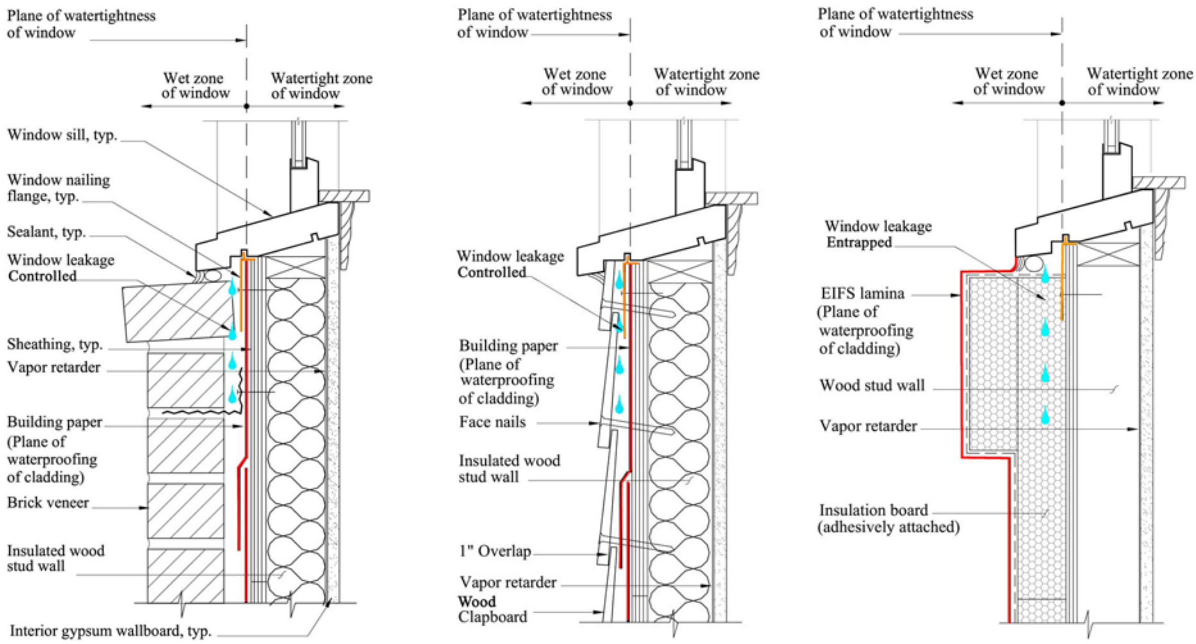

Figure 3. Verical sections through walls. Typical wood-framed wall shown with brick cladding (left), wood cladding (center), and barrier-EIFS cladding (right). Note the location of the plane of watertightness of windows assumed by certain test standards (shown in orange) with respect to the plane of waterproofing for cladding (shown in red). In brick- and wood-clad walls, the location of cladding waterproofing accomodates and controls typical window and sealant joint leakage (shown in blue) in the "wet zone," protecting the wood structure from water and decay. In EIFS cladding, the outer location of wall waterproofing with respect to windows and sealant results in a wall system that cannot accommodate typical window and sealant joint leakage in the "wet zone." When window or sealant-joint leakage inevitably occurs, moisture is trapped behind the EIFS cladding, promoting rapid deterioration of the wood-framed wall and sheathing.

In traditional residential claddings, the secondary waterproofing layer is shingled behind the nailing flange; thus, window leakage occuring between the nailing flange and the outermost portion of the sill (an area termed the "wet zone" by some wood window manufacturers) is accomodated by the secondary waterproofing. However, Figure 3 makes clear the incompatibility between the location of the wateproofing on a barrier EIFS cladding and the location of the watertight zone of common residential windows. With the 25–50 mm thickness of EIFS common at windows, the outermost surface of the EIFS (the cladding's sole waterproofing) is significantly outboard of the nailing flange of the window (the outermost plane at which the window is required to be watertight by certain test standards). Thus, water passing through the window frame corner outboard of the nailing flange (in the "wet zone" of the window) is behind the waterproofing plane of the EIFS, where it becomes entrapped within the wall system. The low vapor permeability of the EIFS cladding prevents this trapped moisture from readily drying, and the conditions are ripe for the rapid decay of the unprotected wood sheathing and framing [Bronski & Ruggiero 2000].

2.4 Broader lessons learned toward the design of durable wall systems

Laboratory testing of individual wall components, materials, and systems, such as windows, sealant, and EIFS, would not necessarily predict (and did not predict) the durability failures that occurred in EIFS-clad wood-framed walls in the U.S. However, this durability failure, and the inherent design incompatibility between barrier EIFS and common residential windows described in Figure 3 above, could have been predicted by critical visual review of the wall-system-design details by a professional experienced in wall-system design and forensics.

Masonry walls clad with EIFS, and wood-framed walls clad with brick or wood siding that include secondary waterproofing, have proved significantly more durable than wood-framed walls clad with EIFS primarily because the former can tolerate or accommodate the weathering, degradation, and localized failure of individual wall components and materials such as sealant and windows while minimizing the serious adverse consequences to the overall wall system, whereas wood-framed walls clad with barrier EIFS cannot.

3 CONCLUSIONS

Laboratory testing of the durability of individual wall materials and components is often useful and necessary, but testing of individual components and materials alone is not sufficient to assess the durability of a wall system. For assessing overall wall-system durability, critical review and assessment of design details by technically knowledgeable design and forensic professionals experienced with wall systems is also necessary.

To be durable over the long term, wall-system designs should consider in detail the integration of numerous wall components (e.g., windows, cladding, sealant, etc.), anticipate and accomodate the inevitable degradation and performance failure of individual components (such as sealant), and design the system to accommodate such localized component failures and minimize their adverse consequences.

4 ACKNOWLEDGMENTS

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5 REFERENCES

Williams, M. & Williams, B. 1994, *Exterior Insulation and Finish Systems: Current Practices and Future Considerations*, ASTM Manual Series: MNL 16, PCN 28-016094-10, American Society for Testing and Materials, Philadelphia, PA, p. 5.

International Conference of Building Officials (ICBO) 1993, “Acceptance Criteria for Exterior Insulation and Finish Systems” ICBO Evaluation Report, ICBO Evaluation Services, Inc., Whittier CA.

American Society for Testing and Materials (ASTM) 1993, “ASTM E 331-93 Standard Test Method for Water Penetration of Exterior Windows, Curtain Walls, and Doors by Uniform Static Air Pressure Difference” *Annual Book of ASTM Standards,* Vol. 04.07, pp. 521–524.

Bronski, M. & Ruggiero, S. 2000, “EIFS on Wood-Framed Residential Construction: Design Concepts to Avoid Common Moisture Intrusion Problems,” *Journal of Testing and Evaluation*, vol. 28, no. 4, pp. 290–300.

Durability of PVC Roofing Membranes - Proof by Testing After Long Term Field Exposure

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ABSTRACT

A leading supplier of thermoplastic roofing membranes inspected and removed samples from 44 different roofs in America, Austria, Canada, England, Germany and Switzerland. The roofs ranged in age from 9 to 34 years at the time of sampling. The dual European and North American survey is believed to be the most exhaustive ever conducted for PVC membranes.
The assessment on site was based on the general impression of the roof, constructive details, roof construction, surroundings, upstands, gutters, drains, connections and the status of the membrane seams. Comprehensive photographic evidence was collected and will be presented. For the laboratory evaluation a variety of physical properties were tested according to ASTM (USA), DIN (Germany) and SIA (Switzerland) standards. These properties are considered as essential in the estimation of the long term behaviour of plastic roofing membranes. Additionally, thermostability, glass transition temperature and hail testing were conducted on most of the aged samples. The paper will present the results of the testing. Relevant correlations which may exist between various physical properties, will also be looked at. These correlations may be useful in assessing the relevance of some tests in the context of material standards.
All of the 44 inspected roofs were fully functional and none was leaking. On none of them any repair work was necessary or advisory. The general impression on site was very positive. The laboratory evaluation of the field samples revealed the degree of material deterioration over the years. Although subject to a certain degree of aging, the majority of the determined material data revealed values better than the normative requirements for new materials. In conclusion the study proves the excellent durability of PVC roofing membranes in exposed applications.

KEYWORDS

roofing, pvc, longevity, weathering, waterproofing

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1 INTRODUCTION

Poly(vinyl chloride) – PVC, also known as vinyl - is one of the most versatile thermoplastics in use today. PVC roofing membranes can now look back to history of five decades of use in Europe. All roofs are expected to provide decades of problem free service. When new products are developed and introduced, there is little knowledge of how they will age beyond data generated in accelerated artificial weathering tests. Although testing the physical properties of new materials can be useful in trying to compare and even rank them against other similar products, nothing is more useful or informative than actual field experience [1].

Physical properties of all roof systems change with age and outdoor exposure. The change in physical properties of a roof membrane may be the result of many factors. A few factors that may affect the physical properties of a vinyl membrane include chemical formulation stability, thickness of the polymer, reinforcement, method of manufacturing, geographic location, heat and ultra violet radiation exposure, other products used in conjunction with the membrane and roof slope. These factors cannot adequately be simulated in any test program. The certainty of service life predictions increases with increasing application experience.

A major international supplier of PVC membranes with a vast inventory of roofs across Europe and North America, decided to survey a large sampling of their older roofs to assess how their materials were performing over time. The survey was expected to provide valuable insight on the ageing behavior of the products and will serve as a basis for life cycle costing (LCC) and life cycle analysis (LCA) evaluations.

2 METHODOLOGY

The manufacturer reviewed their internal project data bases and files in the various countries in which they operate to determine some of the oldest projects in each of their regions. 20 roofs were selected to be surveyed and sampled in Europe and 25 in North America. The roofs were chosen on the basis their age, geographic location (reasonable cost to access and to insure diversity of climate), and owner willingness to allow the company to access their roof and remove samples. A thorough visual inspection was conducted on each roof and samples were taken. In the USA local roofing consultants were invited to participate in every investigation. The North American roofs were surveyed in 2001 and the European roofs in 2002. Only roofs with exposed membranes were included in the survey. The manufacturer promotes the use of membranes with a glass mat carrier (G type) in adhered applications, and those with a synthetic polyester reinforcement (S type) in mechanically attached assemblies. Information on all inspected projects in Table 1. Unless otherwise specified, the installed thickness of all membranes was 1.2 mm.

All samples were sent to the manufacturer's research and development laboratory in Switzerland for testing. All samples were tested to the requirements of the German standard DIN16726 [2] or the Swiss standard SIA V 280 [3], the relevant standard for single ply PVC roofing membranes in each country.

A second set of samples taken from the North American roofs studied was sent to the National Research Council Canada for testing according to the requirements of ASTM D4434 [4]. Additional measurements not called for in the standard such as glass transition and reflectivity were also conducted on this set of samples. It is far beyond the limits of this paper to report the full set of data. More detailed information on the background of the study and the test methodologies can be found in previous papers by the same authors [5] [6]. A smaller sub set of all of the samples was subjected to hail resistance testing at the EMPA in Zurich, Switzerland.

Table 1: Summary of all projects. Samples 1-26: North America, samples 101-137: Europe

ID	Project Location	Type*	Instal-led	Age years	ID	Project Location	Type*	Instal-led	Age years
1A	Canton MA	G – 12	1979	22	21A	Haileybury ON	G - 12	1981	20
1D	Canton MA	S – 12	1979	22	21C	Haileybury ON	S - 12	1981	20
2A	Wenham MA	G - 12	1984	17	22A	Hamilton ON	S - 12	1984	17
2D	Wenham MA	S - 12	1984	17	23A	Alouette QC	G - 12	1983	18
3A	Woburn MA	G - 12	1983	18	25A	Sarnia ON	G - 12	1984	17
4B	Dickson TX	G - 12	1984	17	26	Calgary AB	G - 12	1982	19
5B	Tyler TX	G - 12	1981	20	101	Bregenz, A	S - 12	1978	24
5C	Tyler TX	S - 12	1981	20	102	Villach, A	S - 12	1981	21
6A	Euless TX	S - 12	1984	17	103	Hausmannstätten, A	S - 18	1984	18
7A	City of Industry CA	G - 12	1979	22	104	Vlotho, D	S - 12	1975	27
8A	El Segundo CA	G - 12	1982	19	105	Freiburg, D	S - 12	1977	25
9B	Mountainview CA	S - 12	1983	18	106	Memmingen, D	S - 12	1978	24
10B	Lacey WA	G - 12	1982	19	107	Niedergösgen, CH	S - 12	1978	24
11B	Ft. Steilacoom WA	G - 12	1983	18	108	Schwyz, CH	S - 12	1978	24
12A	Atlanta GA	S - 12	1986	15	109	Geneva, CH	S - 12	1978	24
13A	Jacksonville FL	S - 12	1982	19	110	Bursins, CH	S - 18	1993	9
14A	Appleton WI	S - 12	1985	16	111	Spreitenbach, CH	S - 18	1985	17
15B	Mt. Prospect IL	G - 12	1981	20	112	Canobbio, CH	S - 18	1985	17
15D	Mt. Prospect IL	S - 12	1981	20	131	Arnoldstein, A	G - 14	1986	16
16A	Park Ridge IL	S - 12	1984	17	132	Dortmund, D	G - 14	1979	23
17B	Hackensack NJ	S - 12	1986	15	133	Kempten, D	G - 12	1976	26
18A	Englewood NJ	G - 12	1985	16	134	Camorino, CH	G - 27	1976	26
18C	Englewood NJ	S - 12	1985	16	135	Personico, CH	G - 12	1968	34
19A	Iowa City IA	S - 12	1982	19	136	Lugano, CH	G - 12	1970	32
20B	Davis CA	G - 12	1981	20	137	Reading, UK	G - 12	1987	15

Note: *: Type of membrane, G: glass reinforced, S: polyester reinforced, "- xy": thickness in mm

3 ROOF CONDITION SURVEY

On one of the European objects the owner replaced the roof with the same material due to an external damage, and therefore the roof was nine years old at the time of the investigation, rather than 22 as expected. All of the roofs were in good condition. The roofs exhibited various degrees of soiling, the level of which depended on their location, surroundings, building occupancy/ activity, slope, etc. On some of the adhered roofs, there was evidence of insulation board shrinkage below the membrane. In some instances this resulted in localized areas of un-adhered membrane. There were patches on a few of the roofs indicating that the membrane had been punctured at some point. Typically when there were patches, they were found at access points and adjacent to mechanical equipment. Although various skill levels were observed, all welds, including field seams, patches and flashings were watertight. Samples were removed from all roofs. Without exception, new material was welded to the existing, aged membrane. Large weeds were growing in an area were soil had accumulated on one roof. The area was cleared for inspection. The roots had not had any effect on the membrane. On another roof, the skylights had been damaged by hail, although there was no damage to the membrane.

4 TEST STANDARDS

DIN and the SIA standards for roofing membranes were established in 1976 and 1977 respectively. The ASTM standard was first introduced in 1985. All were the first single ply standards introduced in their respective countries. It is interesting to note that many of the roofs surveyed were installed before these standards came into existence.

5 TENSILE PROPERTIES

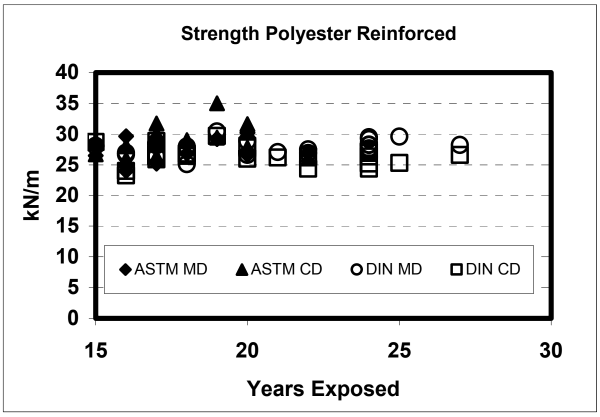
Test data for all the polyester reinforced samples, for both machine and cross directions, is shown in Figure 1. The North American samples were tested according to both the ASTM and DIN test procedures, while the European samples were only subjected to the latter.

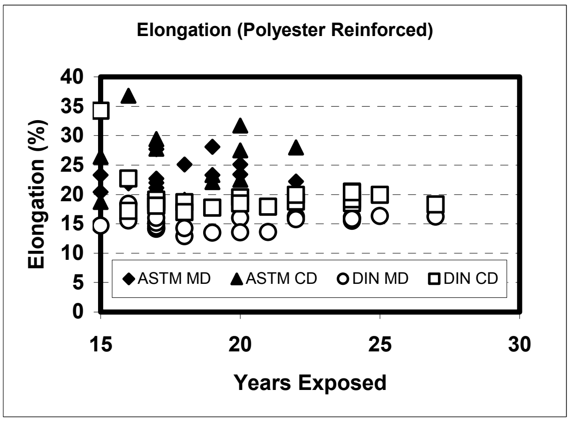
None of the North American samples met the minimum breaking strength requirement (35 kN/m) as stated in ASTM D4434 except Sample 13A in the cross direction. The samples retained 70-90% of the minimum breaking strength required for new membranes as specified in ASTM D4434 and over 60% of the samples retained more than 80% of that requirement. Note at the time the membrane was made for most of these projects the ASTM Standard did not exist.

All of the samples, European and North American exceeded the minimum requirements of the DIN standard for new materials (16 kN/m), by 60% to 75%.

The German requirement (16 kN/m) is less than half of the American minimum (35 kN/m). It is interesting to note however that despite the different test methodologies, the tensile results for a given sample correlate remarkably well between the two standards. Additionally, as can be seen in Figure 1, there is little variation in tensile strength as the membranes age beyond 15 years. It would appear that the polyester reinforcement is well encapsulated within the PVC matrix and is therefore very effectively protected. As mechanically attached membranes are subjected to countless cycles of wind uplift over their service lives, the maintenance of high tensile strength is a critical factor in the long term performance of these membranes.

Figure 1: Tensile strength (left) and elongation at break (right) of polyester reinforced membranes versus age





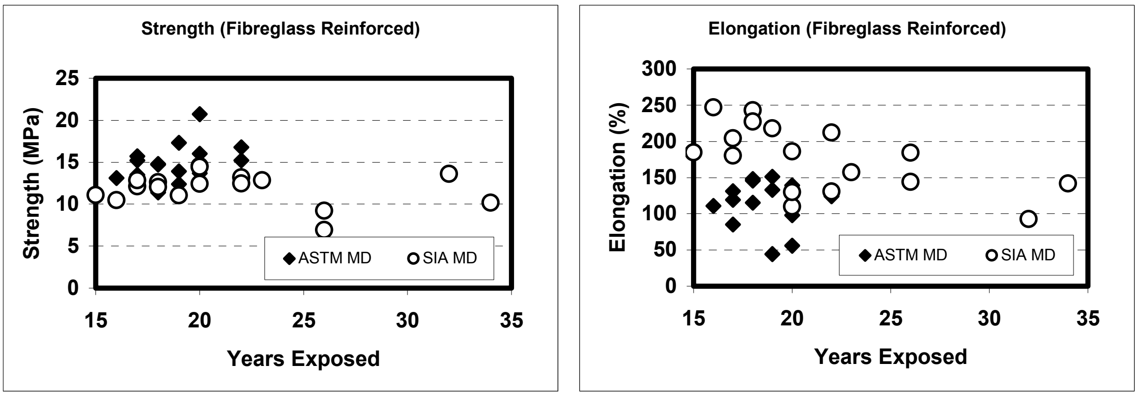
All the North American samples exceeded the minimum elongation at break value (15%) specified within ASTM D4434 for new material. All samples exceeded the minimum requirements of the DIN standard for new membranes (10%). As can be seen in Figure 1 however, unlike the tensile data, the elongation values generated by the two test methodologies do not correlate very well. The ASTM method appears to yield consistently higher results than the DIN test. The ASTM procedure not only results in higher values but also significantly greater data scatter. The DIN data conversely is quite consistent.

As would be expected, the membranes supported by the light weight, glass mat behave differently under tensile load than the much stronger polyester reinforced sheets. The glass mat in these membranes is there simply to insure dimensional stability. These membranes have the lowest level of shrinkage of any single ply membrane on the market. Test data for all glass mat supported samples is shown in Figure 2.

Whereas with polyester reinforced membranes, the strength of the sheet depends almost exclusively on the scrim, in glass mat supported membranes the strength comes from the polymer. To account for the thickness of the sample (i.e. greater strength with increasing membrane thickness), data is reported in MPa. All North American samples exceeded the ASTM minimum requirement for new material (10.4 MPa). The tensile strength of all the samples was greater than the DIN minimum (8 MPa). As can be seen in Figure 2, there is a tendency to increased tensile strength with age in the 15 to roughly 23 year range. This is expected as the sheet loses some flexibility over time. Beyond that range, there are insufficient data points to observe a clear trend.

A minimum elongation at break value of 250% is required for new materials in ASTM D4434. The measured elongation at break for the North American samples ranged from 45-150%, which corresponded to 18-60% of the minimum value specified for new materials. Samples 4B, 5B, 8A, and 20B had significantly lower elongation at break values (18-40% of ASTM minimum) than the rest (44-60% of ASTM minimum). The reasons for these values are not clear at this time. The DIN standard calls for new membranes to achieve a minimum of 150% elongation at break. As can be seen in Table 6.3, 4 of the 7 European samples achieved this value, one sample was at 95% of this value and another was at 92% of it (in the machine direction). Overall 11 of 17 samples (European and North American) surpassed this requirement for new products. Even amongst the samples with the lowest elongation values, all of the roofs were performing at the time of the survey and none showed any signs of any distress.

Figure 2: Tensile strength and elongation at break of glass mat supported membranes versus age

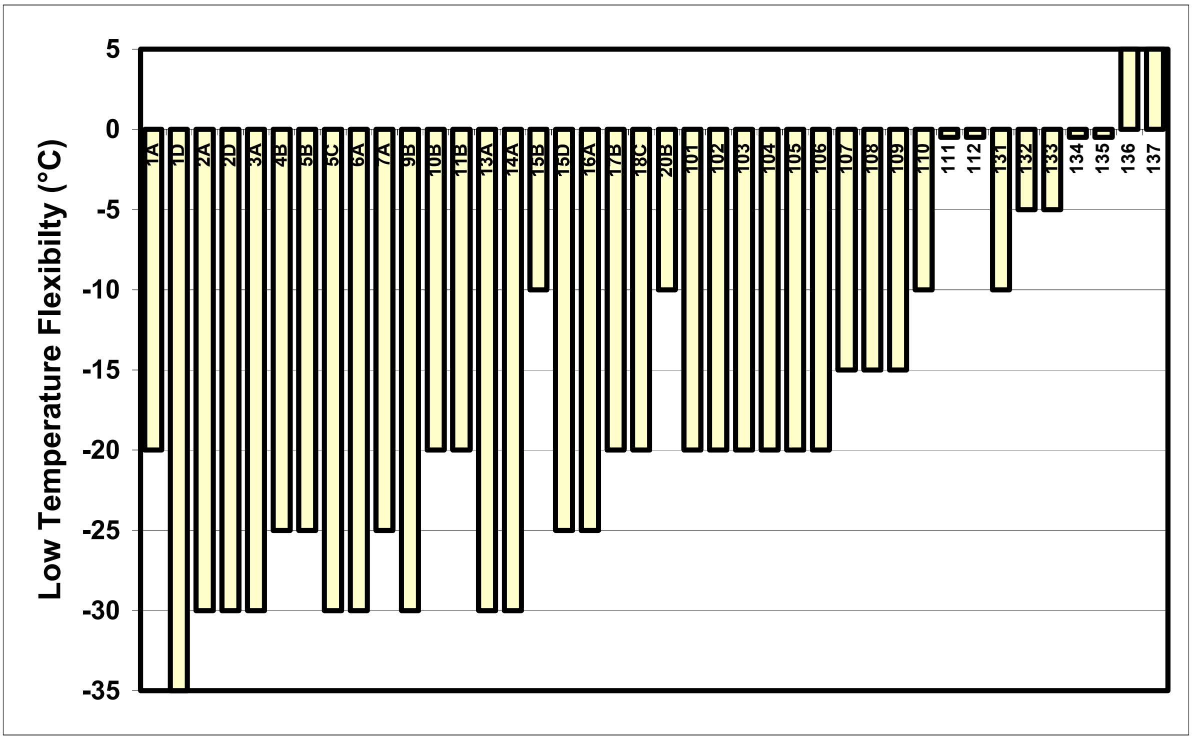


As can be seen in Figure 2, there is no correlation between the elongation data generated by the two different test methods. With the glass mat sheets, the SIA procedure results in higher values than the ASTM method, in many cases significantly higher. Once more, the different test parameters are assumed to be the reason for the differences. Perhaps not unexpectedly, for both types of membranes, the testing conducted at the lower cross head speed yields the higher elongations at break.

6 Low Temperature Flexibility

Flexibility is an important membrane property, particularly during the application phase. The flexibility of all types of roofing membranes decreases with temperature. For this study, the membranes' low temperature flexibility (LTF) was tested according to the procedure outlined in SIA 280. Five 10 mm wide rectangular specimens are folded with a bending radius of about 15 mm and fixed between two metal plates. The test device is then stored in a chamber and allowed to cool to the desired test temperature. When the samples have reached the required temperature the device is removed from the freezer and the two metal plates are instantly and quickly pressed together so that the samples are bent to a radius of 5 mm. The lowest temperature at which all five specimens do not break or crack is recorded. The reproducibility of the test method is ± 5°C. The SIA 280 requirement for new material is -20 °C. Test results are summarised in Figure 3.

Figure 3: Low temperature flexibility data



Remarkably 25 out of 40 samples still fulfill the requirement for new materials according to the SIA requirement of -20 °C or lower. Even the two samples with the highest values of 5 °C still show considerable flexibility. The testing conditions (rapid 180° bending around a small radius) are obviously severe and do not occur in real roof conditions. Membrane flexibility is an issue mainly during installation and roof maintenance. As can be seen even the aged installed membranes with a LTF value of 5 ° C continue to perform.
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The fact that a majority of all samples are tested with low temperature values above the requirements for virgin material reflects the manufacturer's efforts to formulate their membranes for long term behavior. Potential reduction in plasticizer content over long years of roof service is accounted for by the appropriate formulation of the base vinyl material.

7 Hail Resistance

Twenty seven of the samples received at the manufacturer's laboratory were large enough after all other analytical procedures (minimum 0.5 m x 0.5 m) to be used for hail testing. The age of these 27 roofs ranged from 15 to 34 years. For the purposes of this investigation the hail test method developed by the Swiss Federal Laboratories for Materials Testing and Research (EMPA) was chosen for the determination of the hail resistance. A detailed description of the test procedure and discussion of the results would be beyond the scope of this paper. They can be found in [7].

The Swiss standards SIA280 (polymeric) and SIA281(bituminous) require a minimum impact velocity of 17 m/s for new roofing membranes. In order to determine how aged material would perform on substrates in use today, the aged membrane was tested over the most commonly used thermal insulations: polyisocyanurate (ISO) for North America and expanded polystyrene (EPS, density 20 kg/m^3) for Europe. Testing was also done on glass fiber reinforced gypsum boards. For comparison purposes new membranes of the same PVC formulation and different thicknesses were also tested. Test results are summarized in Table 2.

Table 2: Hail resistance results. Blank fields indicate that no values have been determined.

ID	type thickness	age	hail resistance (impact velocity)		
			Gypsum	ISO	EPS
	mm	years	m/s	m/s	m/s
	G 1.2	new	66	39	47
	G 1.8	new	96	67	85
	S 1.2	new	79	54	61
	S 1.8	new	95	68	77
	G 1.2 [1)]	new	90		
	G 1.2 [2)]	new	91		
01 A	G 1.2	22	39	39	
01 C	S 1.2	22	37	38	
02 B	G 1.2	17	39	14	
02 C	S 1.2	17	52	45	
03 B	G 1.2	18	40	27	
04 A	G 1.2	17	12	5	
05 A	G 1.2	20	30	33	
05 D	S 1.2	20	19	30	
06 B	S 1.2	17	32	37	
07 B	G 1.2	22	17	7	
09 A	S 1.2	18	46	41	
10 A	G 1.2	19	29	16	
11 B	G 1.2	18	43	20	
13 A	S 1.2	19	14	10	
14 B	S 1.2	16	58	54	
15 A	G 1.2	20	37	30	
15 C	S 1.2	20	34	28	
16 B	S 1.2	17	51	51	
17 A	S 1.2	15	52	55	
18 D	S 1.2	16	59	54	
20 A	G 1.2	20	18	11	
101	S 1.2	24			34
104	S 1.2	27			13
111	S 1.8	17			35
112	S 1.8	17			46
135	G 1.2	34			30
137	G 1.2	15			7

1) membrane fully adhered to gypsum board;
2) felt backed membrane, fully adhered to gypsum board

All measured data of new membranes values exceed the minimum requirements by a multiple. Not surprisingly, 1.8 mm thick membrane provides greater resistance than 1.2 mm membrane. Results over glass faced gypsum board are roughly 1.5 times higher than those measured over polyisocyanurate boards, for a given set of parameters.

Of the European surevy samples 101 and 135, 25 and 34 years old respectively, have hail resistance values below the requirement for new material. However, despite their age, and their locations in regions with high hail risk, these roofs exhibited no signs of hail damage. The other four samples, aged from 15 to 27 years, have hail resistance values far above the SIA280 requirement for new membranes.

Comparing the North American projects, the glass faced gypsum board generally is found to improve hail resistance. With an average age of 18.6 years, 16 out of the 21 samples still fulfill the requirement FM Class 1-MH for new membranes, while 12 samples meet the requirement FM Class 1-SH on glass faced gypsum board (see [7] for a calculatory comparison between SIA and FM hail test values). On ISO, 14 of the samples, aged 17 to 22 years, meet FM Class 1-MH and 11 samples meet FM Class 1-SH. On glass faced gypsum board only one sample (13A) had a hail resistance value below the initial requirement of SIA280. All the others meet the requirement for new material. None of the roofs exhibited any signs of hail damage during the inspection.

In a separate paper [7], one of the authors of this work studied the correlation between hail resistance (impact speed) and other physical properties. Both plasticizer content and low temperature flexibility were found to correlate reasonably well with hail resistance, with correlation coefficients around 0.6 in both cases. This area should be studied in greater depth. No correlation whatsoever was found between hail resistance and impact resistance, confirming that the latter cannot be used as a substitute for assessing the former.

8 Conclusions

Fourty four roofs, located in 6 countries in Europe and North America, were analyzed, and samples from each were subjected to a variety of physical property tests. Overall, the field performance of these fibreglass and polyester reinforced vinyl membranes, were found to be without problem. The roofing systems averaging over 20 years of age were performing well and without leakage. All membranes were capable of being welded to even after up to 34 years of weathering.

The laboratory testing confirms that although the products tested lost some of their initial physical properties, which is to be expected with any materials as they age, they generally held up very well compared to the standard minimum values for testing new PVC roofing membranes according to North American and European standards. It is important to note, however, that some of these membranes, which had been tested in the NRC laboratory about 15 years ago, exceeded the minimum requirements of the ASTM D4434. This is an interesting point because as all roofing materials age and weather, their properties are expected to degrade. Therefore, to ensure that the minimum property values are exceeded after aging/weathering, a new membrane, regardless of the type (i.e., polymeric, elastomeric or asphaltic) must exceed the minimum requirements listed in the standards.

As the roofs examined are essentially the oldest in place, it is not possible to predict how much longer they will perform. But considering the age and the condition of the roofs analyzed, this data would indicate that a properly formulated, properly maintained, reinforced PVC roof membrane system could perform in excess of 20 to 30 years in various climates throughout Europe and North America.

9 References

1. Cash, C.: Comparative Testing and Rating of Thirteen Thermoplastic Single Ply Roofing Materials, Interface, Journal of the Roof Consultants Institute, October, 1999
2. Deutsches Institut fur Normung DIN16726, 1983-05, Beuth Verlag, Berlin, Germany
3. Schweizerischer Ingenieur- und Architektenverein SIA V280, 1986-12, Switzerland
4. ASTM D4434 –96 Standard Specification for Polyvinyl Chloride Sheet Roofing
5. Whelan, B., Graveline, S., Delgado, A., Paroli, R., "Field Investigation and Laboratory Testing of Exposed Poly(Vinyl Chloride) Roof Systems", CIB World Building Congress, "Building for the Future", Toronto, Canada, May 1 – 7, 2004
6. Beer, H.R., „Durability of PVC Roof Membranes – Field Investigation and Laboratory Testing After Up to 34 Years Exposure", ICBEST Symposium, Sydney, Australia, 2004
7. Beer, H.R., Schumann, K., Flüeler, P., „Hail Resistance Of Aged PVC Roofing Membranes - A Field Evaluation Of Roofs Ranging Between 15 And 34 Years", CIB World Building Congress, "Building for the Future", Toronto, Canada, May 1 – 7, 2004

Environmental Friendly Wood Linings for Outdoor Exposure

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ABSTRACT

One of the most important parameters for the durability of wood linings for outdoor exposure is the amount of surface checks that are initiated already in the sawing procedure. The main object for this project is to test the durability under outdoor exposure for wood from two different sawing patterns. At conventional sawing pattern tangential surfaces are exposed, while the star-sawing method gives radial surfaces. The development of cracks and changes in appearance has been investigated on radial and tangential surfaces of pine (Pinus sylvestris L.) and spruce (Picea abies Karst.) which have been exposed for outdoor climates for 61 months. The annual ring orientation is the most important factor for crack development on weathering. The type of wood and impregnation treatment has only a marginal effect on the crack development. After 61 months of outdoor exposure, tangential surfaces of pine have 1.7 – 2.2 times more total crack length per unit area than the corresponding radial surfaces. In spruce, the total crack length on the tangential surfaces is 2.2 – 2.6 times greater than on the radial surfaces. Tangential surfaces of both pine and spruce have a greater number of cracks per unit area and wider cracks than the corresponding radial surfaces. Tangential and radial surfaces show the same colour change in the surface as a result of weathering.

KEYWORDS

Weathering
Pinus sylvestris L.
Picea abies Karst.
Cracks
Star sawing

1. INTRODUCTION

The sensitivity of the wood to degradation is one of its greatest weaknesses in outdoor usage. Like all biological materials, wood decomposes under the influence of the surrounding environment. When wood is exposed outdoors above ground, a complex decomposition process continues in the material as a consequence of chemical, biological, mechanical and light energy related factors. A common name for this process is "weathering" (Feist 1982).
The factors which are generally considered to cause changes in wood surfaces on weathering are sunlight (UV, visible and infrared radiation), moisture (dew, rain, and snow), temperature and oxygen (Hon 1983).
Because of the limited ability of light to penetrate into wood (Browne and Simonson 1957), the effect of the weathering is limited to a 2.5 mm thick surface layer and the erosion is slow, 5 – 12 mm per 100 years (Feist and Mraz 1978).
Investigations of the effect of weathering on wood, carried out by different researchers, have dealt with several aspects, e.g. colour change (Fengel and Wegener 1984, Sandermann and Schlumbom 1962, Sell and Leukens 1971), erosion (Arnold et al 1992, Feist and Mraz 1978, Feist and Hon 1984), free radicals (Hon et al 1980; Hon and Feist 1981; Hon and Shiraishi 1991), surface wetting characteristics (Kalnins and Knaebe 1992; Kalnins and Feist 1993), anatomical changes (Miniutti 1967, Borgin 1970, 1971, Borgin et al 1975, Derbyshire and Miller 1981, Sandberg 1999), and strength (Derbyshire et al 1995, Raczkowski 1980). Of the whole electromagnetic spectrum, it is only the short wave length i.e. energy-rich region, which has a measurable influence on wood and which, is thus of technical interest. As a consequence, a large number of studies have been carried out within this field and summaries of earlier result have been published by e.g. Kenaga and Cowling (1959), Desai (1968), Kringstad (1969), Hon and Glasser (1979) and Hon and Shiraishi (1991).
On a macroscopic level, the colour change of an untreated wood surface is one of the firsts and perhaps the clearest sign of the degradation of the wood during outdoor exposure. Visible light and UV-radiation alter the colour of the wood to a darker or lighter shade, depending on the type of wood (Sandermann and Schlumbom 1962; Fengel and Wegener 1984). After a long period of outdoor use, all types of wood develop a greyish appearance (FRN 1966; Sell and Leukens 1971) due to the fact that water-soluble decomposition products are removed and the more or less delignified fibres are exposed. If, on the other hand, the wood surface is protected against rain, it develops a dark red-brown surface (Browne 1959).
The photochemical degradation is a very slow process, which during a decade degrades only a few millimetres of the wood surface and leaves the underlying wood practically unaffected (Hon and Ifju 1978). The combined effect of water and sunlight degrades the main components of the wood and transforms the wood surface into a network of weakly connected cellulose fibrils which are strongly contaminated by spores from micro-organisms (Sell and Wälchli 1969).
Visible cracks arise in the wood surface during outdoor exposure because of the growth of micro-cracks formed during the drying of the wood, photochemical reactions or moisture-induced stress fields (Coupe and Watson 1967). Stamm (1965a) considers that wood for outdoor use should have vertical annual rings giving radial surfaces. This minimizes the risk of cracks as a consequence of anisotropy in moisture movements. Cracks in the radial surface are also smaller than in the corresponding tangential surfaces (Browne 1960; Stamm 1965a, 1965b).
The aim of the present investigation has been to characterize differences in the degradation process on macro-level between radial and tangential wood surfaces of Scots pine and Norway spruce exposed outdoors above ground.

2. MATERIAL AND METHOD

In this test fully quarter-sawn and plain-sawn wood of Scots pine (*Pinus sylvestris* L.) (denoted for short pine) and Norway spruce (*Picea abies* Karst.) (denoted for short spruce) have been used. The timber for the investigation was of forest-scale quality and was taken from Sweden.
The sawing was carried out according to a sawing pattern, which was a combination of star sawing and through-and-through, as shown in figure 1. For the test only boards with a rectangular cross section was used. All the pieces were edged immediately after the sawing. The edging was carried out as parallel as possible to the pith direction of the wood, i.e. no so-called taper edging was carried out. All the test material was then dried simultaneously in a drying chamber with the same drying schedule. The samples were planed on all surfaces to facilitate the determination of crack length and the planning was made with the top end in the planing direction to a depth of 2.5 mm. The final cross section dimension was 95 x 22 mm. After planning, the wood has been pressure impregnated with a CCA-agent.
From each board 4 knot-free and defect-free test pieces with a length of 484 mm have been prepared. The end-wood surfaces were sealed with an oil alkyd primer and a silicone-based sealing compound.
The dry density was determined for all samples: pine 565±44 kg/m^3 and spruce 475±42 kg/m^3.
The samples were exposed in Stockholm for 61 months (July 1997 – July 2002) at an inclination of 45 degrees towards the south. Three different surfaces were exposed; radial, tangential surface inside face exposed and tangential surface outside face exposed, figure 1. Table 1 presents a summary of the test material.
After outdoor exposure, all the samples were conditioned for two months at a temperature of 20°C and a relative humidity of 65 %. Thereafter, the lengths of all cracks, i.e. both on the exposed flat side and on the back side, were determined with crack widths greater than 0.25 mm, which was the smallest crack width that could be measured in practice. The method for crack measurement is described in Malmquist (1984) and Sandberg (1999). For the analysis below the portion of specimens with observable cracks in each group was also determined.

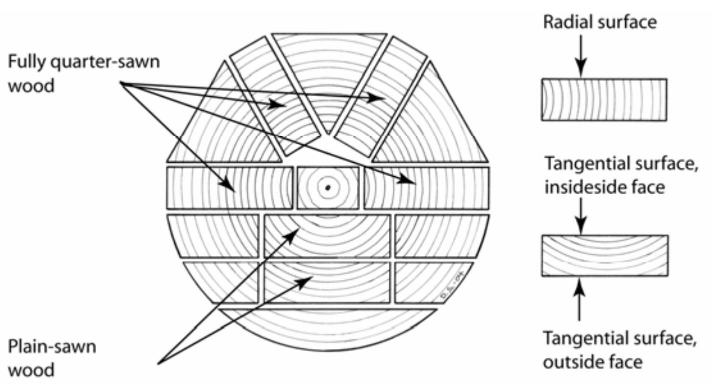

Figure 1. Sawing pattern used for preparation of the specimens.

No.	Species	Surface exposed to the south	Number of samples	Mean density, (kg/m^3)	Standard deviation, (kg/m^3)
1	Spruce	Tangential surface inside face exposed	30	484	43
2	Spruce	Tangential surface outside face exposed	31	483	48
3	Spruce	Radial surface	24	455	23
		Total spruce:	85	475	42
4	Pine	Tangential surface inside face exposed	17	559	33
5	Pine	Tangential surface outside face exposed	18	558	31
6	Pine	Radial surface	38	571	52
		Total pine:	73	565	44

Table 1. Densities of the 158 specimens of spruce and pine.

3. ANALYSIS

The following analysis of cracks follows an earlier published paper (Söderström 1990). In a given group there are n specimens and k of them have observable cracks lengths x_i, where i is in the interval from 1 to k. The number k is assumed to be binomial distributed with

$$P(x > 0) = p \tag{1}$$

and of course then

$$P(x = 0) = 1 - p \tag{2}$$

The expectation value (mean value) of x is then

$$E(x) = 0 \cdot (1-p) + p \cdot E(x|x > 0) = p \cdot E(x|x > 0) \tag{3}$$

The formation of cracks is a relaxation of energy according to the mechanisms of fracture mechanics. The probability find a crack in the crack length interval Δx is proportional to the interval length $\lambda\Delta x$, where λ denotes the intensity. The probability of finding no cracks in this interval is then of course $1 - \lambda\Delta x$. The probability of finding no cracks with lengths in the interval from 0 to x is denoted P(x). The probability of finding no cracks in the interval $x + \Delta x$ is assumed to be independent on P(x) and $1 - \lambda\Delta x$ i.e.

$$P(x + \Delta x) = P(x) \cdot (1 - \lambda\Delta x) \tag{4}$$

or in differential form

$$\frac{dP}{dx} = -\lambda x \tag{5}$$

which integrated becomes

$$P(x) = e^{-\lambda x} \tag{6}$$

Therefore, the probability to find a crack length > 0 but less than x is $1\text{-}e^{-\lambda x}$. The frequency function is consequently $\lambda e^{-\lambda x}$ i.e. the cracks length is distributed according to an exponential distribution. The mean value E(x|x > 0) is then

$$E(x|x > 0) = \int_0^\infty \cdot e^{-\lambda x} dx = \frac{1}{\lambda} \qquad (7)$$

which together with equation (3) gives

$$E(x) = \frac{p}{\lambda} \qquad (8)$$

The total crack length X is then

$$X = \sum_i^k x_i \qquad (9)$$

which is a gamma-distribution with the mean value E(X) and variance V(X) according to

$$E(X) = \frac{np}{\lambda} \qquad (10)$$

$$V(X) = \frac{np \cdot (2-p)}{\lambda^2} \qquad (11)$$

According to the central limit theorem the mean crack length for a group is approximately a normal distribution as

$$N(\frac{p}{\lambda}, \frac{p}{\lambda} \cdot \sqrt{\frac{2-p}{p \cdot n}}) \qquad (12)$$

By using equation (12) for the various groups above and a normal distribution it is possible to assess the significance of the difference between the mean values of the crack length from the observations of the different groups.

4. RESULTS AND CONCLUSIONS

Table 2 presents the result from measurements of the crack lengths and analyzed with the method above, which gives a better estimation of the standard deviation than the ordinary method by assuming a normal distribution of the crack lengths.

No. According to table 1	**Mean crack length at exposed side, (mm)**	**Standard deviation, (mm)**	**Mean crack length at backside, (mm)**	**Standard deviation, (mm)**
1	5149	1248	445	108
2	6114	1441	338	79
3	2332	378	55	23
4	2769	505	602	135
5	3530	634	296	80
6	1586	352	50	34

Table 2. Mean value of the crack length and the standard deviation on the exposed side and backside after five years of outdoor exposure.

From table 2 it is clear that there is a strong significant lower crack length at the surface of specimens with radial surfaces exposed to the outdoor climate. The same conclusion is valid for corresponding backsides. The explanation may be that the radial surfaces have varied densities over the surface depending on the early wood, with a low density, and the late wood, with a high density. The propagation of cracks is the result of relaxation of stored elastic energy and it is hindered by the local density gradients. The effect is at least clearly demonstrated in this paper for impregnated wood out-door exposed without contact with soil. The amount of cracks is supposed to be the main factor for the estimation of service life as the crack promotes capillary suction of water that gives the condition for growing of mould and rot. Therefore, the estimated service life for e.g. wooden panels is higher for radial surfaces and this fact may be taken into account by a proper choice of the design factor B in the factor method according to ISO 15686-1.

5. REFERENCES

Arnold, M., R.L. Lemaster and W.A. Dost. 1992. *Surface characterization of weathered wood using a laser scanning system.* Wood and Fiber Science *24*(3), 287-293.
Borgin, K. 1970. *The use of the scanning electron microscope for the study of weathered wood.* J. of Microscopy *92*(1), 47-55.
Borgin, K. 1971. The *mechanism of the breakdown of the structure of wood due to environmental factors.* J. Inst. Wood Sci. *5*(4), 26-30.
Borgin, K., N. Parameswaran and W. Liese. 1975. *The effect of aging on the ultrastructure of wood.* Wood Science and Technology *9*, 87-98.
Browne, F.L. 1959. *Understanding of the mechanism of deteriation of house paint*. Forest Product J. *11*, 417-427.
Browne, F.L. 1960. *Wood siding left to weather naturally.* Southern Lumberman, December *15*, 141-143.
Browne, F.L. and H. Simonson. 1957. *The penetration of light into wood*. Forest Products J. *7*(10), 303-314.
Coupe, C. and R.W. Watson. 1967. *Fundamental aspects of weathering*. Proc. Ann. Conv. Br. Wood Preservation Ass. pp. 37-49.
Derbyshire, H. and E.R. Miller. 1981. *The photodegradation of wood during solar irradiation. Part I: Effects on the structural integrity of thin wood strips.* Holz Roh- Werkstoff *39*, 341-350.
Derbyshire, H., E.R. Miller and H. Turkulin. 1995. *Investigations into the photodegradation of wood using microtensile testing. Part 1: The application of microtensile testing to measurement of photodegradation rates.* Holz Roh- Werkstoff *53*, 339-345.
Desai, R. L. 1968. *Photodegradation of cellulosic materials - A review of literature.* Pulp Pap. Mag. Can. *69*, 53-61.
Feist, W.C. 1982. *Structural use of wood in adverse environments.* Van Nostrand Reinhold Co, New York, pp. 156-178.
Feist, W.C. and E.A. Mraz. 1978. *Comparison of outdoor and accelerated weathering of unprotected softwoods.* Forest Products J. *28*(3), 38-43.
Feist, W.C. and D.N.-S. Hon. 1984. *Chemistry of weathering and protection.* Adv. Chem. Ser. *207*, 401-457.
Fengel, D. and G. Wegener. 1984. *Wood - chemistry, ultrastructure, reactions.* Walter de Gruyter, Berlin. New York.
FRN 1966. *Wood finishing: Weathering of wood.* US. For. Res. Note FPL-0135.
Hon, D.N.-S. 1983. *Weathering reactions and protection of wood surfaces*. J. of Applied Polymer Science: Applied Polymer Symposium *37*, 845-864.
Hon, D.N. -S. and G. Ifju. 1978. *Measuring penetration of light into wood by detection of photo-induced free radicals*. Wood Sci. *11*, 118-127.
Hon, D.N.-S. and W. Glasser. 1979. *On possible chromophoric structures in wood and pulps - A survey of the present state of knowledge.* Polym. Plast. Technol. Eng. *12*, 159-179.
Hon, D.N.-S., G. Ifju and W.C. Feist. 1980. *Characterization of free radicals in wood*. Wood and Fiber *12*, 121-130.
Hon, D.N.-S. and W.C. Feist. 1981. *Free radicals formation in wood: The role of water*. Wood Science *14*, 41-48.
Hon, D.N.-S. and N. Shiraishi. 1991. *Wood and Cellulosic Chemistry.* Marcel Dekker Inc. New York.
ISO (2000) ISO 15686-1:2000 *Buildings and constructed assets - Service life planning. Part 1: General principles.*
Kalnins, M.A. and M.T. Knaebe. 1992. *Wettability of weathered wood.* J. of Adhesion Science and Technology *6*(12), 1325-1330.
Kalnins, M.A. and W.C. Feist. 1993. *Increase in wettability on wood with weathering.* Forest Products J. *43*(2), 55-57.
Kenaga, D.L. and E.B. Cowling. 1959. *Effect of gamma radiation on Ponderosa pine: Hygroscopy, swelling, and decay susceptibility.* Forest Products J. *9*, 112-116.

Kringstad, K. 1969. *Degradation of wood and high-yield pulp by light - A survey of the present state of knowledge.* Tappi *52*, 1070-1074.
Malmquist, L. 1984. *Compound drying of timber. An analysis of drying defects.* Wood Technology Report No. *62*, Swedish Institute for Wood Technology Research
Miniutti, V.P. 1967. *Microscopic observations of ultraviolet irradiated and weathered softwood surfaces and clear coatings.* US Forest Service Research Paper FPL 74.
Raczkowski, J. 1980. *Seasonal effects on the atmospheric corrosion of spruce micro-sections.* Holz Roh- Werkstoff *38*, 231-234.
Sandberg, D. 1999. *Weatering of radial and tangential wood surfaces of pine and spruce.* Holzforschung Vol. 53/1999/No.4
Sandermann, W. and F. Schlumbom. 1962. *On the effect of filtered ultraviolet light on wood. Part II: kind and magnitude of colour difference on wood surfacees.* Holz Roh- Werkstoff *20,* 285-291.
Sell, J. and O. Wälchli. 1969. *Changes in the surface texture of weather exposed wood*. Material and Organismen *4*, 81-87.
Sell, J. and U. Leukens. 1971. *Investigations on weathered wood surface - Part II: weathering phenomena on unprotected wood species.* Holz Roh- Werkstoff *29*, 23-31.
Söderström, O. 1990. *Torkningssprickor och slutfuktkvoter.* (In Swedish). Trätekrapport 9011062.
Stamm, A. J. 1965a. *Wood and cellulose - water relationships and their effect upon finishes.* Official Digest, June pp. 654-669.
Stamm, A. J. 1965b: *Modification of wood for improved finishing.* Official Digest, June pp. 707-717.

Artificial and natural weathering tests of externally bonded FRP reinforcement to define their service life in an outdoor environment

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ABSTRACT

The paper presents the first results of an experimental research, still running, finalized to study the durability problem of externally bonded FRP reinfocements (FRP EBR) of masonry structures. The aim is to study their behaviour in service, when they are utilized outside the buildings (wrapping, planting of masonry walls, vaults and arches) and exposed to climatic agents, particularly to temperature cycling, moisture and UV light exposure.
The innovative character of this research is most represented by the fact that, other than in the biggest part of accelerated aging experiments performed worldwide, it takes care of the performance decline of the entire masonry-interface-FRP-plaster system, to finally come to performance-time diagrams for each system component with reference to the most significant performances of each of them.
The durability valuation method used in this research is the "NIC" (UNI norm draft, working group GL 15). Consequently, at the same time it has been checked the number of real cases in which such strengthening systems have really been applied; this should be used to define the value of the midi-normal duration. The accelerated artificial aging tests are made with reference to protocols based on the ASTM (American Society for Testing and Materials) norms for fibre-reinforced plastics. They are needed for the valuation of the incidence of each researched parameter relatively to the system durability and, in consequence, for dimensioning the relative modifying factors, which have to be utilised, following the method formula, for obtaining, starting from the mid-normal durability value, the most probable service life of the system in real use conditions.

KEYWORDS

Carbon Fiber-Reinforced Polymer; Durability, Influencing Agents, Method of M. Nicolella

1 INTRODUCTION

The aim of this paper is to present the experimental phases of a still running research, finalized to study the durability problem of externally bonded FRP reinfocements (FRP EBR) of masonry structures. In Italy most of the applications of these innovative strenghtening systems are in the field of the rehabilitation of ancient structures, they are very often used to plant arches, vaults and walls, to wrap columns, beams and walls, indeed. In the last decades a lot of intervention have been made on important historical buildings of the country, in total absence of a serious experimentation on their durability. There are a lot of experimental results of aging tests carried out worldwide on the strenghtening of concrete structures, while the problem of the combination of these materials with masonry structure is less recognized. There are two other aspects to underline:

- The most difficult issue regarding durability studies is taking into account all factors (environmental, climatic, etc.) that affect the service life of a building or its components. The method, used in this research, provides a simplified procedure for considering each of the variables that are likely to affect service life;
- It takes care of the performance decline of the entire masonry-interface-FRP-plaster system, to finally come to performance-time diagrams for each system component with reference to the most significant performances of each of them.

2 THE METHOD

The adopted method “NIC” (UNI norm draft, working group GL 15), estimates the service life of a building component as a deviation from a standard value, named mid-normal, which is calculated by using modifying factors deriving from influencing agents. The “mid-normal” value is obtained through statistical elaboration of field-collected data for the assumed conditions.
The field –collection of data is still running, through the monitoring of about 20 application carried out in Italy.The result of that monitoring will be the extrapolation of performance-time diagrams for each system component with reference to the most significant performances of each of them. Once the *mid-normal* value has been determined, the service life of a FRP system in any context, will be evaluated according to the formula (1)

$$D_{pp} = D_{mn} \times \sum \frac{\lambda_i \cdot F_i}{100} \qquad (1)$$

where *Dpp* is *" the most probable "* service life, *Dmn* is the mid-*normal* duration and *Fi* the modifying factors, while λ_i is the weight of each factor. *Fi* are the modifying factors which are associated to every agent that influences the service life of the considered building component. *λi* are the weight of every influence agent. In this way the *mid-normal* value is the starting point to determine the durability of the same EBR FRP system applied to any other context, if adjusted by the modifying factors, which are thought to be a function of the specific case. The considered influencing agents for FRP strenghtening systems are those in table 1

N	*Groups of agents*	*Influencing agents*
1	Climatic agents	main temperature, UV exposure, humidity and moisture, freeze-thaw cycles
2	Environmental Agents	chemical agents, exposure to salts, sustained loading
3	Configuration	shape/lying, extension, presence of discontinuity / chines
4	Technological characteristic	application surface state protection

Table 1:Influencing agents

So that, once $\mathbf{D_{mn}}$ value will be defined, the second important step is to estimate and define, for every agent, the influence of the variation conditions on the EBR FRP strenghtening systems life with reference to the mid-normal case under the same conditions for all other agents.
For example, we can consider two different situations of influence for the same agent with reference to the same system, the mid-normal situation and another case shunting from the first only as far as the condition of the UV exposure agent is regarded. In particular, the mid-normal situation refers to the no exposure and the second to standard UV exposure. The assessed difference of behaviour, that is influenced only by the difference of the U.V. agent since the others have been considered equal, concur to define the modifying factor that numerically translate the shunting of the condition of variation in object from the mid-normal case. This approach will be applied for all conditions of variation of all the agents and the corresponding estimations can lead to the definition of all the modifying factors (Fi).
The next step is the attribution of a weight (λi) to every agent. This weight defines the influence of the agent on the degradation of the considered building element.
Once the factors have been defined and the weights for every agent chosen, basing on the methods of estimate and of the information available, the application of the method consists in the choice, by the designer using the method, of the factors that correspond to the conditions of the component and in the successive weighted arithmetical mean of these coefficients. The result of this mean is multiplied by the reference value of duration, the mid-normal value, to obtain the most probable duration of the considered EBR FRP strengthening system in the analyzed case.
This paper focuses on the effects of combined loading history and environmental exposure on the durability of Carbon Fiber-Reinforced Polymer reinforcement lamine.
Specifically, degradation of FRP lamine due to coupled freeze-thaw cycling and U.V. exposure is experimentally investigated. After a series of cyclic environmental preconditioning and mechanical loading, the degradation and variable load rate dependency of CFRP lamine is evaluated from these aspects: adhesion to the substrate degradation of due to freeze-thaw cycling and U.V. exposure, failure mode analysis including lamina failures and substrate failures.

3 LABORATORY TESTS

3.1 Used facilities

An environmental chamber, manufactured by Angelantoni (Massa Martana, Perugia, Italy) and shown in Figure 1, is used to test the influence of U.V. exposition and of sub- zero ambient temperatures on the mechanical and visco-elastic properties of the strenghtening system.

Figure 1. Climatic chamber

Characteristic	*Value*
Useful capacity	224lt
Internal dim. (mm) (LxPxH)	600x535x700
External dim. (mm) (LxPxH)	850x1460x1515
Temperature Range (°C)	-40°C/180°C
Precision over time (T)	±0.25°...±0.3°C
Heating Speed (-40/+180°C)	5°C/min
Cooling Speed (+180/-40°C)	3.5°C/min
R. H. range (t=-20/+94°C)	10%... 98%
R. H. precision	±1%...±3%
Dissipation (W) T=-25°C	400 W
Absorbed Power (kW)	3.7 kW
Weight (Kg)	490
Noise dB (A)	59

Table 2. Technical characteristics

3.2 Materials and specimen preparation

The specimens are prisms made of Naples yellow tuff, mortar, pre-mixed plaster and siloxane paint, their dimensions have been conditioned by climatic chamber dimensions. As a matter of fact the prisms have a rectangular section of about 12x12 cm and are about 50 cm long, as illustrated in Figure 2.

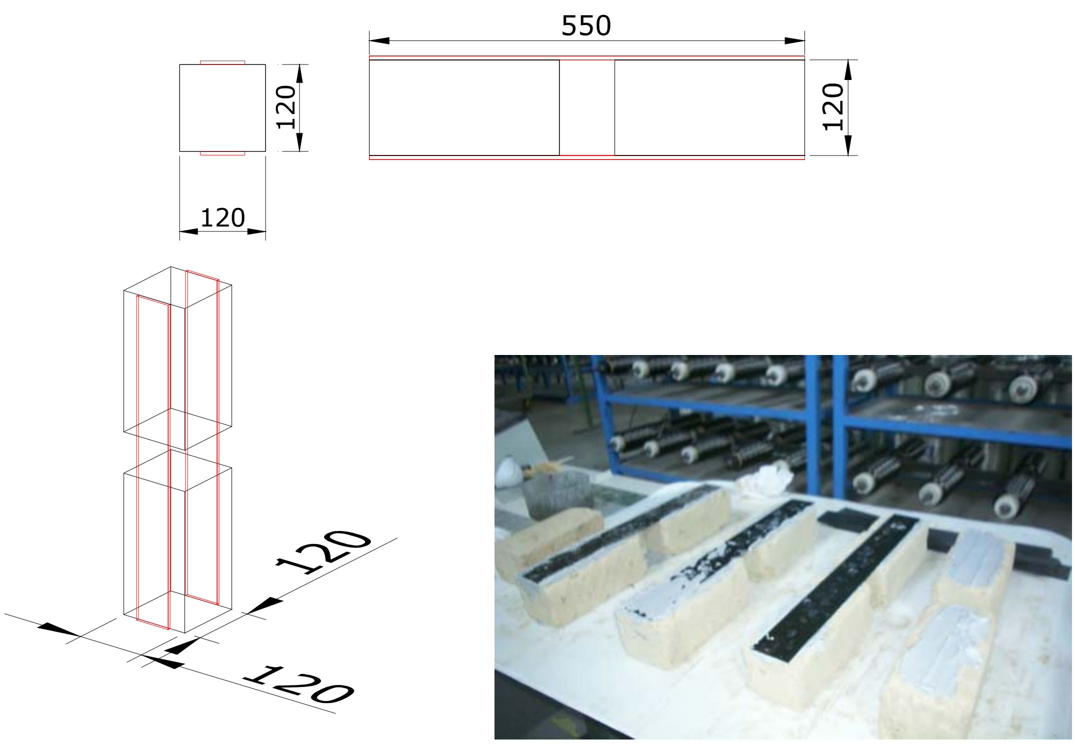


Figure 2. Specimen – planning and manufacturing

A tuff stone has been splitted along the longest side in 4 parts, obtaining 4 blocks. Two of those blocks have been utilized to compose the specimen. Each specimen is made of two tuff blocks of 12x12x24 planting with a couple of CFRP laminate with a section of 65x1,5 mm in vinylester matrix with a 35% of fibres in volume, with pre-mixed plaster and sylossane paint. A special steel device allowed to put in traction the system, to mobilize the adhesion stresses on the laminate- substrate interface. Interface is made of a bi-component epoxy putty, with the following characteristics:

Propriety	*Value*
Density	1,77 kg/l (A+B)
Tensile Modulus	12.800 N/mm 2
Bond shear stress	> 4 N/mm 2
Shear strength	> 15 N/mm 2
Coefficient of thermal expansion	9 x 10 -5 per °C (da -10°C a +40°C)

Table 3. Technical characteristics

Propriety	*Value*
Laminate thickness	1,5 mm
Width	65 mm
Laminate area	97,5 mmq
Volume fraction of fibres	35%
Nominal area of fibre sheet	34,15 mmq

Tensile strength	4.500 N/mm 2
Modulus of elasticity	234.000 N/mm 2
Ultimate elongation	1,9

Table 4. Technical characteristics

3.3 Conditioning and measurements of test specimen

Conditioning program is guided by ASTM Standards: D 618; *Standard Practice for Conditioning Plastics for Testing* [4], D 2565-99 *Xenon –Arc Exposure of Plastics Intended for Outdoor Applications* [5], and C 666; *Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing* [6]. The following measurement have been made on conditioning test specimens prior to testing, after reconditioning, and at any intermediate stage as prescribed in the test procedure.

3.3.1 Dimesions and weight

Specimen n.	*Dimensions (mm)*	*Weight (kg)*
1	115x115x560	7,2203
2	110x110x560	7,3005
3	110x115x555	7,4007
4	110x105x560	6,9980

Table 5. Technical characteristics

3.3.2 Visual examination

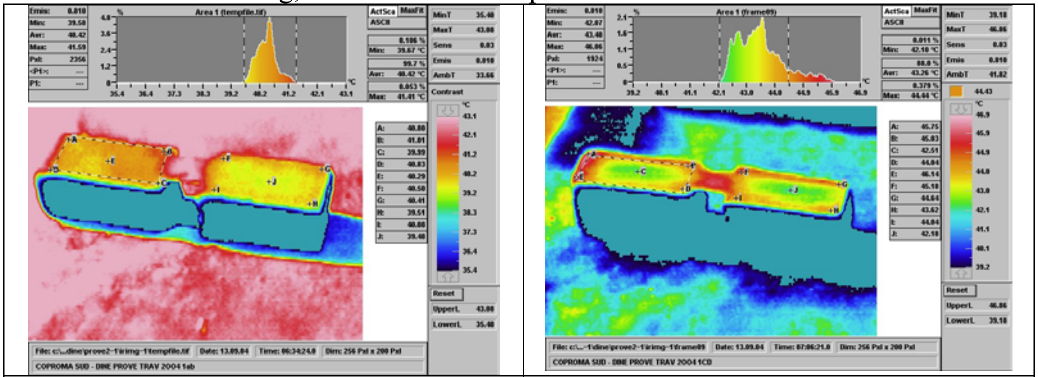
Visual inspection consists of a superficial examination to establish changes in colour, debonding, peeling, blistering, cracking, crazing, deflections, indications of eventually present cracks in the plaster, and other anomalies.

3.3.3 Thermographic tests

These tests have been finalized to evaluate for delaminations, debondings or air voids between multiple plies or between the FRP system and the masonry prior to testing, after reconditioning, and at any intermediate stage as prescribed in the test procedure.
Subjecting to a natural heating, the non - bonded areas show an upper temperature, represented on the video with an associated colour. This depends on the presence of an air film between the laminate and the application surface (in the debonding case) or between the laminas of the laminate (in the delamination case).

Figure 3. Execution test

The thermographic results consist in color pictures elaborated from the technician who executed the thermographic surveying. He is able to supply the purchaser with the quantifications of the eventual degraded areas, geometrically characterizing it, and to describe with a good approximation the inquired element. The following, are the documents produced:



Panel 1A – 1B T main = 40,42 ° C (on the 99,7 % of the surface)	**Panel 1C – 1D T main = 43,26 ° C** (on the 99,7 % of the surface)

Figure 4. Thermographic test results

3.4 Conditioning program

3.4.1 Freeze-Thaw Cycling

In a FRP- interface - masonry composite system, self-equilibrating stresses develop in two cases: differential thermal expansion and contraction of the FRP, interface and masonry and when the distribution of temperature over the cross-section of the FRP is non-linear. In the longitudinal direction, CFRP laminates have a coefficient of thermal expansion less then substrate, even negative. In regions of drastic temperature changes, this can negatively affect the bond characteristics and break down of the lamina. A lot of time was spent analyzing the freeze-thaw cycling capabilities of D.IN.E. laboratory facilities. After extensive research on previous experiments on temperature effects of FRP reinforcement, it has been concluded that there seemed to be a lack of datas for FRP for low temperatures. Each specimen requiring pre-conditioning was thus subjected to low temperature thermal cycling between 30°C and –18°C excursions with a 1-hour-hold at –18°C and 1-hour-hold at 30°C, achieving a 6 cycles per day rate. Figure 5 graphically represents the cycling procedure for a 24-hour period.

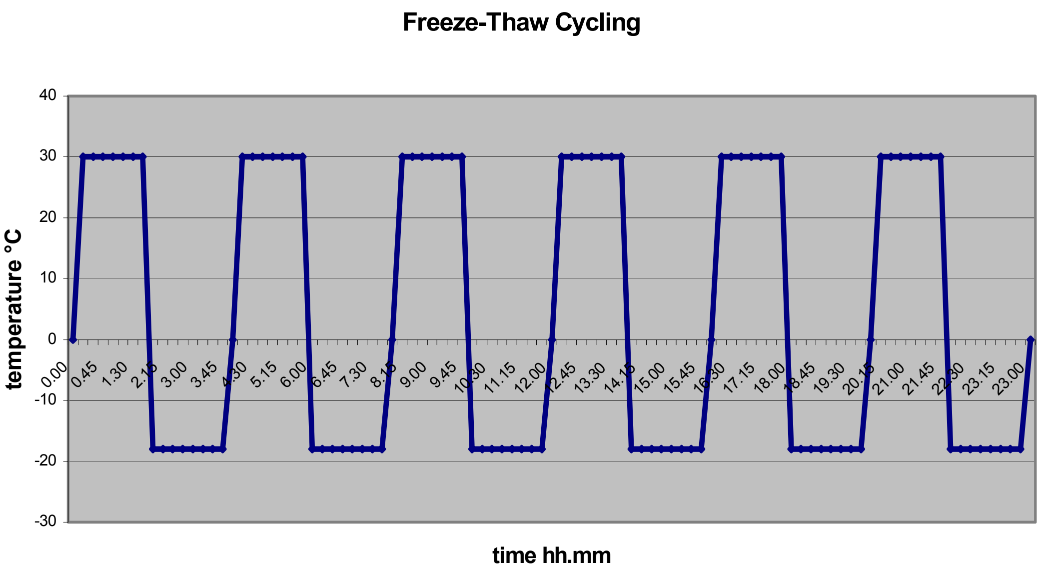


Figure 5. temperature – time diagram in the freeze thaw cycling

Specimens were exposed to 105 freeze-thaw cycles corresponding to 420 hours of exposure. Specimens that were not subjected to freeze-thaw cycling were stored in the structures testing laboratory at room temperature and standard relative humidity.


Figure 6-7. Specimens in the climatic chamber subjected to Freeze-Thaw Cycling

3.4.2 U.V. exposure

Each specimen requiring pre-conditioning was subjected to Xenon-Arc Exposure in laboratory environment for 50 cycles, with - intensity of 0,35±0,02W/mq and temperature of 63°C, have shown these results:

- Ultimate value of bond shear stress CFRP- support decay is about 3%
- All areas are still bonded (evaluated with thermographic obay)
- Small cracks appeared on the plaster along CFRP lamina perimeter

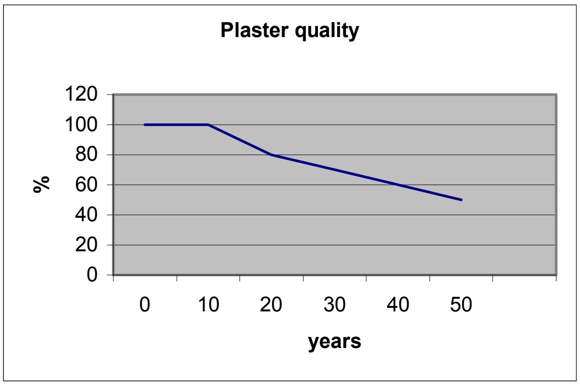
Cycle Description	*Un-insulated Black Panel Temperature, °C*	*Typical Irradiance*
18-h, consisting of alternating intervals of 102 min light only followed by 18 min of light with water spray	63±2	0,35±0,02W/mq a 340 nm
6 h dark, at 95 6 4 %RH with no water spray	38±2	41,5±2,5 W/mq da 300 a 400 nm

Table 6. Test cycles commonly used for xenon-arc exposure testing of plastics (ASTM D 2565-99)

Figure 8. Specimens in the climatic chamber subjected to U.V. exposure

4 CONCLUSION AND FUTURE RESEARCHS

Once performance decays, varying the number of freeze-thaw cycling and of xenon-arc exposure, will be evaluated, with reference to non conditioned specimens, the dimensioning of modifying factors will be possible.
In practice the cycle driving the same performance decay (for ex. lamina debonding) will be counted varying the conditioning of the specimen. If for example such a decay is obtained through the conditioning of freeze-thaw cycling after 100 cycles, through the conditioning of U.V. exposure after 150 cycles or through mid-normal conditions after 250 cycles, it will be possible to calculate the numerical value of the modifying factors for EBR FRP strenghtning system analyzed, when subjected to the examinated influencing agents (freeze-thaw and U.V. exposure).
This experimental results are not avaible yet, because the laboratory tests are still running. It is only possible to anticipate that at the end performance – time diagrams will be outlined, as shown in figure n.9



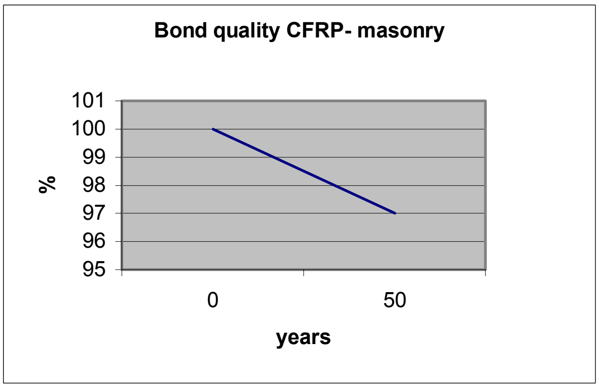


Figure 9. Performance – time diagrams

5 ACKNOWLEDGMENTS

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6 REFERENCES

[1] ACI (2000) Guide for the design and construction of externally bonded FRP systems for strengthening concrete structures, Reported by ACI Committee 440, Revised 12 July 2000
[2] Fib Bulletin No. 14 *Externally bonded FRP reinforcement for RC structures,* Federation International du Beton, Bruxelles, 2001
[3] ISO 15686-1 *"Buildings – Service life planning – part1: general principles."* International Organization for Standardization, 2000
[4] ASTM D 618 Standard Practice for Conditioning Plastics for Testing;. ASTM.
[5] D 2565-99 *Xenon –Arc Exposure of Plastics Intended for Outdoor Applications*
[6] ASTM C 666/C 666M – 03 Standard test method for resistance of concrete to rapid freezing and thawing
[7] ASTM D756-99 Determination of weight and shape changes of plastics under accelerated service conditions
[8] De Pascale A., *"Experimental determination of the service life of building components to schedule the operations of programmed maintenance"* Tesi di dottorato in "Tecnologie per il recupero edilizio e l'innovazione tecnologica", anno 2002
[9] Nicolella M., *Affidabilità e durabilità degli elementi costruttivi in edilizia – Un'ipotesi metodologica per il calcolo*, CUEN, Napoli. 2000.

Life-time estimation of polymeric glazing materials for solar applications

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ABSTRACT

The economic viability of solar collector systems for domestic hot water (DHW) generation is strongly linked to the cost of such systems. An attractive approach to cost reduction is to replace glass and metal parts with less expensive, lighter weight polymeric components. The use of polymeric materials also allows the benefits and cost savings associated with well established manufacturing processes, along with savings associated with improved fastening, reduced part count, and overall assembly refinements.

A key challenge is to maintain adequate system performance and assure requisite durability for extended lifetimes. Results of preliminary and ongoing screening tests for a large number of candidate polymeric glazing materials are presented. Based on these results, two specific glazings are selected to demonstrate how a service lifetime methodology can be applied to accurately predict the optical performance of these materials during in-service use. A summary is given for data obtained by outdoor exposure and indoor testing of polyvinyl chloride (PVC) and polycarbonate (PC) materials, and an initial risk analysis is given for the two materials. Screening tests and analyses for service lifetime prediction are discussed. A methodology that provides a way to derive correlations between degradation experienced by materials exposed to controlled accelerated laboratory exposure conditions and materials exposed to in-service conditions is given, and a validation is presented for the methodology based upon durability test results for PVC and PC.

KEYWORDS

Service life estimation, limeric glazing, degradation modeling, accelerated testing, outdoor weathering

1 INTRODUCTION

Polymeric glazings offer significant potential for cost savings both as direct substitutes for glass cover plates in traditional solar collector systems and as an integral part of all-polymeric systems. A review of polymeric solar collector systems development efforts is provided in [1]. Cost savings result from lower base material costs and lower costs associated with shipping, handling and installation. Glazings must have high transmittance across the solar spectrum and must be able to survive 10 to 20 y exposure to service conditions including operating at temperatures of 55 to 90°C and in solar ultraviolet (UV) light. They must also retain mechanical integrity e.g., impact resistance and flexural rigidity, under these harsh environmental stresses. The emphasis of current efforts is to identify new or improved candidate glazings and to evaluate their optical and mechanical durability during exposure to actual and simulated in-service conditions.

Recently, several reviews of candidate polymeric glazing materials have been undertaken [2-4]. These were guided by the expectation that advances in the polymer manufacturing and materials industry would allow identification of potential new and improved collector glazing candidates. An international collaborative effort surveyed commercial producers of advanced polymer materials in the U. S., Europe, and Japan [2]. The most promising class of polymers were fluoropolymers. These have excellent thermal and optical durability but are expensive and are limited to use with thin film collector designs. Film products such as Tefzel® (ethylene-tetrafluoroethylene copolymer; ETFE), Duralar® (also an ETFE), Halar®, Teflon®, and Kynar® exhibit very high spectral transmittance and many have sufficient tear resistance to be considered as collector glazings [3]. Suitably UV-stabilized polyetherimide (PEI), polyimide (PI), and polycarbonate (PC) were also suggested for consideration, although PI is quite expensive. Preliminary exposure test data for several dozen polymeric glazing materials being screened was reported in [3]; most materials identified by [2] were included. These exposures have continued and further results are discussed herein. Another complementary review also surveyed potential polymeric glazing materials [4]. In addition to twin walled PC, fluoropolymer films, and multilayered polyethylene (PE) films under test, consideration of polyurethane films, silicones, enhanced acrylics, clay-filled thermoplastics, and polycyclohexylethylene (PeCHE) were also recommended.

Screening tests have revealed the more promising candidates, along with glazing materials that have failed. The most common mode of failure has been yellowing of the glazing material. These have generally included non-fluoropolymer thin film materials (PET and PE, including UV-stabilized versions), and non-UV stabilized PC constructions. Additional (less common) modes of failure include materials developing a cloudy white opaque appearance, temperature-related deformation and/or discoloration, and physical damage caused by hail and other natural weathering events. Materials that have maintained high solar-weighted hemispherical transmittance values (>90%) after more than 2 years outdoor and accelerated exposure include: Kynar®, Duralar®, Tefzel®, and Halar® and PC with UV-screening layers.

Polycarbonate has high optical clarity and excellent impact strength. However, it will yellow during UV exposure and become brittle. Recently, stabilized versions of PC have been developed. For example, Bayer has two products designated APEC 5391 and APEC 5393. The first is a thermally stabilized formulation, which is offered for a maximum continuous use temperature of up to 180°C, and the second is stabilized for UV exposure and elevated temperatures. General Electric has incorporated an integral UV-screening coating into a number of their Lexan products. Because PC has excellent initial properties and is available in a variety of forms, e.g., sheet or channeled, suitable for use with solar collectors, is has been extensively studied as a promising glazing candidate. Parallel test results for PVC serve as a control, because it is known to weather poorly.

2 DURABILITY EXPOSURE TESTING

From 1993-2002, numerous samples of PVC and PC materials were exposed to accelerated life testing in laboratories and to the outdoor environment at test sites located in Europe and in the USA by colleagues participating in the IEA Task 27 Solar Heating and Cooling Program Working Group. The details of these test results are provided in [5]. Samples of PC and PVC, along with other candidate polymeric glazing materials, were subjected to in-service outdoor and accelerated laboratory exposure conditions. Outdoor testing was carried out in Switzerland at the Institut für Solartechnik (SPF), Germany at the ISE in Freiburg, and at three sites in the United States, namely, Golden, CO; Phoenix, AZ; and Miami, FL. A precise and detailed knowledge of the specific environmental stress conditions experienced by weathered samples is needed to allow understanding of site-specific performance losses and to permit service lifetime prediction of candidate glazings. Consequently, each operational exposure site is fully equipped with the appropriate meteorological and radiometric instrumentation and data-logging capability.

2.1 Outdoor Exposure Testing

The materials tested are for the intended use in solar thermal flat plate collectors. Thus, the samples for outdoor exposure were attached to mini-collector boxes of 15 cm side legths. To simulate the elevated temperature collector covers are exposed to, the "mini-collectors" are made of solar selective coated stainless steel. A thermocouple is affixed to the glazing material to monitor sample temperature, and a reflective light shield hood is used to prevent direct heating of the thermocouple.
The samples prepared in this way were exposed to the ambient climate at locations in Europe and in the USA, facing south at an inclination angle equal to the lattitude of the site. The spectral transmittance of all samples was measured prior to exposure. After some time, some of the samples were remeasured and exposed again without any cleaning. Other samples were measured before and after cleaning and then exposed again. Solar-weighted transmittance values integrated over the solar spectrum (τ_{sol}) and between 400-600 nm ($\tau_{400\text{-}600}$) are computed as degradation indicators. The bandwidth 400-600 nm is useful because degradation of optical transmittance of many polymeric glazing materials is most pronounced in that spectral range.
Representative $\tau_{(400\text{-}600)}$ data are plotted in Figs 1 and 2 for PC and PVC, respectively. Fig 1 shows a loss in $\tau_{(400\text{-}600)}$ of about 5% per year for samples of PCexposed outdoors in Europe and the US. The PVC glazing material degrades very rapidly when exposed to UV light. Fig 2 presents outdoor test results for PVC exposed in the United States and Europe. Samples of PVC exhibited between 25-40% loss in $\tau_{(400\text{-}600)}$ after only one year exposure, depending upon the outdoor site.

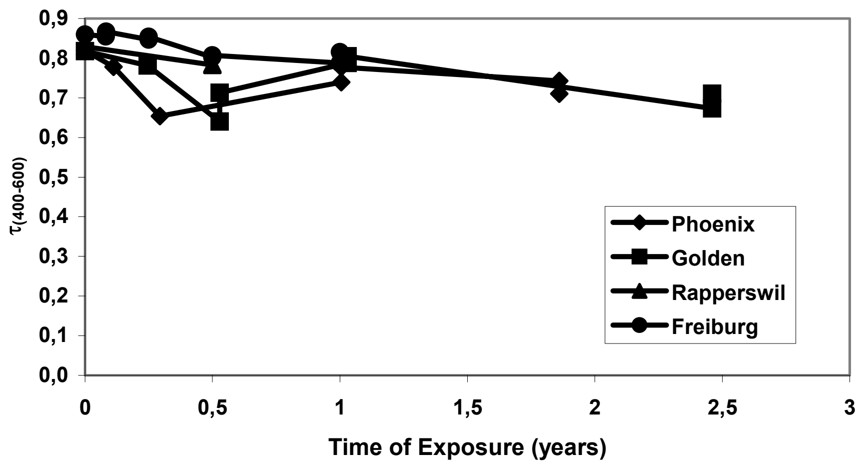

Figure 1 Outdoor exposure test results for PC

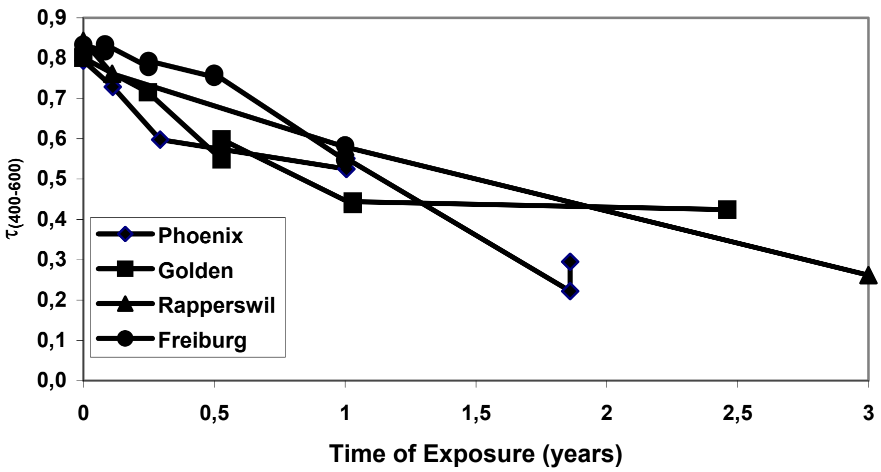

Figure 2 Outdoor exposure test results for PVC

2.2 Accelerated Laboratory Exposure Testing

Accelerated indoor testing was carried out with different types of test equipment available at the participating laboratories. Several test protocols were performed using corresponding types of exposure chambers. In the first type, UV exposure was combined with various combinations of elevated temperature and a defined level of relative humidity (RH), i.e., 60°C / 80% RH, 80°C / 40% RH, and 50°C / 95% RH. These tests were performed in climatic cabinets in Rapperswil and Freiburg with an unfiltered metal halide (HMI) lamp as a light source. The intensity of the irradiation compared to an air-mass (AM) 1.5 solar spectrum is about 3 times as much UVA and 7 times as much UVB. In the second type of exposure test, an Atlas Ci5000 Weather-Ometer® (WOM) was operated at 60ºC and 60% RH, and an irradiation level of about twice an AM 1.5 solar spectrum throughout the UV and visible portion of the spectrum. In the final test protocol, an Atlas XR35 WOM – SPART 14 test was used. The SPART 14 test procedure was originally developed for clear coats in automotive paint systems. The test is a weatherability test that includes acidic rain spraying. In test method SPART 14, which is a modification of SAE J1960 [6], the Xenon arc light source is filtered through borosilicate filters and has an irradiance level of 0.5 W/m^2 at 340 nm; this corresponds to an intensity of roughly 1.4 times an AM 1.5 spectrum. The test cycle is comprised of a) 40 min of light only; b) 20 min of light with water sprayed on the front surface of the sample; c) 60 min of light only; and d) 60 min of no light with water sprayed on the back surface of the sample. Every fourteenth cycle, the water used to spray the front of the samples is acidic, with a pH of 3.2. The black standard temperature and relative humidity during light periods are 70°C and 75%, respectively. The chamber temperature and relative humidity during the dark periods are 38°C and 95%.

An exposure time of 1000 h (~6 weeks) in the SPART 14 test is estimated to correspond to about 1.3 years of outdoor testing in Miami, Florida for automotive paints. Thus, 4000 h of SPART 14 testing corresponds to about 5 years outdoors in Florida. However, one can assume that the temperature of an automotive coating will be at least 10ºC higher than for transparent low light absorbing glazing materials. Consequently, the acceleration factor for the glazing can be estimated to be a factor of 2 higher. Accordingly, 1000 h of artificial weathering corresponds to 2.5 y outdoors.

Highly accelerated exposure testing of selected samples using natural sunlight was also performed at NREL [7]. Parallel testing with the relevant stress factors of UV, temperature, RH, and acid spray at different levels was intended to allow the sensitivity of materials degradation to these factors to be quantified, and allow damage function models to be evaluated. These in turn can be used to compare the time-dependent performance of these materials with measured results from in-service outdoor exposure.

The values obtained for $\tau_{(400\text{-}600)}$ are plotted in Figs 3 and 4 for APEC 9353, and PVC after the different types of exposure. Fig 3 shows that results for PCexposed in the SPART 14 chamber are in good agreement with Ci5000 data. However, exposure of PCin the unfiltered metal halide chambers is much more severe than in the Ci5000 and SPART 14. With the unfiltered metal halide light source, a ~15% loss in $\tau_{(400\text{-}600)}$ occurs after only 25 days, whereas it took roughly 100 days for an equivalent loss to occur in the Ci5000 and SPART 14.

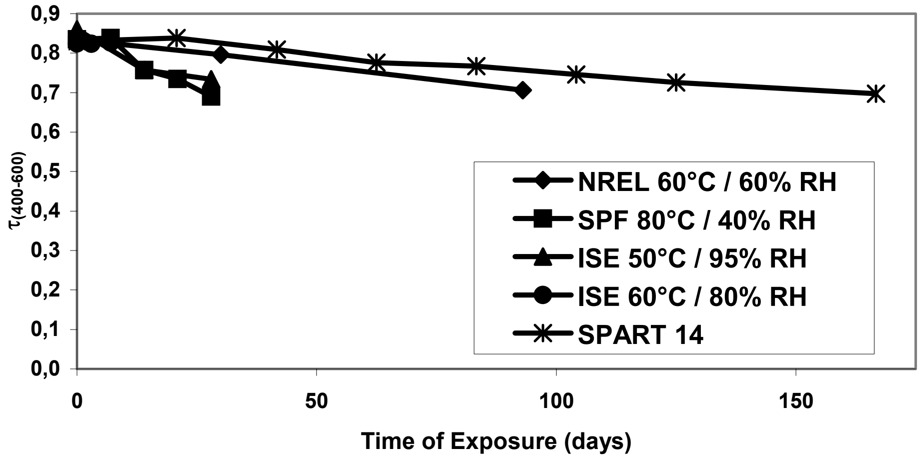

Figure 3 Accelerated exposure test results for PC

Accelerated exposure test results for PVC are provided in Fig 4. Exposure of PVC in the unfiltered metal halide chambers at a variety of temperature and RH conditions produced rapid degradation. Results for Ci5000 WOM exposures were less severe, although precipitous degradation did occur in fairly short time periods. PVC exposed to the SPART 14 chamber conditions results in considerably less degradation than for the unfiltered metal halide and Ci5000 chamber exposures.

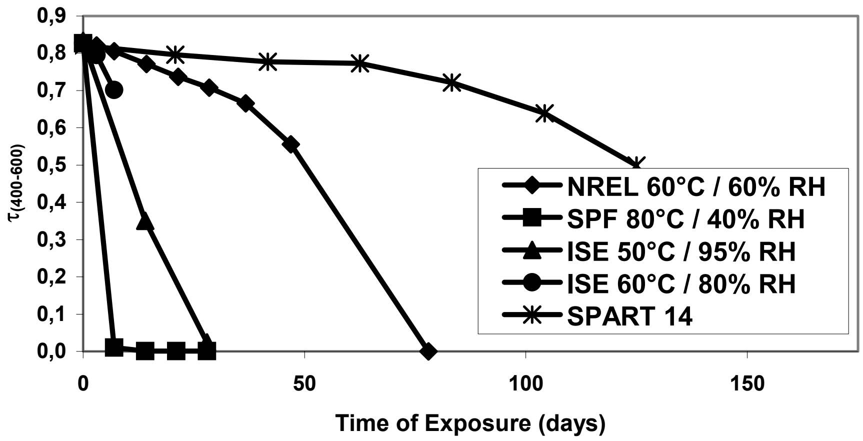

Figure 4 Accelerated exposure test results for PVC

The approach developed by the IEA Working Group on Materials for Solar Thermal Collectors [8] is applied below to PVC and UV stabilised PC cover plate materials to illustrate how the general methodology can be used to assess the durability of polymeric-type materials.

3. ANALYSIS OF DURABILITY RESULTS FROM ACCELERATED AGING

Using artificially aged samples from screening tests, changes in the key functional properties or the selected degradation indicators are analyzed with respect to the associated changes in the materials. The analyses were made to identify the predominant degradation mechanisms of the materials. Possible mechanisms of degradation of the PC glazing were assumed to be (a) photooxidation (PO), (b) thermal oxidation, and (c) combined photooxidation and hydrolysis. From the screening tests, it was concluded that only photooxidation contributes significantly to the service life of the glazing. A suitable time-transformation function yields the acceleration factor:

$$a_{PO} = \frac{\left(I^{p} \cdot e^{-E/kT}\right)_{acc}}{\left(I^{p} \cdot e^{-E/kT}\right)_{ref}} \tag{1}$$

where I is the intensity of photoreactive light, T is temperature, E is an activation energy, p is a material dependent constant, “acc” is accelerated test conditions, and “ref” is some set of reference conditions, e.g., use conditions.

For the PVC glazing, degradation mechanisms that could reduce the service life were assumed to be (a) dehydrochlorinization, (b) photooxidation and (c) physical aging. For (a), the mechanism is a chain reaction type because hydrogen chloride formed from the dehydrogenation reaction acts also as a catalyst for this reaction. The reaction is consequently difficult to model mathematically in a simple way and thus, it is also difficult to express the rate of degradation in terms of a time-transformation function. The best time-transformation function for the PVC degradation was the same general photooxidation time-transformation function used to model the degradation of the PC glazing (Eq. 1).

During life-testing, PC and PVC glazing materials were exposed to the various accelerated conditions discussed above. Hemispherical transmittance measurements were made to characterize the loss in optical performance of the glazing materials during these exposures. Performance-versus-time data were thereafter used to determine the parameters of the time-transformation function (Equ. 1). The results, obtained from a subset of the data accumulated, are shown in Table 1. Values of the activation energies, E, derived are reasonable for photo-thermal degradation mechanisms. The value of $p \sim 2/3$ for PVC indicates that some shielding or rate limiting reactions occur and do not allow all photons to participate in degradation. For the UV-stabilized PC sample, the value of $p = 1$ indicates that exposure of this material follows strict reciprocity. Thus, all incident photons fully contribute to the degradation reactions, even at elevated levels of irradiation.

Polymer Glazing	*p*	*E (kcal/mole)*
PVC	0.669	8.440
UV-Stabilized PC	1.093	6.688

Table 1. Coefficients derived from accelerated exposure for the tested polymeric glazing materials

4. VALIDATION OF METHODOLOGY

If it is assumed that the rate in transmittance change is constant if the surface temperature and the UV-light intensity are maintained at the same values during the time interval Δt_i, then the transmittance change $\Delta\tau_i$ may be expressed as

$$\Delta\tau_i = A\left(I_{UV}\right)^p \ \Delta t_i \ e^{-E/kT} \tag{2}$$

using the time-transformation function shown in Equ. 1. The parameter A is a constant independent of surface temperature and light intensity but it is material dependent. It may be determined from the same series of aging tests as used to determine the activation energy E and the parameter p. For $\Delta\tau_i$ equal to the mean global transmittance between an even more narrow bandwidth (400 and 500 nm; chosen to accentuate the region over which degradation occurs, resulting in a more highly sensitive degradation indicator), the values of A were estimated as 2892 $(MJ/m^2)^{-1}$ for PVC and 5.497 $(MJ/m^2)^{-1}$ for UV-stabilized PC.

By integrating Eq. 2, Eq. 3 is obtained.

$$\Delta\tau_i(t) = A\int_0^t \left[I_{UV}(t)\right]^p \ e^{-E/kT(t)} \, dt \tag{3}$$

Applying Equ. 3, the expected transmittance after different time-periods of outdoor exposure may be estimated.

Using the values of the coefficients E and p from Table 1 and the time-monitored values of sample temperature and UV irradiance, the loss in performance was predicted for both the PVC and the UV-stabilized PC as exposed outdoors in Golden, CO, and Phoenix, AZ. Predicted values were then compared with actual measured data for these materials exposed at these sites. The results are shown

in Fig 5. The time-dependent changes in the weathering variables result in the irregular shapes of the predicted curves. Excellent agreement is evident between the measured and predicted data. Thus, the phenomenological approach to data analysis is validated i.e., obtaining model coefficients from accelerated test results and then using these coefficients to predict time-variable in-service degradation.

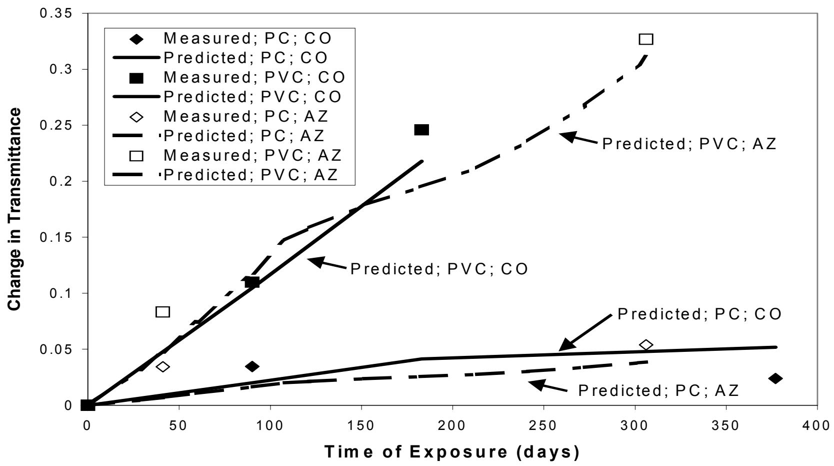

Figure 5: Comparison of the actual and predicted changes (loss) in hemispherical transmittance between 400-500 nm of PVC and PC for exposures of up to 380 days at Golden, CO and Phoenix, AZ.

5. CONCLUSIONS

Durability test data for both accelerated laboratory conditions and outdoor in-service conditions have been presented for PC and PVC glazing materials. Some of the accelerated exposure data were used to demonstrate how to derive damage functions that allow prediction of performance degradation. This methodology also allows the effect of multiple stress factors to be modeled. The usefulness and validity of this approach has then been confirmed by comparing predicted results with actual measured data for samples exposed to variable outdoor conditions. Consequently, highly abbreviated testing times at elevated stress conditions can be substituted for long-time exposures at lower stress levels. The procedure developed allows much shorter development cycle times for new materials and allows improvements to be identified and readily incorporated into new products.

REFERENCES

1. Davidson, J. H. & Mantell, S. C. & Jorgensen, G. J. 2003, 'Status of Development of Polymeric Solar Water Heating Systems', in *Advances in Solar Energy, An Annual Review of Research and Development*, 15, D. Y. Goswami, Ed., American Solar Energy Society, Inc., Boulder, CO, pp. 149-186.
2. Möller, K., 1996 'Identification of New Types of Transparent Polymeric Materials', *IEA Task 10 Working Group on Materials in Solar Thermal Collectors Report*.
3. Raman, R. et al 2000, 'A Review of Polymer Materials for Solar Water Heating Systems', in *Journal of Solar Energy Engineering*, 122[2], pp. 92-100.
4. 'Solar Hot Water Heating Systems Identification of Plastic Materials for Low Cost Glazings', *NREL Subcontract No. TAR-9-29449-02 final report*, PolyNEW Inc., Golden, CO, 2000.
5. Jorgensen, G. J., et al, 'Case Study on Polymeric Glazings', in *Performance and Durability Assessment of Optical Materials for Solar Thermal Systems*, A. W. Czamderna, Ed., Elsevier Science, in press.
6. SAE J1960, 'Accelerated Exposure of Automotive Exterior Materials Using a Controlled Irradiance Water-Cooled Xenon Arc Apparatus', Society of Automotive Engineers, 400 Commonwealth Drive, Warrendale, PA 15096.
7. Jorgensen G., et al, 2002, 'Use of Uniformly Distributed Concentrated Sunlight for Highly Accelerated Testing of Coatings', in *Service Life Prediction Methodology and Metrologies, ACS Symposium Series 805*, J. W. Martin and D. R. Bauer, Eds., American Chemical Society, Oxford University Press, Washington, pp. 100-118.
8. Carlsson, B., et al, 2003 'Weathering of Polymer Products: Assessment of Service Life of Solar Thermal Components by Accelerated Life Testing', in *Proceedings of the 1st European Weathering Symposium EWS (XXIIIrd Colloquium of Danubian Countries on Natural and Artificial Ageing of Polymers)*, Gesellschaft für Umweltsimulation e.V., Prague, Czech Republic, September 25-26, 2003.

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Degradation of the efficiency of vacuum insulation panels by gas permeation through barrier films

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ABSTRACT

Vacuum insulation panels (VIP) have the potential to reduce the thickness of building insulation by a factor of ten. They consist of a porous matrix encapsulated by a liner of foils, which is evacuated down to a final pressure below one mbar. Metal foils have the disadvantage of high thermal conductivity spoiling the u-value of the VIP at the borders and edges. More suitable are polymeric films equipped with barrier coatings reducing the permeation of water vapour and air by some orders of magnitude clearly below 1 $cm^3/(m^2*d)$ for oxygen and 1 $g/(m^2*d)$ for water.

The required lifetime of the panels of some 30-50 years need an extremely low permeation rate over the entire lifetime in order to maintain the vacuum conditions and the thermal properties. The samples are exposed to high temperatures, at south-oriented walls especially. The knowledge of the temperature-dependent permeation rate is needed for modelling the permeation and the vacuum conditions over time in order to estimate the lifetime of the panels.

The paper describes the measurement of the temperature dependent permeation coefficient for different combinations of polymeric films and inorganic barrier coatings enforced by additional sol-gel based coatings, and initial results of modelling the service lifetime by integrating time-series of in-use conditions.

KEYWORDS

Vacuum isolation panels, barriere films, water vapour permeation, lifetime assessment.

1 INTRODUCTION

1.1 Development of gas barrier layers with hybridpolymers

As of today the polymer films, having sufficiently low O_2 and water vapor permeability values in combination with a vacuum coated layer of Al, AlO_X or SiO_x, are available as flexible packaging solution for the sensitive foods such that these sensitive foods can be protected against oxidation, moisture effect or damages throughout their shelf-lives. The evaporated layers are manufactured industrially in big quantities and available at convenient prices. Their permeability values are below 1 cm³/m² d bar for O_2 and below 1 g/m² d for water vapor..
These film combinations generally do not meet the requirements for the technical applications demanding high barrier properties. A maximum water vapor transmission rate of 10^{-2} g/m² d is required for the encapsulation of solar cells in flexible PV-module applications in order to guarantee the functional efficiency of the cells during the required life-time. The water vapor permeability value required for the vacuum insulation panel (VIP) applications vary below 10^{-2} to 10^{-4} g/m² d depending on the type of the application and the required lifetime of the VIP. The highest barrier properties are required for the encapsulation of organic light emitting diodes (OLED) and organic solar cells (OPV). In the actual state of the art, permeation properties ofhigh-barrier films are in theorder of magnitude of 10^{-4} cm³/m²·d·bar for the O_2 and 10^{-3} g/m²·d for the water vapor permeability.

1.2 Permeability properties of polymers and vacuum coated layers

The permeability values of the individual polymers that are used in technical applications must be considerably reduced. A technically well-controlled and economically advantageous possibility represents the vacuum web coating. By this way, the films (or paper) are evaporation coated with metallic or ceramic layers within a thickness range of 20-100 nm. At a vacuum level of 10^{-4} mbar, the material to be vaporized is heated up and vaporized by an electron beam, in which it condenses on the cooling roll and directed to the coated web. Plasma pre-treatment supports the adhesion by critical coating substrates. Metallic coating layers are rather soft, while the ceramic layers like SiO_x are brittle. However, this behavior shows itself only by unprotected layers, as soon as an additional organic coating like varnish or adhesive is applied, these layers are also relatively insensitive. The permeability through the evaporated layer is definitely through the defects in the layer. These defects can be aroused due to the roughness of the substrate surface or contaminations existing in the substrate such as anti-block particles. By the appropriate actions, the number and the size of the defects can be minimized, but it is not possible to get rid of them completely. The rest of the layer can be considered as impermeable. The numerical simulations over the concentration profile of a substance permeating through the layers of the adjacent polymer have shown that there is a critical layer thickness for the polymer over which no barrier improvement can be achieved. The critical layer thickness exclusively depends on the size of the defects. For a medium defect size of 1 µm², the critical polymer layer thickness is below 2,5 µm.
The concept of a barrier improvement factor (BIF) is commonly used in order to describe the performance of a vacuum deposited barrier layer on a polymer film. This term is usually defined as the ratio of the permeability of the base substrate (Q_0) and the permeability of the coated film (Q):

$$BIF = Q_0 / Q.$$

In this representation the improvement factor still depends on the thickness of the substrate (on the value of Q_0). Based on the theoretical and experimental investigations, the permeability of the coated substrate film, Q, is independent of the substrate thickness over a wide range; therefore another simplification possibility arises, a normalized improvement factor BIF_{100} can be introduced that would in practice be measured on a coated substrate of 100 µm thickness:

$$BIF_{100} = Q_{100} / Q$$

Thus, the permeability of a coated substrate film is basically given by the normalized permeability of the substrate polymer material and the normalized improvement factor.

In most cases thin inorganic barrier layers must be further coated with other polymer layers. These layers are in general lacquers or adhesive layers, on that, if the application weight lies above the critical layer thickness, the above-mentioned simplification can be used in such a way that a 3-layer system arises. For this structure, the total permeability Q can be approximated via the equation:

$$Q^{-1} = BIF_{100} / Q_{1,100} + BIF_{100} / Q_{2,100}$$

In contrast to the usual polymer film materials of which the permeability values are well known and reproducibly adjustable, still other influencing variables affect the structure properties and also the permeability of the coated polymer layers: The applied process conditions, the dryer process and by the reactive systems, the cross-linking conditions. The values of $Q_{2,100}$ are therefore subjected to bigger fluctuations.

1.3 Sol-gel based barrier coatings

In order to achieve further improvements in the development of high barrier multi-layers, the class of the inorganic-organic hybrid polymers must be introduced. These consist of an inorganic silicate-network, which is cross-linked to the organic bridges. The ratio between the organic and inorganic portion determines the properties of the formulation of these kinds of materials. The properties are influenced almost arbitrarily by the functional groups and hetero atoms of the inorganic structure elements. Depending on the type and chemistry of the organic portion, these coatings are hardened thermally or radiated chemically (preferably by UV light). Glass-like structures are obtained, which also reveals a high density (barrier effect) as of glass.

It is possible to apply these sol-gel coatings by conventional lacquering process techniques. Hereby it is possible to apply layers in a low thickness of 1µm. In connection with inorganic vacuum deposited barrier layers, there appears to be a phenomenon, indicating a synergistic effect between the vacuum deposited and the lacquered sol-gel-layer: The barrier improvement is found to be considerably higher than the expected values calculated according to the above-mentioned equations. This is proved by means of the measured permeability values of the films, where these lacquer is directly applied onto the polymeric film in comparison to the values of the films, where it had been applied on top of an inorganic layer. The difference is approx. one order of magnitude: Lower permeability values were obtained for the films applied on the metallized or the oxidized layer. This may be due to the formation of covalent bonds between the silanol-groups of the sol-gel lacquer and the atoms of the inorganic barrier layers. Thereby, the density increases in the interface, the defect size decreases and with that also the total defect area decreases.

2. MEASUREMENT OF THE TEMPERATURE-DEPENDENT WATER VAPOUR PERMEATION

2.1 Experimental set-up

The gas permeation rates are usually determined by standard test procedures at a constant temperature. VIPs in use are exposed to changing temperatures and humidity concentrations, however. Therefore the temperature dependence of the permeation is needed for the determination of the vacuum conditions over time in order to estimate the service lifetime. Moreover, a more sensitive measurement procedure is needed for characterisation of the permeation rate. We used a set-up based on a mass-spectrometer. The sample film is mounted on a high-vacuum flange of 100mm diameter by means of a butyl sealant and UHV-metal sealant and exposed to controlled temperature and humidity conditions in a climatic cabinet. The volume behind the sample film is filled with argon under atmospheric pressure. A high vacuum valve allows the extraction of a small amount of gas into the high vacuum part for analysis by the quadrupole mass spectrometer (see schematic in figure 1). All tubes are heated to 80 °C in order to avoid condensation of the water vapour when the pressure is reduced. The pressure behind the sample is monitored for correction of the measured partial pressures.

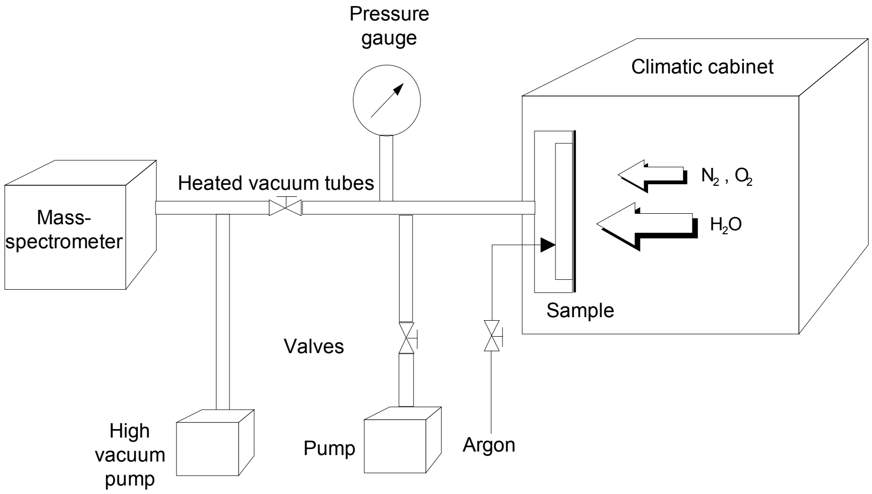

Figure 1. Set-up of the temperature-dependent permeation testing

A fast increase of the humidity until a partial pressure p_{CC} is reached in the cabinet after pre-conditioning the set-up to the set temperature causes the start of the water permeation like a response function (figure 2). The progress of the humidity content behind the foil p(t) is a measure for the permeation according to:

$$p(t) = p_{KS} (1-\exp(\tau^*t)) \qquad (1)$$

where the exponential factor τ can be used for the determination of the permeation constant P(T) at the temperature T.

$$P(T) = t^*K/T \qquad (2)$$

The constant factor K is given by the geometry and the gas constant. The samples are dried before a measurement at another temperature is performed. The reproducibility was found to be very good.

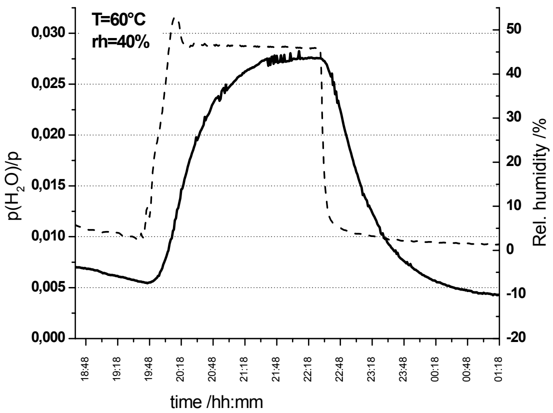

Figure 2: Partial pressure of water-vapour (solid line) behind the film as response of a rapid humidity increase in the climatic cabinet (dashed line)

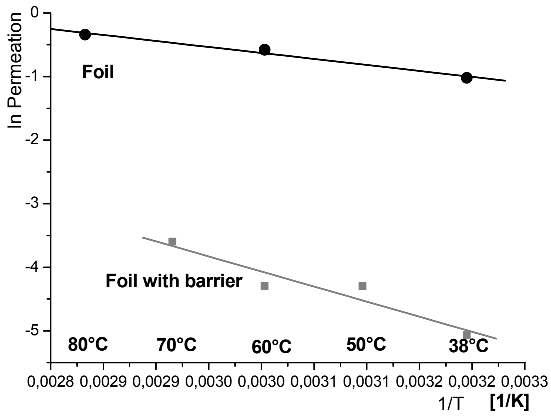

Figure 3. Permeation coefficient as a function of the reciprocal temperature (Arrhenius plot) for two samples: polymeric film (black dots) and polymeric film with barrier (grey squares)

2.2 Results

The investigated samples were one polymeric films used for food packaging and improved films that were coated with aluminum and alumina as permeation barriers resulting in a BIF of about 95. The products which are intended for VIP applications are still under development and should have better barrier properties.
The results shown in figure 3 approve the efficiency of the barrier coating, which was improved by a factor of 100 at the standard test temperature of 23 °C (see table 1). On the other hand, the stronger temperature dependence of the barrier – the activation energy is 40 kJ/mol compared to 24 kJ/mol - reduces the difference of the permeation with increasing temperature.

3. INTEGRAL PERMEATION IN A FACADE

3.1 Climatic conditions

Base for the further evaluation of the data were the results of monitoring the ambient climate (humidity, irradiation e.g.) and the surface temperature (figure 4) of a plaster wall during 2 years in Freiburg (Germany). These data allow the calculation of the absolute humidity or the water vapour partial pressure at the surface.

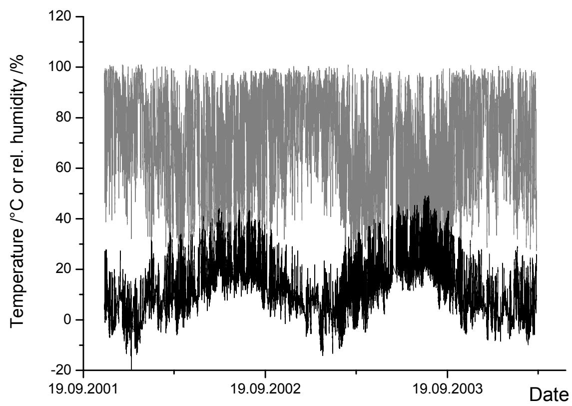

Figure 4. Surface temperature (black) of a white plaster and relative ambient humidity monitored for two years (5 minutes averages) in Freiburg (Germany)

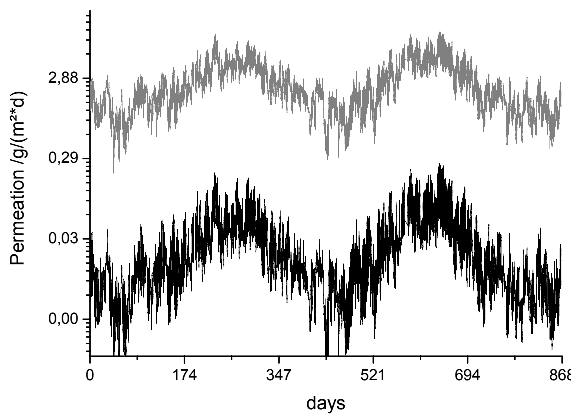

Figure 5. Modelled permeation rates for a polymeric film (grey) and a film with barrier coatings (black) for two years in Freiburg (Germany). Please note the logarithmic scale.

We made the following assumptions for first preliminary considerations:

- The temperature and rel. humidity of the film is the same as at the surface (gradients and time delays in the plaster are neglected).
- The film is considered as infinitely thin. No buffering of the water in the film is taken into account and it follows without any delays the changing surface temperature and partial pressure of the water vapour.
- The temperature dependence of the permeation coefficient follows the usual Arrhenius' law: $P(T) = P(296K) * \exp(-E_a/R\ (1/T-1/296K)$

The transient permeation rate, calculated for the data in figure 4 follows the temperature progress. The absolute level depends on the permeation coefficient of the respective film (figure 5).
Of higher importance for vacuum insulation panels is the cumulated water as shown in figure 6. The permeation through the film would sum up to roughly 2 kg per m² within two years and about 25 % more, if the maximum surface temperature would be 80 °C instead of 40 °C. The range of

temperatures was simply linearly expanded while keeping the same minimum temperatures for these calculations. The barrier coating would reduce the amount of permeated water to about 16 g per m². The higher activation energy for the permeation constant would result in an increase of 100 % at higher surface temperatures up to 80 °C (figure 6 and table 1).

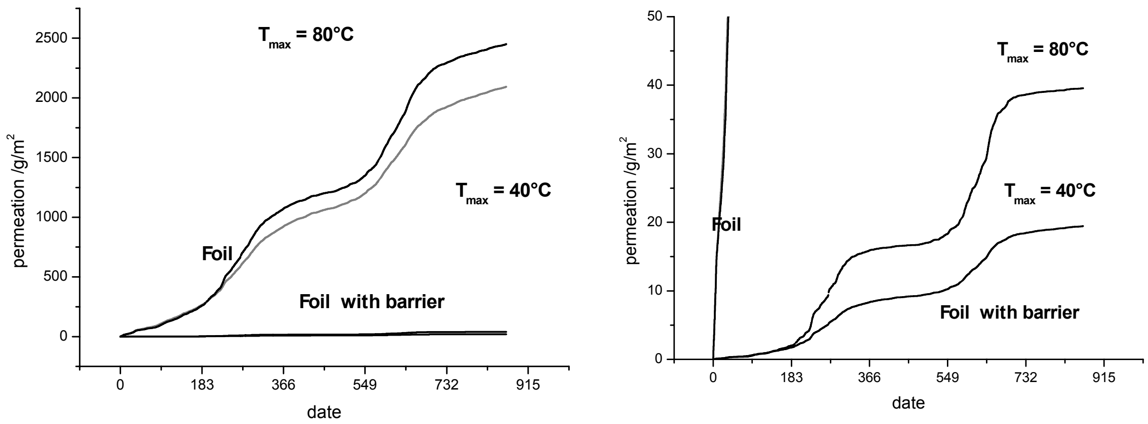

Figure 6. Cumulative permeation of water for a polymeric film and a film with barrier coatings (enlarged in the right hand sided diagram) for two years in Freiburg (Germany). The integration with the data shown in figure 5 yields the grey curve. A higher temperature load (black)

Property	*Dimension*	*Polymer-Film*	*Polymer + Barrier*
Permeation-coefficient @ 23°C	g/(m²*d*mbar)	0.275	0.0029
Activation energy E_a	kJ/mol	24	40
Integral permeation after 1 year	g/m²	883	8.2
Integral permeation after 1 year at enhanced temperatures	g/m²	1033	16.7

Table 1: Permeation properties and vacuum changes of films for vacuum isolation panels

The permeated water vapour increases the pressure inside the vacuum insulation panel and therefore the thermal conductivity, resulting in a degradation of the thermal insulation properties. A panel with a surface area of 1 m² has a content of about 10 dm³ at a thickness of 20 mm. The minimum requirements for the vacuum are 5mbar. The gettering of the water by the huge surface area of the pyrogenic silica helps to buffer the permeated water, otherwise this limit would already be reached with 0,045 g of water, which is of course completely evaporated at these low pressure conditions, accumulated during the design lifetime of 50 years. The permeation rate must be lower than the investigated barrier film by 3 orders of magnitude in order to reach this goal, if the gettering is neglected.

4 ACKNOWLEDGMENTS

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5 REFERENCES

S. Amberg-Schwab, M. Hoffmann, H. Bader and M. Gessler: "Inorganic-organic Polymers With Barrier Properties for Water Vapor, Oxygen and Flavor". J. of Sol-Gel Sci. Technol. 1/2, 141, 1998

H.-C. Langowski, A. Melzer und D. Schubert, Ultra High Barrier Layers for Technical Applications. Proc. Of the 45th Annual Technical Conference, Orlando, April 2002, Society of Vacuum Coaters, Albuquerque, 2002, S. 475-479

M. Hanika, H.-C. Langowski und U. Moosheimer, Layer defects and the synergetic effect between inorganic and organic layers. Proc. Of the 45th Annual Technical Conference, Orlando, April 2002, Society of Vacuum Coaters, Albuquerque, 2002, S. 523-528

H.-C. Langowski, Flexible Materialien mit ultrahohen Barriereeigenschaften. Vakuum in Forschung und Praxis 14 (2002) Nr. 5, S. 297-302

A. Hollaender, H.-C. Langowski and A. Wedel: "Flexible polymeric light-emitting diodes". Fraunhofer Institute for Applied Polymer Research, Golm / Fraunhofer Alliance Polymer Surfaces, Freising, 2003

K. Noller and K. Vasko, Flexible polymer barrier films for the encapsulation of solar cells. Proc. of the Photovoltaic Conference, Paris, June 2004.

Study on Durability and Service Lifetime Prediction of some Static Solar Energy Materials

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ABSTRACT

To achieve successful commercialisation of new advanced windows and solar facade components for buildings, the durability of these need to be demonstrated prior to installation by use of reliable and well-accepted test methods. In Task 27 of the International Energy Agency Solar Heating and Cooling Programme work has therefore been undertaken with the objective to develop a general methodology for durability test procedures and service lifetime prediction (SLP) methods adaptable to the wide va-riety of advanced optical materials and components used in energy efficient solar thermal and build-ings applications. As the result of this work a general methodology has been developed. The proposed methodology includes three steps: a) initial risk analysis of potential failure modes, b) screening testing/analysis for service life prediction and microclimate characterisation, and c) service life prediction involving mathematical modelling and life testing.

The general durability assessment methodology is now adopted to some static solar materials to allow prediction of service lifetime and to generate proposals for international standards. The work is performed in three case studies on anti-reflective and polymeric glazing materials, reflectors and solar facade absorbers. Anti-reflective materials that are studied include sol-gel coated and etched AR glasses. Reflectors that are studied include aluminium alloy based mirrors; some protected by clear coats, and glass mirror reflectors. Solar Facade Absorbers that are studied include coloured sputtered selective solar absorber coatings, absorber coatings made with sol-gel technology and thickness insensitive spectrally selective paints.

For validation purpose outdoor tests are performed involving monitoring of various kinds of climatic data during exposure such as global solar irradiation, UV-radiation, temperatures, humidity, precipitation, time of wetness, wind conditions, and atmospheric corrosivity. Such data will be used to predict expected deterioration in performance over time by making use of degradation models devel-oped from results of accelerated tests.

A number of accelerated screening tests have been performed including simulation of possible degradation in performance under the influence of high temperature, high humidity/condensation, UV, and corrosion loads; either single or combined loads. To identify degradation mechanisms for the tested materials various analytical techniques have been employed. Fault-tree analysis has also been made as an aid to better understand observed loss in performance and associated degradations mechanisms of the different materials studied. By use of time-transformation functions and measured microclimatic data, service lifetime of studied materials have been estimated and their predicted long-term performance compared with actually measured loss in performance of the studied materials during long-term outdoor exposure.

KEYWORDS
Service life prediction, accelerated testing, solar antireflective glazing materials, solar reflectors, solar facade absorbers

1. INTRODUCTION

The IEA Solar Heating and Cooling Programme, Task 27 on the Performance of Solar Facade Components started at the beginning of year 2000 with the objectives of developing and applying appropriate methods for assessment of durability, reliability and environmental impact of advanced components for solar building facades [Köhl 2000].

For the work on durability there are two main objectives. The first is to develop a general framework for durability test procedures and service lifetime prediction (SLP) methods that are applicable to a wide variety of advanced optical materials and components used in energy efficient solar thermal and buildings applications. The second is to apply the appropriate durability test tools to specific materials/components to allow prediction of service lifetime and to generate proposals for international standards.

As the result of this work, a general methodology has been developed [Carlsson *et al.* 2004], which is now adopted to some static solar materials. The work is performed in three case studies on anti-reflective glazing materials, reflectors and solar facade absorbers. Anti-reflective materials that are studied include sol-gel coated and etched AR glasses. Reflectors that are studied include aluminium alloy based mirrors; some protected by clear coats, and glass mirror reflectors. Solar Facade Absorbers that are studied include coloured sputtered selective solar absorber coatings, absorber coatings made with sol-gel technology and thickness insensitive spectrally selective paints.

2. GENERAL METHODOLOGY FOR DURABILITY ASSESSMENT

The methodology adopted by Task 27 includes three steps: a) initial risk analysis of potential failure modes, b) screening testing/analysis for service life prediction and microclimate characterisation, and c) service life prediction involving mathematical modelling and life testing.

2.1 Initial risk analysis

The initial risk analysis is performed with the aim of obtaining (a) a checklist of potential failure modes of the component and associated with those risks and critical component and material properties, degradation processes and stress factors, (b) a framework for the selection of test methods to verify performance and service life requirements, (c) a framework for describing previous test results for a specific component and its materials or a similar component and materials used in the component and classifying their relevance to the actual application, and (d) a framework for compiling and integrating all data on available component and material properties.

The programme of work in the initial step of service life assessment is structured into the following activities: a) Specify from an end-user point of view the expected function of the component and its materials, its performance and its service life requirement, and the intended in-use environments; b) Identify important functional properties defining the performance of the component and its materials, relevant test methods and requirements for qualification of the component with respect to performance; c) Identify potential failure modes and degradation mechanisms, relevant durability or life tests and requirements for qualification of the component and its materials as regards durability.

Function and general requirements	General requirements for long-term performance during design service time	In-use conditions and severity of environmental stress

Efficiently reflect solar radiation to increase the solar gain of a flat plate solar collector	Loss in material performance should not result in reduction of the solar system performance with more than 5%, in relative sense, during the material service life. Material service life should exceed 25 years	The reflector is exposed to open air conditions involving climatic stress of UV irradiation, high temperature, high humidity and moister, and the effect of icing. It may be exposed to corrosion promoting air pollutants and acid rain. It may also be subjected to mechanical loads from hail and wind, stress from mechanical fixing and due to its own weight. Soiling agents, e.g. from birds, may effect performance as well as cleaning agents as required to maintain performance

Table 1 Specification of end-user and product requirements for the booster reflectors studied in IEA SHCP Task 27

Critical functional properties	Test method for determining functional properties	Requirement for functional capability and long-term performance
Reflectance (specular, $\Delta\rho_{spec}$, and diffuse, $\Delta\rho_{dif}$)	ASTM E903 Standard test method for solar Absorptance, Reflectance, and Transmittance of Materials Using Integrating Spheres	$PC = 0.35\cdot\Delta\rho_{spec} + (0.1/C)\cdot\Delta\rho_{dif} < 0.05$ with concentration ratio C=1.5
Adhesion between coating and substrate	Visual assessment ISO 4624 Pull-off test for adhesion ISO 2409 Cross cut test	No blistering Adhesion > 1 MPa Degree 0 or 1

Table 2 Specification of critical functional properties of booster reflectors and requirements set up by the IEA SHCP Task 27 group

The first activity specifies in general terms the function of the component and service life requirement from an end-user and product point of view, and from that identifies the most important functional properties of the component and its materials. In Table 1 and Table 2 results are shown from the analysis made by the Task 27 group on booster reflectors. How important the function of the component is from an end-user and product point of view needs to be taken into consideration when formulating the performance requirements in terms of those functional properties. If the performance requirements are not fulfilled, the particular component is regarded as having failed. Performance requirements can be formulated on the basis of optical properties, mechanical strength, aesthetic values or other criteria related to the performance of the component and its materials.

Potential failure modes and important degradation processes should be identified after failures have been defined in terms of minimum performance levels. In general, there exist many kind of failure modes for a particular component and even the different parts of the component and the different damage mechanisms, which may lead to the same kind of failure, may sometimes be quite numerous. In Table 3 an example from the Task 27 work on booster reflectors is presented.

Fault tree analysis is a tool, which provides a logical structure relating failure to various damage modes and underlying chemical or physical changes. It has been used for the static solar materials studied in Task 27 to better understand observed loss in performance and associated degradations mechanisms of the different materials studied. Fig. 1 shows an example on how failure in optical performance of a booster reflector can be related to different damage or degradation mechanisms.

Failure/Damage mode / Degradation mechanism	Critical factors of environmental stress
Degradation of the protective layer	High humidity, high temperature, air pollutants (acid rain), UV irradiation, hail, wind

Corrosion of the reflecting layer	High humidity, high temperature, air pollutants (acid rain), and impacts from other materials in contact with reflecting layer
Surface abrasion	Sand, dust, cleaning, icing, hail, touching, scratching
Surface soiling	Microorganisms, wind, dust, pollutants, birds, etc
Degradation of the substrate	High humidity, high temperature, air pollutants (acid rain), UV irradiation, and impacts from other materials in contact with reflecting layer
Loss of adhesion of protective coating	High humidity, high temperature, air pollutants (acid rain), and UV irradiation
Loss of adhesion of reflector from substrate	High humidity, high temperature, air pollutants (acid rain), and UV irradiation

Table 3 Failure and associated degradation mechanisms and critical factors of environmental stress for booster reflectors related to unacceptable loss in optical performance as identified by the IEA SHCP Task 27 group

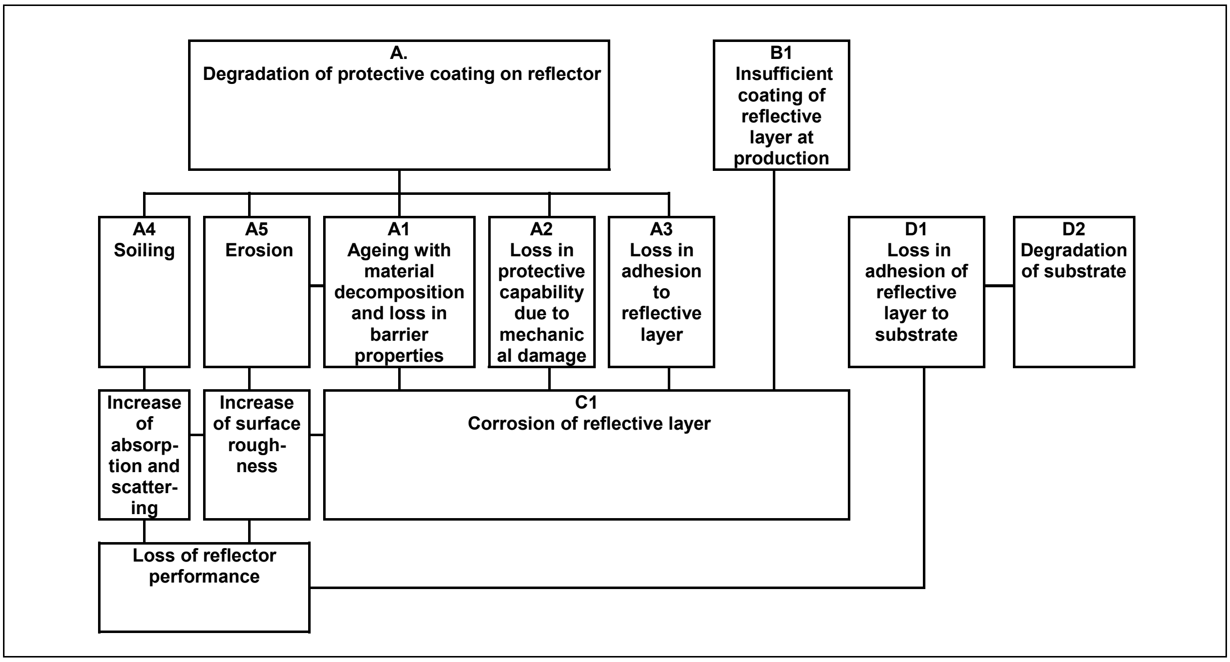


Figure 1 Representation of failure modes and associated degradation mechanisms for booster reflectors from the IEA SHCP Task 27 study

The risk associated with each potential failure/damage is taken as the point of departure to judge whether a particular failure mode needs to be further evaluated or not. An expert group may estimate the risks jointly by adopting the methodology of FMEA (Failure Modes and Effect Analysis) [Carlsson et al. 2004, Mc Dermott et al 1996]. In Table 4 the result of a risk analysis made by the Task 27 group on booster reflectors is presented.

Failure/Damage mode / Degradation process	Estimated risk number
A1 Degradation of the protective layer - Ageing with material decomposition	80
A2 Degradation of the protective layer- Loss in protective capability due to mechanical damage	40
A3 Degradation of the protective layer - Loss in adhesion to reflective layer	64
A4 Surface soiling	56
A5 Surface erosion	50
B1 Insufficient coating of reflective layer at production	70
C1 Corrosion of the reflecting layer (Result of mechanisms A1-A3, B1)	112
D1 Loss of adhesion of reflector from substrate	70
D2 Degradation of the substrate	32

Table 4 Risk assessment on different damage modes of booster reflectors made by the IEA SHCP group using the methodology of FMEA [Carlsson et al. 2004, Mc Dermott et al 1996]

2.2 Screening testing/analysis for service life prediction

Screening testing is thereafter conducted with the purpose of qualitatively assessing the importance of the different degradation mechanisms and degradation factors identified in the initial risk analysis of potential life-limiting processes.

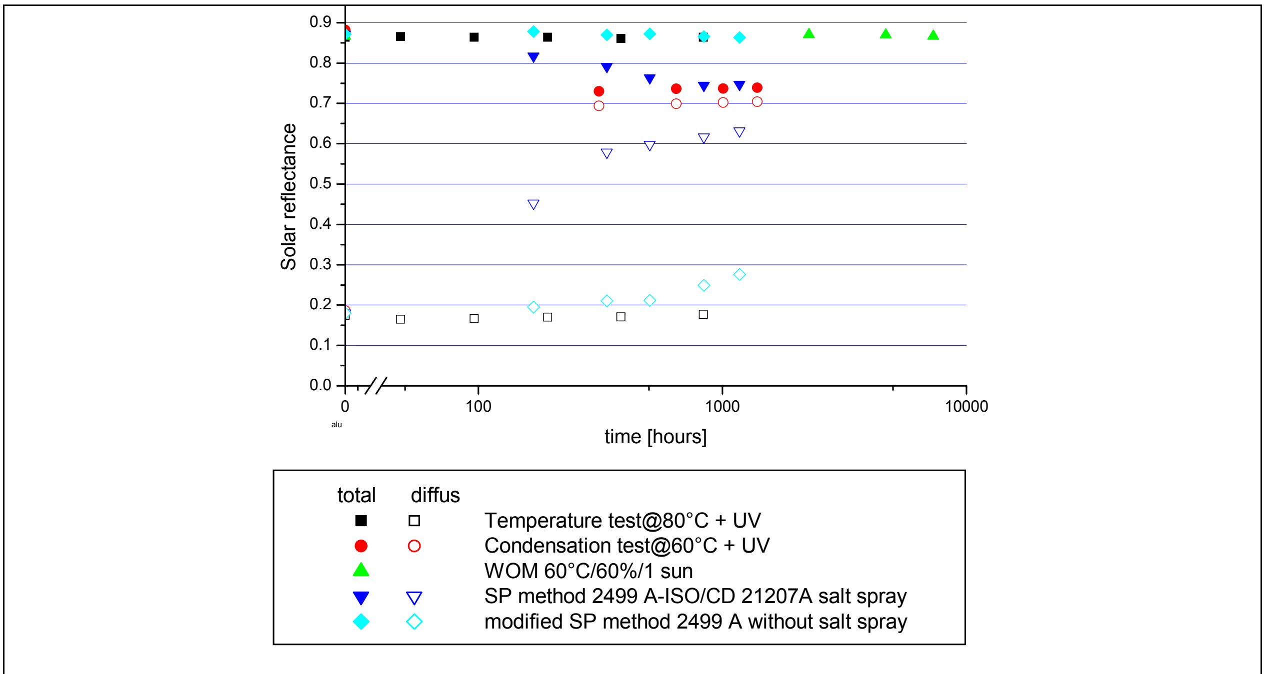


Figure 2 Results from screening tests on pure Aluminium in the IEA Task 27 study

When selecting the most suitable test methods for screening testing, it is important to select those with test conditions representing the most critical combination of degradation factors.

Using artificially aged samples from the screening testing, changes in the key functional properties or the selected degradation indicators are analysed with respect to associated material changes. This is made in order to identify the predominant degradation mechanisms of the materials in the component. When the predominant degradation mechanisms have been identified also the predominant degradation factors and the critical service conditions determining the service life will be known. Screening testing and analysis of material change associated with deterioration in performance during ageing should therefore be performed in parallel. Suitable techniques for analysis of material changes due to ageing may vary considerably.

On the static solar materials of Task 27, a number of accelerated screening have been performed including simulation of possible degradation in performance under the influence of high temperature, high humidity/condensation, UV, and corrosion loads; either single or combined loads.

In Fig. 2 the results from a series of screening tests on pure aluminium, used as reference reflector material, are shown as an example of result from the Task 27 study. Degradation in optical performance is observed mainly, as expected, in the corrosion tests. To identify degradation mechanisms for the tested materials various analytical techniques are presently employed.

2.3 Microclimate characterization

In order to be able to predict expected service life of the component and its materials from the results of accelerated ageing tests, the degradation factors under service conditions need to be assessed by measurements. If only the dose of a particular environmental stress is important then the distribution or frequency function of a degradation factor is of interest.

For measurement of microclimatic variables relevant in the assessment of durability of the static solar materials studied in Task 27, various kinds of climatic data during outdoor exposure at different test sites are monitored such as global solar irradiation, UV-radiation, surface temperatures, air humidity, precipitation, time of wetness, wind conditions, and atmospheric corrosivity. Such data will be used to predict expected deterioration in performance over time by making use of degradation models developed from results of accelerated tests. Some results from the measurement of microclimatic data are shown in Table 5 and Fig. 3.

Exposure Site with metal coupon orientation **South/90°** - South/45°	**First year metallic mass loss** [g/m^2]		
	Copper	Zinc	Carbon steel
ISE, Freiburg, Germany	**7.2** – 9.5	**2.8** – 4.7	**73** – 83
SP, Borås, Sweden	**4.0**	**2.6**	**43**
SPF, Rapperswil, Switzerland	**4.0** – 5.2	**2.6** – 7.9	**71** – 81

Table 5 Atmospheric corrosivity measured at three test sites for outdoor exposure of solar facade absorbers in the IEA Task 27 study

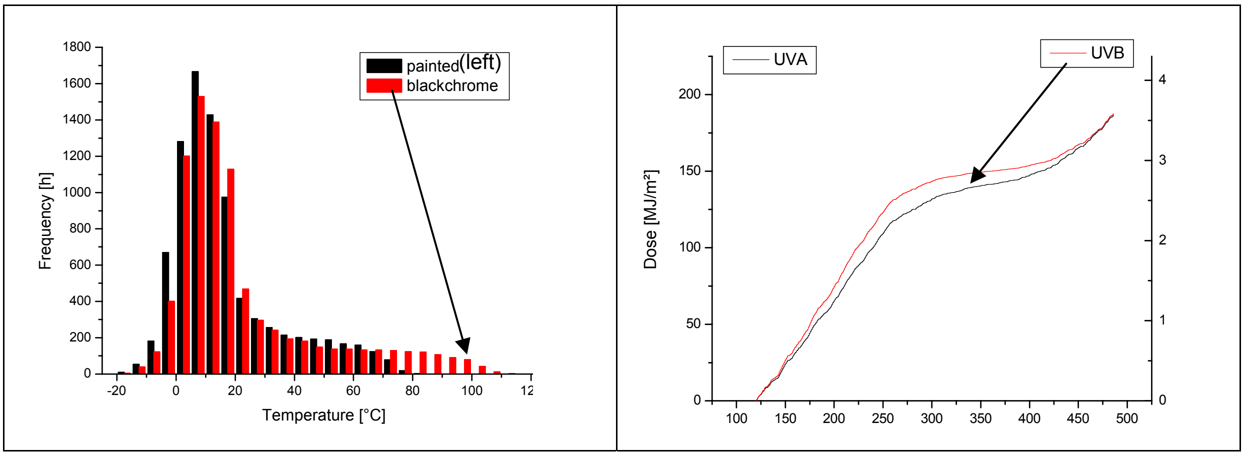
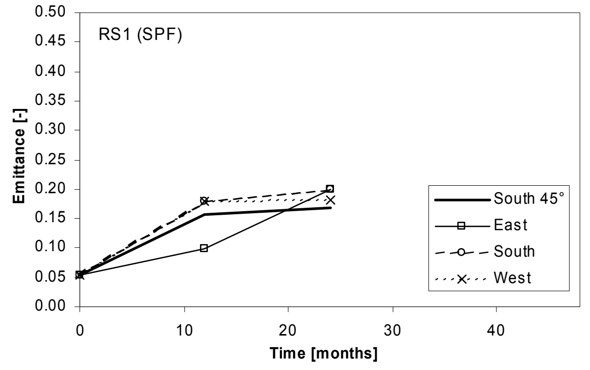


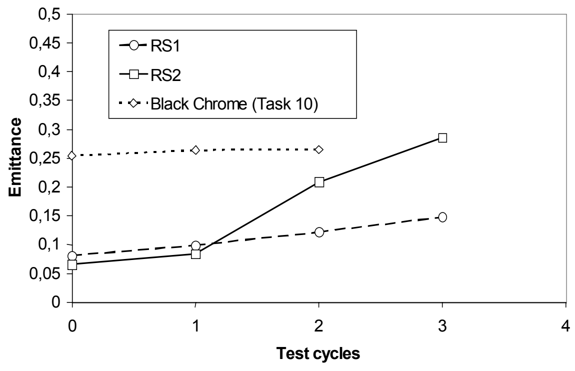
Figure 3 Microclimatic data measured during outdoor exposure of solar facade absorbers at ISE in the IEA Task 27 study. Left diagram: Surface temperature frequency histograms for a black painted and a black chrome absorber; Right diagram: UVA and UVB light doses versus exposure time

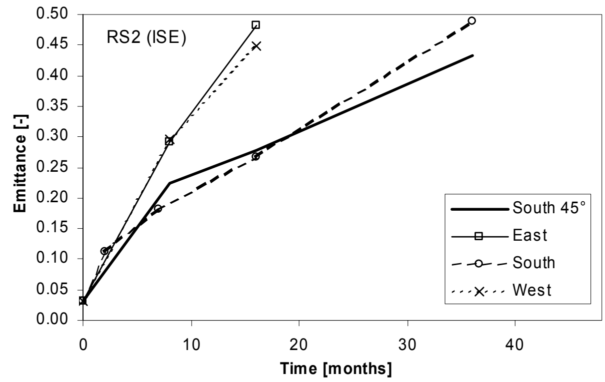
2.4 Service life prediction from results of accelerated testing

TT2-232 Study on Durability and Service Lifetime Prediction of some Static Solar Energy Materials, B. Carlsson et. S. Brunold[2], A. Gombert[3], M. Heck[3], M. Köhl[3], V. Kübler[3], O. Holk[4], G. Jorgensen[5], B. Karlsson[6], M.L. Prates[7], K.Möller[1], M. Brogren[8], A. Roos[8], A. Werner[8], M. Zinzi[9], and M. Ghaleb[10]

In accelerated life testing the sensitivity to the various degradation factors on the overall deterioration of the performance of the component and its materials is quantitatively assessed.







Test	Time	Mass loss of Cu (g/m²)
Accelerated[1]	2 cycles	5.6
Outdoor ISE[2]	1 year	5.2
Outdoor SPF[2]	0.5 year	4.8

[1] Corrosion test according ISO 21207 method A without salt spray exposure
[2] Outdoor exposure with samples faced south and tilted 45°

Figure 6 Change in thermal emittance observed for some reference solar facade absorber materials during outdoor testing and during accelerated corrosion testing. The corrosivity dose in terms of metallic mass loss of copper at an exposure time is also given for the different tests to illustrate that outdoor performance of those absorbers can be predicted by making use of the equivalent corrosivity dose approach.

Mathematical models are then set up to characterize the different degradation mechanisms identified and from the accelerated life test results the parameters of the assumed model for degradation are determined and the service life then estimated.

In Fig. 6 is illustrated how the principle of equivalent corrosivity dose in accelerated corrosion testing can nicely be adopted in the prediction of the long-term outdoor performance of some solar facade absorbers. A prerequisite for this is that the accelerated corrosion test correctly simulates the predominating corrosion mechanism occurring under normal outdoor conditions.

2.5 Validation

The best approach in validating an estimated service life from accelerated testing is to make use the results from the accelerated life tests to predict expected change in material properties or component performance versus service time and then by long-term service tests check whether the predicted change in performance with time is actually observed.

The results of validation tests therefore can be used to revise a predicted service life and form the starting point also for improving the component tested with respect to environmental resistance, if so required. It should be remembered that the main objective of accelerated life testing is to try to identify those failures, which may lead to an unacceptable short service life of a component. In terms of service life, the main question is most often, whether it is likely or not, that the service life is above a certain critical value.

In the case studies of Task 27 outdoor tests at different test sites are performed for measurement of micro-climatic variables and for validating predicted loss in outdoor performance from accelerated test results. Tests are performed by CSTB in Grenoble (France), ENEA in Rome (Italy), INETI in Lisbon (Portugal), ISE in Freiburg (Germany), NREL in Colorado/Florida/Arizona (USA), SP in Boras (Sweden), SPF-HSR in Rapperswil (Switzerland) and Vattenfall in Älvkarleby (Sweden). In Fig. 8 some results from outdoor exposure of antireflective glazing materials are shown to illustrate the effect of soiling.

3. CONCLUSION

The work in IEA Task 27 on durability assessment of static solar energy materials has shown that it is possible to employ a systematic approach in the evaluation of the expected service life of the materials studied. Based on the work performed recommended test procedures will be worked out for qualification of new materials with respect to durability.

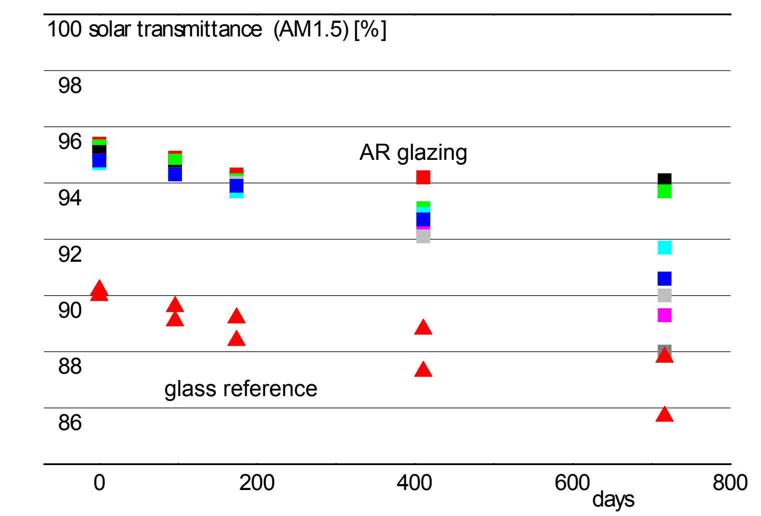


Figure 8 Results from outdoor exposure of antireflective glazing materials performed at SPF-HSR Rapperswil, Switzerland. The decrease in the solar transmittance with time is due to soiling effects, which vary very much with exposure site.

For recommended durability test procedures to be accepted as international standards, it is of utmost importance to demonstrate their relevance for predicting real in-service long-term performance. We think that the work of Task 27 will meet this requirement.

4. REFERENCES

M. Köhl ; New IEA-SHC Task: Performance of Solar Façade Components – Performance, durability and sustainability of advanced windows and solar components for building envelopes, Proceedings Eurosun 2000 Conference, Copenhagen 19-22 June 2000

B. Carlsson, K. Möller, M. Köhl, M. Heck, S. Brunold, U. Frei , J.- C. Marechal, G. Jorgensen; The applicability of accelerated life testing for assessment of service life of solar thermal components, *Solar Energy Materials and Solar Cells* **84**(2004)255-274

R.E. Mc Dermott, R.J. Mikulak, M.R. Beauregard, R. Mikylak, *The basics of FMEA,* Productivity Inc. 1996, ISBN: 0527763209

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Stainless Steel rebar, the choice of a long lasting quality

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ABSTRACT

The civil engineering works and buildings made of reinforced concrete may, in some conditions (e.g. marine environments, when submitted to de-icing salts, chemically aggressive environments, water treatment plants) be subject to important and quick defacement due to corrosion.
This is generated mainly by two phenomena:

- Carbonation which leads to a progressive diminution of the pH inside the concrete up to a threshold under which the carbon steel reinforcement bars lose their passivity
- Penetration of chlorides through the concrete which reaches the carbon steel bars thus becoming corroded

The volume of the rust being bigger than this of the steel, the expansion causes many damages such as scaling or cracking and is prejudicial to aesthetics and security. This generates important costs of maintenance and repairs. It also significantly reduces the life duration of the works. This kind of damages can easily be prevented by the use- total or partial- of stainless steel reinforcement bars which in the same environmental conditions do not corrode. This solution offers many advantages, for instance:

- Low costs of maintenance
- **Increased durability**
- Lower costs of operation
- Better security
- Optimisation of the thickness of the coatings and lighter structures

This solution constitutes an important technological progress. Compared with carbon steels, it does not require any significant extra skills for installation and its cost overrun (low), which depends of course on the proportion of stainless steel rebars which have been used, is very quickly amortized by the economies made on the maintenance costs.

KEYWORDS

Stainless steel, corrosion, costs of maintenance, carbonation, chlorides

1 INTRODUCTION

Corrosion is the source of much defacement of buildings and civil works.
The global cost of corrosion in the occidental countries is estimated by some experts at 3% of the BNP. It is said that, every year, the wet corrosion causes the destruction of 150 millions metric tons of iron and steel, equivalent to about 15% of the annual world production of steel [A. Durupthy *et al. 1996*] .
It is a priority to fight against corrosion. Several solutions already exist in order to avoid or prevent it:

- Increase of the concrete cover
- Improve the tightness of the concrete
- Cathodic protection
- Corrosion inhibitors…

The reinforcement of concrete with stainless steel is a new solution which appears on the market.

2 STAINLESS STEELS

Stainless steel is a generic term of a large family of corrosion resistant alloys (C.R.A.) containing at least 10,5% chromium (European standard EN 10088) and which may contain other alloying elements.

2.1 Stainless steels are corrosion resistant

One of the most important properties of stainless steels is their resistance to corrosion. The resistance of these metallic alloys to the chemical effects of corrosive agents is determined by their ability to protect themselves through the formation of an adherent insoluble film of reaction products that shields the metal substrate from uniform and localised attack, called passive layer; this very thin layer of the order of 1.0 to 2.0 nm, reduces the corrosion rate to negligible levels. The most crucial element in stainless steel is chromium, but other elements such as molybdenum or nickel, contribute to their corrosion resistance.

2.2 The different families of stainless steels

A pure metal appears as an arrangement of atoms into an organised structure.
Basically, the assemblage of atoms of pure iron has two different structures according to the temperature:

- under 910°C the crystalline structure is body centered cube as shown on fig.1, so called ferritic
- between 910°C and 1400°C the structure is face-centered cube, fig.2, so called austenitic
- over 1400°C it becomes again body centered
-

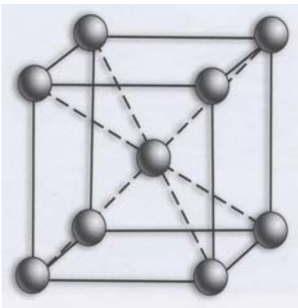

Figure 1. body centered cube structure

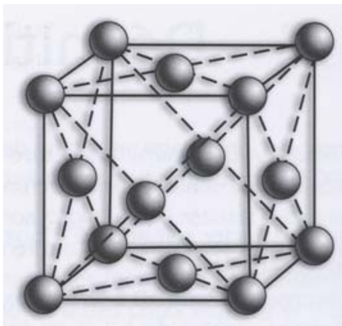

Figure 2. face-centered cube structure

The addition of alloying elements modifies the basic structure of the metal as follows:

- elements such as chromium or molybdenum develop a ferritic structure
- elements such as nickel, manganese, nitrogen develop an austenitic structure
- carbon develop a martensitic structure which is, again, another structure

Therefore, according to their composition, stainless steels can be classified into 4 different families:

- ferritic stainless steels
- austenitic stainless steels
- martensitic stainless steels
- austenitic-ferritic stainless steels, so called duplex

Each family has its own specific properties; for stainless steel concrete reinforcement, the most utilised families are austenitic and austenitic-ferritic (duplex).

2.3 The different grades of stainless steels

Inside each family, there are different grades which have slightly different alloying elements contents. These small differences can generate important differences of the properties of utilisation. Practically, there are more than 100 different grades of stainless steels, which offer the possibility to choose the most appropriate product for a given application.
For reinforcement bars in concrete, there are many possibilities but, in order to simplify, basically only two different grades are recommended depending on the aggressivity of the medium(see further 6.4).

2.4 Properties of stainless steels compared with carbon steels

Other than the ability of being corrosion resistant, stainless steels have also got specific physical and mechanical properties different from those of carbon steels thanks to the presence of alloying elements in their structure.

2.4.1 Physical properties[Table 1.]

	Carbon steels	Austenitic stainless steels	Austenitic-ferritic stainless steels
Linear expansion between 20°C and 100°C ($10^{-6}K^{-1}$)	10	16	13
Thermal conductivity at 20°C ($W.m^{-1}K^{-1}$)	40	15	15
modulus of elasticity at 20°C (GPa)	206	196	200
magnetism	Yes	No	yes

Table 1. comparison of the physical properties of stainless steels with carbon steels

Stainless steels have a poor thermal conductivity and austenitic stainless steels are non magnetic.

2.4.2 Mechanical properties

Mechanical characteristics of stainless steels are generally much higher than those of carbon steels; for instance the yield strength of stainless steels is generally between 500 MPa and 650 MPa, and their elongation at rupture often over 50% in the case of the austenitic stainless steels.

3 THE MAIN REASONS FOR CORROSION IN CONCRETE

Inside the concrete, corrosion of the reinforcement bars may occur in certain conditions: corrosion is mainly due to carbonation and/or chlorides penetration through the concrete cover.

3.1 Carbonation of the concrete

In brief, the combination of carbonation of the concrete (by adsorption of CO_2 with the alkaline species, in particular $Ca(OH)_2$ dissolved inside the aqueous interstitial solution) leads to the formation of $CaCO_3$ following the reaction: $Ca\,(OH)_2 + CO_2 + H_2\,0 \rightarrow Ca\,CO_3 + 2H_2O$. Consequently, as the concentration of $(OH)^-$ decreases, the pH which is originally around 12 to 13, also decreases and reaches a value of about 10 or 9, which leaves the carbon steel unprotected against corrosion. The rate of carbonation depends on several parameters, e.g. humidity of the medium, porosity of the concrete, number of cracks.

3.2 The action of the chlorides

In some specific environments (e.g. close to the seashore or in the neighbourhood of a road which is regularly de-iced) buildings can be exposed to the action of the chlorides, due to the presence of salted water sprays. The chloride ions penetrate through the concrete cover and reach the carbon steel reinforcement which leads to local corrosion.

3.3 The effects of corrosion of reinforced concrete

When they lose their passivity the carbon steel reinforcement bars corrode and, as the volume of the rust is about 6 times bigger than that of the steel, they exert pressure against the concrete which can burst and break in some places and generate defacement. For this reason, corrosion should be avoided.

4 STAINLESS STEELS: A SOLUTION AGAINST CORROSION

There already exist many solutions to protect the works against the corrosion such as cathodic protection, increase of the cover thickness, use of more compact concretes. The use of stainless steel which is a non corroding material for the reinforcement is a new solution in progress.

4.1 Properties of the passive layer of the stainless steels

As seen in section 2.1, stainless steels are resistant against corrosion thanks to the formation on the surface of a very thin passive layer, which has several specific properties:

- it is inert, which means that it protects the metal against corrosion
- it is neutral, which means that the ions cannot migrate from inside the metal through the outside environment; this is a good property for the protection of the environment.
- it self repassivates in the eventuality of a superficial damage; this is a very important property, as it means that whatever may happen, stainless steels remain ever protected.
- it is also very stable: its thickness remains constant.

Nevertheless, in some conditions, elements such as chlorides can break the passive film according to their concentration, the temperature of the medium and the stainless steel grade. It is therefore necessary to have a good understanding of the environmental conditions in which a stainless steel is to be used.

4.2 Stainless steels behaviour in concrete

The type of corrosion which mainly occurs on the reinforcement bars inside concrete is "pitting corrosion". As stainless steels have an excellent resistance to this type of corrosion, they are very suitable for this type of application. It can take many years to evaluate the performance of a stainless steel rebar in real conditions; therefore accelerated tests have been developed in Ugitech laboratories.
The medium used for testing can be considered as particularly severe: carbonated concrete, pH 10, high chlorides level (35 g/l), which is the average Cl^- level of the seawater
The materials which have been tested are: carbon steel – 1.4597- 1.4301 (304)- 1.4404 (316)- 1.4462
Tests carried out in an aqueous solution ($NaHCO_3$ 0,025 M + Na_2CO_3 0,025 M + NaCl 35g/l ; pH 10)
Measurements results are shown in Fig.3 as the pitting potential of the different grades, or the energy to be applied in order to obtain pitting corrosion; the higher the pitting potential is, the better the corrosion resistance.

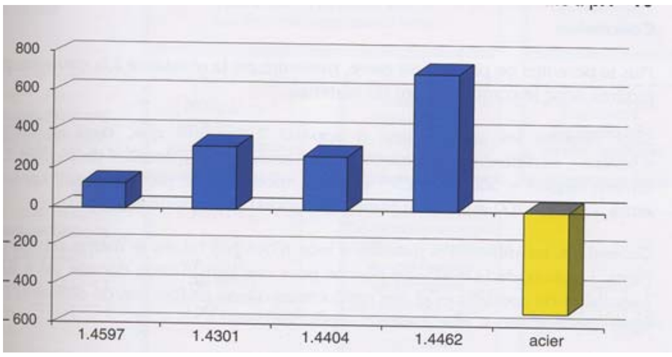

Figure 3 : comparison of the pitting potential (mV/ECS)of different grades of stainless steel and a carbon steel

As for a given medium the performances of the different grades are not the same, the choice of the appropriate grade has to be adapted to the environment. Theoretically, there are many possible grades, but practically, two types of grades can be used for concrete reinforcement bars; previously, it is important to select the kind of works and buildings that should be concerned by reinforcement with stainless steel bars.

5 CIVIL WORKS AND BUILDINGS CONCERNED BY STAINLESS STEEL REINFORCEMENT BARS

5.1 Concrete exposure classes

The European standard EN 206 for the concrete has been published recently; it is now possible to select a specific concrete according to the expected durability and the conditions of outside exposure which are grouped into 6 classes:

- XO no risk of corrosion or attack
- XC corrosion induced by carbonation (XC1 to XC4 according to the degree of humidity)
- XD corrosion induced by chlorides (origin other than marine) (XD1 to XD 3 depending on the degree of humidity)
- XS corrosion induced by seawater chlorides (XS1 not immersed to XS3 totally immersed)
- XF frost, thaw exposed works (XF1 to XF4 depending the use of de-icing salts and the degree of humidity)
- XA chemical attacks (XA1 moderately aggressive to XA 3 very aggressive)

Out of this classification it is possible to draw the classes of exposure in which the use of stainless steel bars should be recommended

5.2 Environments and structures affected by stainless steel reinforced concrete

Among the above mentioned classes, 5 of them correspond to environmental actions in which exists a potential risk for degradation: XA (chemical attacks), XC (carbonation), XD and XS (chlorides) XF (frost /defrosting with de-icing agents). Stainless steels are well adapted for all types of uses of concrete: prefabricated, poured in situ as well as for all structures damaged by corrosive phenomena.
In civil engineering works, the elements which can be constructed using stainless steel reinforcement are:

- Structures - roads, motorways or railways bridges, footbridges, concrete dams, covered trenches-
- Water treatment plants –e.g. basins, storage reservoirs, waste water discharge networks, swimming pools, water supply and distribution networks-
- Road works –safety structures, rainwater purification works, acoustic screens-
- Special foundations – piles, moulded walls, bars in groundwater-
- Underground works – e.g. tunnels, water supply and drainage galleries,-
- Works on maritime sites –basins, jetties, docks, locks-

In buildings:

- Industrial buildings
- Cattle breeding buildings
- Buildings on the seashore
- Connectors for assembling together re-inforcement bars

6 CHARACTERISTICS OF THE STAINLESS STEEL REINFORCEMENT BARS

6.1 Existing standards

The standards describe the stainless steel bars, their shape, designation and main characteristics.
In Europe: UK, BS 6744; Norway, ISO/TC 17/SC 16 N 486 and 487; France, XP35014.
A European standard (EN 100XX) is in progress (launched in November 2004)
European recommendations: concrete society [technical report n°51], UK Highways Agency [Design manual for roads and bridges]; DTU 21 and fascicules 65A.
Existing standard in the USA: ASTM 955M.

6.2 Shape characteristics

There are three types of stainless steel reinforcing bars: smooth (without any engraving), ribbed, indented.
In France the shapes must be in conformity with the EN 10080: 2004 used for carbon steels

6.3 Conventional yield strength

The conventional yield strength range which can be obtained with stainless steel bars is:
From 500 MPa, to 650 MPa, and for one grade and some diameters 800 MPa.

6.4 Selections of the grades according to the concrete exposure class

The existing standards give a rather spread out range of useable grades according to the outside environmental conditions. Practically, and simply, only two grades are recommended :

- Grade EN 1.4462 - UNS 31803 in severely corrosive environments (marine or chemical attacks), classes XS2, XS3, XF4, XA3
- Grade EN 1.4301 - AISI 304 for the rest of the applications, classes XC2, XC3, XC4; XD1, XD2, XD3; XS1; XF1, XF2, XF3; XA1 XA2

7. SPECIFICATIONS AND DESIGN CONDITIONS

The methods and rules for dimensioning frameworks using stainless steels are the same as those applied for steels. Consequently, in Europe, Eurocode 2 is applied for dimensioning stainless steel reinforcement.

7.1 Adherences of stainless steel reinforcements

As mentioned in the EN 10080 standard “ribbed and indented steel products covered by this document are characterized by their surface geometry, by means of which bond with the concrete is achieved” [pr EN 10080:2005 (E), 7.4, Bond strength and surface geometry]
Providing the stainless steel reinforcement re-bars have the geometry as prescribed in the standard, their adherence is guaranteed.

7.2 Cover of reinforcements

The recommendations of Eurocode 2 [pr EN 1992-1-1] regarding covers take in account the durability, and the nature of the outside environment.
The minimum cover is defined as the distance between the surface and the nearest reinforcement. It is the thinnest coating capable of satisfying a standard durability requirement of the structures exposed to class X0, i.e. no risk of corrosion. If longer service life is required, or if the environment is corrosive, according to the exposure classes, an additional cover is prescribed. But in case of stainless steel reinforcement, the value of the additional cover can be reduced to a value “ΔCdur, st”. In Eurocode the recommended value is=0, but it is allowed in the national annexes to give a value # 0.
In the French working group of the national annexe, it has been decided that this value could fully compensate the additional coating in case of stainless steel reinforcement. As long as the purpose of the additional coating is to protect the work against the corrosion of the reinforcements, it is reasonable not to increase the cover if the bars have no risk of corrosion.

7.3 Crack control criteria

Excessive cracking may lead to corrosion. As corrosion does not occur on stainless steel reinforcements, this may allow wider cracks, as long as it remains compatible with the concrete criteria.

7.4 Fundamental design provision

All design provisions related to steel reinforcements are applicable to stainless steel reinforcements.

8. COST OF THE STAINLESS STEEL REINFORCED CONCRETE

There is a strong perception on the market that stainless steel reinforcement is so expensive that it is not even worth calculating the price of any stainless steel reinforced structures. In fact, this approach seems fair if only the difference of bulk price between steel and stainless steel re-bars is considered.
However, nowadays and more and more in the future, any economic approach takes into account many parameters such as the durability, the concept of “return on investment”, the “life cycle cost” including the costs of usage and ownership, and e.g. the possible savings on the construction site. This is the reason

why many works containing stainless steel reinforcements have already been built in many countries (e.g. USA, Canada, Switzerland, Germany, UK). It must be stressed that stainless steel reinforcement can just be a part of the total armouring preferably close to the surface, where corrosion has more risks of occurring.

8.1 Economic analysis on a standard roadbridge with partial or exclusive use of stainless steel

The following example is extracted from the book "Béton armé d'inox" [collection technique CIM Béton n°T81- Cim Béton / I.D.Inox 2004].The structure is a prestressed concrete plate bridge with lateral corbelling as shown on fig.4. The armouring is as shown on fig.5

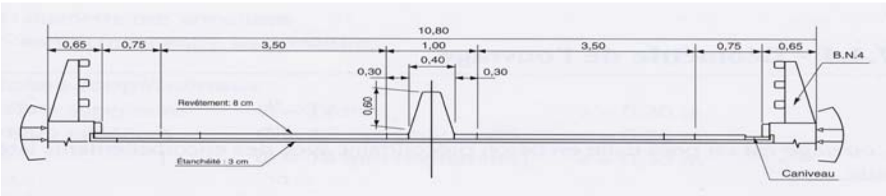

Figure 4. transversal arrangement of the structure

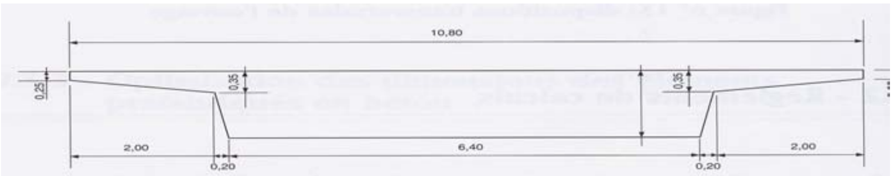

Figure 5. typical cross section of the superstructure

Hypotheses for unit prices (excl. tax) of the materials are:
- Concrete 150 €/kg; formwork 50 €/kg
- Prestressed cables 3 €/kg; prestressed anchorage 750 €/unit
- Steel reinforcement 1,2 €/kg (supply **0,4** €/kg + transport, shaping and installation 0,8 €/kg)
- Stainless steel reinforcement 3,3 €/kg (supply **2.5 €**/kg + transport, shaping & installation 0.8 €/kg)

(Note: the prices are given against the background of highly unfavourable conditions)
The cost of the superstructure has been estimated for several solutions: 100 % carbon steel reinforcement, 10 %, 25 %, 50 % 100 % stainless steel reinforcement.
The comparison of prices of the different solutions is summarized in the table below:

% carbon steel	100 %	90 %	75 %	50 %	0 %
% stainless stel	0%	10%	25%	50%	100%
Total cost €	102 980	106 130	110 750	118 520	134 060
Difference %		3 %	7.5 %	15 %	30 %

Table 2 : comparison of cost of the superstructure in different solutions of reinforcement

Conclusion: in many cases 10 % of stainless steel reinforcements is enough to minimise the risk of corrosion of the work and it generates an ***additional cost of 3 % only.***

8.2 Optimisation of the quantities of different materials

Firstly, as seen above (6.3), as the mechanical characteristics of the stainless steels are currently 500, 650 or even 800 MPa, it is possible to use a significantly lower quantity of armouring, which reduces the additional price of the above table due to the use of stainless steel.
Secondly, according to the recommendations of the French national annexe of Eurocode 2, when stainless steel reinforcement is used it is no more necessary to increase the concrete cover in order to protect the work against corrosion. This allows optimising considerably the volumes of concrete and also reduces the additional prices of stainless steel of the above table. And by the way this represents an excellent opportunity for designing lighter, hence more aesthetic, structures. The other national annexes in Europe are in progress and they may refer to the recommendations of the draft of the French national annexe.
Of course the same reduction of cover in case of the use of stainless steels is also possible with the prefabricated concrete elements.
As a ***conclusion*** if the economies made on the total amount of concrete and armouring, without forgetting associated expenses such as transport and energy, are well taken in account, the additional prices shown for stainless steel in table 2 should be reduced.

8.3 Other economical advantages of the stainless steel reinforcements

The use of stainless steel reinforcements also presents, on the economical point, several other advantages than those above mentioned such as:

- Reduction of the costs of maintenance (direct: costs of repairs and indirect: losses of exploitation) due to the much lower risk of defacement
- Increase of the durability of the constructions, which means that a same amount of invested money will last longer
- The safety being much improved, the frequency of the inspections will be lower

9. OTHER ADVANTAGES OF THE STAINLESS STEEL REINFORCEMENT

Some other characteristics and/or advantages of stainless steel reinforcements should be mentioned:

- Possibility of non magnetism; this may have applications for instance in the hospitals (MRI)
- Better resistance in case of earthquakes thanks to the high capacity of energy absorption of the stainless steels
- Low thermal conductivity; important factor to take in account when used as thermal bridges breakers in the buildings
- And, last but not least, an evident important contribution to the aesthetic of the building, thanks to the absence of defacement, and the possibility to design lighter structures in the corrosive environments.

10. CONCLUSION

The use of stainless steel for reinforcement is a new and very efficient solution to protect the constructions against the risks of corrosion.
If the cost of the bulk metal is much higher than this of carbon steel, in the end it appears that in the case of a partial use of stainless steel reinforcement, the cost increase is moderate and can be significantly reduced close to 0, through many other savings such as the amount of concrete or armouring.
In balance to this eventual over cost appear many other advantages such as very low costs of maintenance, very competitive life cycle cost, aesthetic and **durability.**

11. REFERENCES

Durupthy, A., Casalot, A., & Jaubert, A., *Chimie des matériaux inorganiques,* H Prépa 2è année PC-PC*

Concrete Society, Technical report n°51, 1998 *Guidance of the use of stainless steel reinforcement*

Pr EN 10080:2005 ; pr EN 1992-1-1 (Eurocode 2), 2005, 1998, *European standards*

EN 10088 parts 1, 2 and 3, 2001, *stainless steels European standards*

CIM Béton and I.D.Inox, Collection Technique CIM Béton T.81, 2004, *Béton armé d'inox.*

OVE ARUP & PARTNERS – Research & development, Stainless Steel Reinforcement for Concrete, British Stainless Steel Development Association, Apr.2003, *The use of stainless steel reinforcement in bridges*

D.B.McDonald, M.R. Sherman, D.W. Pfeifer, Y.P Virmani, NiDI Nikel Development Institute, May 1995, *Stainless steel reinforcing as corrosion protection*

Lacombe (P.), Barroux (B.) et Berranger (G.), avec le concours d'Ugine, Editeurs Scientifiques, Les Editions de la Physique, 1990, *Les Aciers Inoxydables.*

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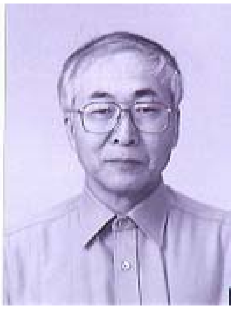
Rapid Measurement of Moisture Diffusive Material Properties



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ABSTRACT

Moisture migration in building component can accelerate the deterioration of the related component. To avoid such problem the moisture migration should be predicted in the process of the design of construction. Heat and moisture simultaneous transfer model can predict the moisture movement if the moisture diffusive properties of every material. The major diffusive properties are the moisture sorption isotherm and the moisture diffusivity. Under the isothermal condition we can predict the moisture content distribution using the heat moisture program if both of the two diffusive properties are obtained. The only problem of the heat and moisture simultaneous transfer model is that it takes very long time to measure the moisture diffusive material properties. This paper is to mention how to shorten the time for measurement of moisture diffusive material properties.
When we measure the sorption isotherm of a material, we should initialize the specimen of this material in homogeneous conditions of moisture content and of temperature. This process is already difficult to achieve perfectly particularly when the equipment for initialization is different from the measurement equipment. We propose to execute the initialization and the measurement in the same measurement chamber in a test equipment. Our relative humidity generator can create any relative humidity from 0% to 100% with high accuracy in an isothermal condition of high accuracy and change its relative humidity in a quasi instantaneous time. We initialize the specimen in a test chamber in this equipment and measure the sorption isotherm of this specimen without taking out from this chamber. We know already that the mass of the specimen installed in a measurement chamber, whose relative humidity is kept constant but different from the initialization condition, varies according to its moisture content until this reaches an asymptotic value in an exponential way. It takes so much time to reach the final equilibrium condition. Our measurement method we propose here is to abandon its accomplishment but to assume the asymptotic value. If the shape of the specimen can be supposed to be a sphere, we can assume the equilibrium condition with high precision using the McBain type curve fit program. And from the equilibrium values at different relative humidity we can get the sorption isotherm as an approximation curve of a function of relative humidity. Using the same data obtained through the measurement of sorption isotherm of the specimen, we can obtain its moisture diffusivity.

KEYWORDS
Dynamic Measurement method, Sorption Isotherm, Moisture Diffusivity, Heat and moisture simultaneous transfer model.

1 INTRODUCTION

It is rare to find a deteriration process of building and building component which progresses without the presence of moisture. The prediction of moisture content in the building component can be a very useful tool for judgement of deterioration of the building. Moisture transfer in the building component is always accompanied with heat transfer. Thus Heat and Moisture Simultaneous Transfer Model is quite useful for the prediction of moisture condition in the building component. Heat and Moisture Simultaneous Transfer Model has already a long history since the first developments in 1956 by Luikov, in 1957 by Philip and De Vries. And today there exists several commercialized simulation programs. Nevertheless, its application in the buiding conception is quite limited. One of the reasons lies on the fact that it is quite difficult to obtain the material properties necessary for the use of the model. It takes a very long time to measure the material properties related to the moisture migration. They are moisture sorption isotherm and moisture diffusivity of the material. This paper is to mention how to shorten the time for measurement of moisture diffusive material properties.

2 HEAT AND MOISTURE SIMULTANEOUS TRANSFER MDODEL

There exist several models for heat and moisture simultaneous transfer. The major difference between the models lies on the choice of driving force of moisture transport. The gradient of temperature is treated as one of the driving forces of moisture in any model. As for another force for moisture transport each model takes different parameter according to the application field of the model like the gradient of moisture content, of relative humidity, of water capillarly pressure, of osmotic pressure or of the chemical potential of water. These parameters are mutually convertible if thermal dynamic theorem are respected.

Here we can show a model based on the gradient of chemical potential derived from the Gibbs thermal dynamics.

Equation of conservation of water:

$$\rho_w \frac{\partial \phi}{\partial \mu}\frac{\partial \mu}{\partial t} = \frac{\partial}{\partial x}\left[D_\mu \frac{\partial \mu}{\partial x} + D_T \frac{\partial T}{\partial x}\right] \qquad \text{(Eq-1)}$$

Equation of conservation of heat:

$$c\rho \frac{\partial T}{\partial t} = \frac{\partial}{\partial x}\left[(\lambda + \gamma D_{TG})\frac{\partial T}{\partial x} + \gamma D_{\mu G}\frac{\partial \mu}{\partial x}\right] \qquad \text{(Eq-2)}$$

Here the symbols are difined as follows:

ρ_w: density of water, $\mu = R_v T \ln\left(\frac{p_v}{p_s}\right)$, R_v: gas constant of water, T: temperature, p_v: vapor pressure, p_s: saturation vapor pressure, $\phi = F(h,T) \cong F(h)$, $h = \frac{p_v}{p_s}$, *t*: time, *x*: distance, *D*:moisture conductivity divided into several components defined by the suffix, *c*: specific heat, ρ: density, λ: thermal conductivity, γ: phase change heat of water between gas and liquid.

3 RELATIVE HUMIDITY GENERATOR

In the traditional method the relative humidity is controlled with different salt solutions. To change the relative humidity the specimen should be transferred from a test chamber to another. In the process of this travel the specimen can be affected by the external condition. Moreover in order to keep the relative humidity the concentration of salt solution should be kept constant and it is already difficult to keep it constant without opening the measurement chamber. In order to reduce such noises we have developed a relative humidity generator whose relative humidity is obtained by mixing of dried air and moisture saturated air which are the divided flows of dried air.

The relative humidity is decided by the portion of mixture of the two airs. As the dried air is not completely dry, this equipment has an inevitable error and the relative humidity of 0% in this equipment is 0.5%. Otherwise the relative humidity has an error of ±1% and the temperature has an error of ±0.5°C. As the relative humidity of the measurement chamber can be changed easily with valve operation, the relative humidity of the chamber can change from any one to another in a quasi instantaneous time.

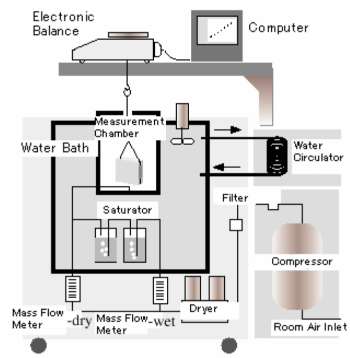

Figure 1: Relative Humidity Generator

4 MEASUREMENT OF MOISTURE DIFFUSIVE MATERIAL PROPERTIES

The measurement methods for the moisture sorption isotherm and the moisture diffusivity were studied using this relative humidity generator.

4.1 Initialization of specimen

Before measurement we should prepare the specimen with homogeneous temperature and relative humidity distributions. As our relative humidity generator can create an isothermal environment with relative humidity from 0% to 100%, we decided to initialize the specimen in the measurement chamber of this equipment. The specimen are set in the chamber of the requested relative humidity for a sufficiently long time until the electronic balance shows that they are in an equilibrium stage. When they are confirmed in an equilbrium stage the measurement can begin without taking out the specimen from the chamber.

4.2 Hypothesis of exponential approach to the asymptotic value in the sorption process

In the traditional way the sorption isotherm of a material is obtained in a measurement chamber set at a certain relative humidity under the isothermal condition until the mass of specimen reach the equilibrium stage. The curve of mass change in the sorption process can be considered as an exponential one and it approaches to an asymptote as described below. By using the data of mass evolution in the sorption process the McBain type curvefit model can determine the asymptote.

$$m(t) = a\left[1 - b\exp(-ct)\right] \qquad \text{(Eq-3)}$$

where m(t): total absorbed or desorbed mass until the time t, a: asymptote, b: specimen shape coefficient and c: diffusivity coefficient. They are determined by curvefit proram.

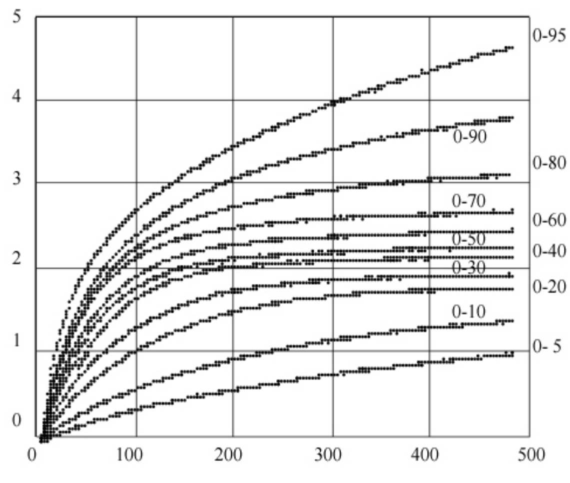

Figure 2 Mass evolution in Sorption process

4.2 Measurement of Moisture Sorption Isotherm

The equilibrium moisture content at a certain relative humidity can be obtained if the specimen is set in a chamber whose relative humidity and temperature are controlled at a constant level. As the sorption isotherm shows difference between the absorption process and desorption process if the initial condition of the specimen is set at a relative humidity lower than the one where the equilibrium moisture content is to be measured, the measured equilibrium moisture content is for the sorption process. If the relative humidity for initial condition is closer to the one for equilbrium condition, the time necessary for approaching to the equilibrium stage is shorter. In the dynamic measurement method it is not necessary to wait until the moisture content reach the equilibrium condition but it is recommended to wait until the measurement approaches to the final stage.
If the equilibrium moisture content is obtained at every 10% of relative humidity from 0% to 100%, the sorption isotherm curve can be approximated. The equation can be described in several forms as shown below:

$$\theta = Ah^3 - Bh^2 + Ch + D \quad \text{(Eq-4)}$$

$$\theta = \theta_s h^a \exp\left[b\left(1-h^c\right)\right] \quad \text{(Eq-5)}$$

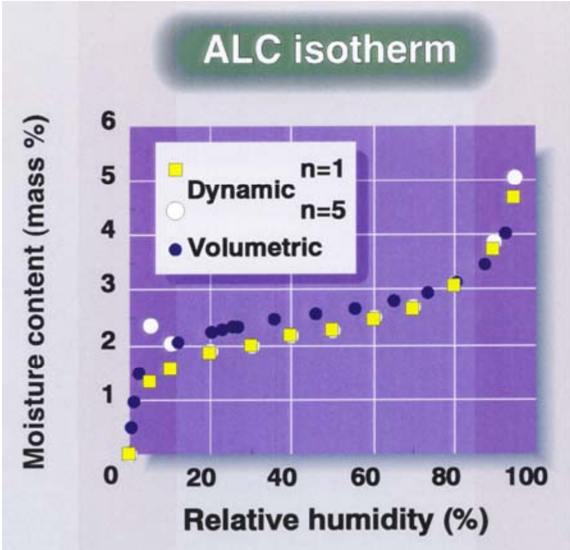

Figure 3 Sorption isotherm of AAC

The result of dynamic measurement method is compared with that of traditional volumetric method. Dynamic n=5 signifies that the equilibrium moisture content is obtained with the equation described below:

$$m(t) = a\begin{bmatrix} 1 - b\exp(-ct) - 0.25b\exp(-4ct) \\ -0.1b\exp(-9ct) - 0.0625b\exp(-16ct) \\ -0.04b\exp(-25ct) \end{bmatrix}$$

4.3 Direct Acquisition of Moisture Diffusivity

When the specimen is supposed to be a sphere, the total moisture transferred from the surface to the spheric body until the time t, m(t) can be described as follows:

$$m(t) = m_e\left[1 - \frac{6}{\pi^2}\sum_{n=1}^{\infty}\frac{1}{n^2}\exp\left(-n^2kt\right)\right] \quad \text{(Eq-7)}$$

where m_e: total moisture transferred from the surface to the specimen until the infinite time, $k=D_{\theta V}\pi^2/R^2$

Supposing that n=1, this equation becomes exactly the same as the McBain type curvefit model described as Eq-3. So that the $D_{\theta v}$ can be obtained by the coefficient c of the curvefit as follows:

$$D_{\theta v} = \frac{R^2}{\pi^2}c \quad \text{(Eq-8)}$$

This $D_{\theta v}$ is the mean value of the moisture diffusivity in the zone where measurement was executed. In order to obtain an accurate result, the measurement condition should be arranged in a way that the zone of moisture content is narrow enough.

4.4 Acquisition of Moisture Diffusivity by reverse calculation

When we have obtained the sorption isotherm, by using the same measurement data we can predict the evolution of moisture content distribution of the specimen with a certain supposition of moisture diffusivity. By comparing the sum total of moisture content distribution correspond to the measured mass, we can consider that the supposed moisture diffusivity was appropriate. Supposing that moisture diffusivity is a function of moisture content described as follows:

$$D_v = A\left(\theta^{-B} - 1.0\right) + C\left(D^{\theta} - 1.0\right) + E \quad \text{(Eq-6)}$$

where D_v: moisture diffusivity, θ: moisture content, the coefficients A, B, C, D, E can be determined through the approximation. When the specimen is a sphere, the distribution of moisture content can be supposed as the following figure:

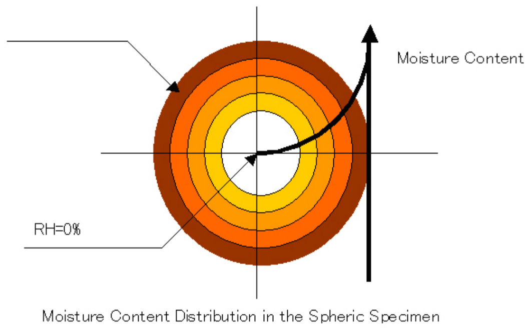

Figure 4 Moisture content distribution in a spheric specimen

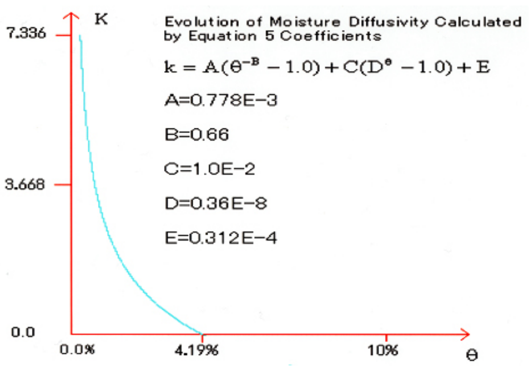

Figure 5 Moisture diffusivity calculated backwardly

The data used for this calculation were the ones obtained through the process of absorption in the chamber set with the relative humidity of 95% for a specimen of AAC initialized with the relative humidity 0%. It seems that the moisture diffusivity obtained through this process is that for quite a dry zone, whereas the moisture diffusivity varies widely.

5 CONCLUSION

From several tests we want to conclude as follows:
The sorption isotherm can be measured in a rapid way with high accuracy. If the difference of initial condition of specimen and the target condition for the equilibrium moisture content is smaller, the accuracy in measurement of the sorption isotherm will include.
As for the moisture diffusvity also, the narrower measurement condition will bring the better results. If the difference between the initial condition for the specimen and the target condition for the sorption test is for example 10% of relative humidity, the mean value of moisture diffusivity for dry zone to high moisture content zone can be easily obtained with higher accuracy. With these values we can obtain the moisture diffusivity as a function of moisture content and using this we can correct the diffusivity backwardly through the simulation of moisture content distribution.
If you need a more precise measurement, as the specimen cannot be dried completely at the condition of 0% of this equipment, it may be better to obtain it by changing the sorption condition from for example 10% of relative humidity to 2% of relative humidity and obtaining a curve of the mass of specimen approaching to the completely dried condition and by using the curvefit procedure described below the dried mass can be approximated with a higher accuracy. For the mass at the condition of 100% in relative humidity, the same procedure as that for 0% can be obtained.

Finally the test method can be modified to the step up procedure for the sorption process and step down procedure for the desorption process. These tests should be executed in the near future.

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7 REFERENCES

Watanabe, K., Ashie, Y. and Shligersky, I., Towards practical simulation of hygrothermal behavior for building components, Part 3 Transient Method for Quick Measurement of Sorption Isotherm, Prep. Annual Meeting of Soc. Heating Air-conditioning and Sanitary Engineering of Japan, Hiroshima, 29-32, 1995

Watanabe, K. and Shligersky, I. Rapid measurement method for definition of moisture related properties in building materials, ACI Annual Meeting, Montreal, 1995

Watanabe, K. and Shligersky, I. Innovative method for the evaluation of the moisture impact on structure durability, 20th Conf. Our World in Concr. & Struct., Singapore, 1995

Watanabe, K. and Tada, S. Rapid measurement methods for moisture transfer properties, Proc. Int. Conf. COST Action E8 Wood Mechanics, Tech. Univ. Denmark, Copenhagen, 51-60,1997

Watanabe, K., Sun, X., Tada, S. Characterization of material properties related to moisture migration in porous media, Advances in Composite Materials and Structures VII (Structures and Materials,) Computational Mechanics; ISBN: 1853128252; (2000/08/22) 115-124, 2000

Tada, S., Watanabe, K., A dynamic method determining equilibrium moisture content of porous materials, 4th International Symposium on Humidity and Moisture, Taipei, 318-324, 2002.

Durability of fly ash based Geopolymer concrete against sulphuric acid attack

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ABSTRACT

In spite of a long-term recognition of the problem of sulphuric acid corrosion in concrete sewer pipes, this issue has not been satisfactorily resolved. Geopolymer binders have been reported as being acid resistant and thus are a promising and alternative binders for sewer pipe manufacture. This paper presents experimental data on the durability of fly ash based Geopolymer concretes exposed to 10% sulphuric acid solutions for up to 8 weeks. A class F fly ash based Geopolymer concrete was initially cured for 24 hours at either 23˚C or 70˚C. The compressive strength of 50-mm cubes at an age of 28 days ranged from 53MPa to 62MPa. After immersion in a 10% sulphuric acid having a fixed ratio of acid volume to specimen surface area of 8 ml/cm^2, samples were tested at 7, 28, and 56 days. The mass loss, compressive strength reduction, and the residual alkalinity were determined on the basis of modified ASTM C267 tests. The results confirmed that Geopolymer concrete is highly resistant to sulphuric acid in terms of a very low mass loss, less than 3%. Moreover, Geopolymer cubes were structurally intact and still had substantial load capacity even though the entire section had been neutralized by sulphuric acid.

KEYWORDS

Geopolymer; fly ash; acid resistance; durability, concrete pipe

1 INTRODUCTION

In spite of a long-term recognition of the problem of sulphuric acid corrosion in concrete sewer pipes, this issue has not been satisfactorily resolved. A research looked at ways of enhancing the acid resistance of Portland Cement (PC) based concretes, using the partial replacement of Portland cement by supplementary materials, the use of epoxy modified binders, and the use of limestone as a sacrificial aggregate [Song et al 2003]. The acid attack in terms of mass loss was reduced, however, even the improved concretes lost significant mass with immersion time. Sulphuric acid resistant binders are still required to enhance the long-term performance of concrete in sulphuric acid corrosion environments.

Sulphur concrete is sulphuric acid resistant. However, weighing the advantages and limitations of sulphur concrete based on the available published data, Malhotra [1988] emphasised that the indiscriminate use of sulphur as a binder for concrete cannot be recommended.

Geopolymer binders might be a promising alternative in the development of acid resistant concrete. Since Geopolymers are a novel binder that relies on alumina-silicate rather than calcium silicate hydrate bonds for structural integrity, they have been reported as being acid resistant. Davidovits et al [1990] found that metakaolin based Geopolymer has very low mass loss when samples were immersed in 5% sulphuric acid solutions for 4 weeks.

Class F fly ash is a cost effective feedstock for Geopolymer concrete. Fly ash based Geopolymer concrete can set at ambient (23˚C) and high temperatures (70˚C). Rostami and Brendley [2003] tested the acid resistance of alkali fly ash concrete (cured at 40-90˚C) in terms of mass loss. Beyond the measure of mass change, there is limited experimental data to provide insight into the sulphuric acid resistance of fly ash based Geopolymer concretes.

This paper reports experimental data on the response of Alkaline Activated Fly ash based Geopolymer (AAFG) concrete against 10% sulphuric acid solutions for up to 56 days, in terms of visual inspection, mass change, and residual compressive strength and alkalinity.

2 Fly ash based Geopolymer concretes and test procedures

2.1 Fly ash based Geopolymer concretes

Two class F fly ashes were activated by alkaline activators to form Geopolymer gel, which binds the silica sand and latite aggregate (< 10mm) to make Geopolymer concrete. The chemical composition of the fly ashes and alkaline activators is given in Table 1, whereas the mix design is listed in Table 2.

Oxide	Weight (%)				
	Fly ash 1	Fly ash 2	GP cement	Na_2SiO_3	NaOH
SiO_2	67.1	51.3	20.3	30.6	--
Al_2O_3	23.6	32.6	4.6	--	--
Fe_2O_3	3.70	11.5	4.5	--	--
CaO	0.80	2.50	65.1	--	--
Na_2O	0.60	0.20	0.04	9.50	30.1
K_2O	1.60	0.30	0.5	--	--
H_2O	--	--	--	59.9	69.9

Table 1: Chemical composition of starting materials

Two grades of AAFG concretes were prepared for this investigation. G54 represents a Geopolymer concrete synthesised at high temperature (12 hours at 70˚C) whereas G71 was achieved at ambient temperature (24 hours at 23˚C). The nominal compressive strength of 50-mm cubes at an age of 28 days is 62 MPa for G54 and 53 MPa for G71.

The control mix PC55 was made using type GP cement (Table 1) with a water/cement ratio of 0.35 and used silica sand and basalt aggregate (Table 2). The curing condition is as same as G54. Its 28 day compressive strength was 65.1 MPa, having a similar strength grade as Geopolymer G54.

Material	Fly ash 1	Fly ash 2	NaOH	Na_2SiO_3	Sand	Latite
% by mass	13.6	4.7	3.4	4.1	26.3	47.9
	Type GP 535Kg/m^3, water/cement = 0.35				24.0	44.4

Table 2: Mix design of concretes

2.2 Acid resistance testing

At an age of 28 days, 15 AAFG concrete cubes from each mix were immersed in 10% sulphuric acid solution based on a modified ASTM C267 test. Three cubes were weighed for mass change, nine were crushed for residual compressive strength after being immersed in acid for 7, 28 and 56 days, and the remaining three were used for long-term visual observation. Three cubes from PC55 were used to measure mass change. In addition, at the end of the acid exposure period, the cubes that were used to assess mass loss were split to examine their residual alkalinity.

A 10% (by mass) sulphuric acid solution was directly diluted from 98% concentrated sulphuric acid with tap water. The 10% sulphuric acid does not represent the actual service condition encountered in sewer pipes, but such a concentration of acid has been used by the Los Angles County for 15 years to test the sulphuric acid resistance of products [Redner 1998]. The use of a 10% sulphuric acid environment provides accelerated experimental data within 8 weeks. The ratio of the sulphuric acid volume to specimen exposure area was fixed at 8 ml/cm^2. The acid concentration was monitored via titration and refreshed weekly.

3 Experiment results and discussion

3.1 Visual inspection

As can be seen in Fig. 1, the binder in the normal PC55 concrete shows significant degradation the aggregate becoming exposed after only 4 weeks in 10% sulphuric acid. By contrast, Geopolymer concrete cubes, G71 and G54, remained structurally intact in the same acidic environments after 56 days, though some very fine localised cracks were observed.

Figure 1. Appearance of concrete specimens expored in 10% sulphuric acid (Left: PC55 for 28 days, right: AAFG for 56 days)

3.2 Mass change

The mass change (negative means mass loss) with immersion time is shown in Fig. 2. The mass change was calculated according to ASTM C267. AAFG concretes, both G54 and G71, have very low mass loss, less than 3%. The PC55 concrete, on the contrary, experienced significant deterioration, losing up to 41%, of its mass within 4 weeks.

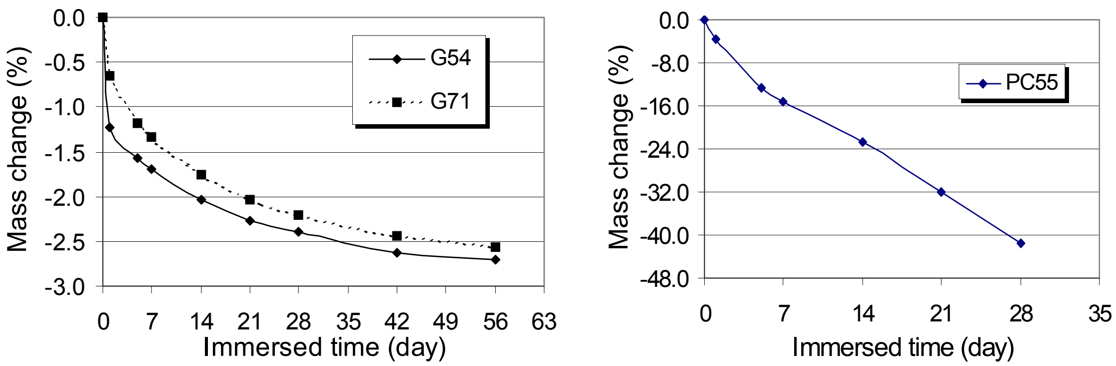

Figure 2. Mass change in 10% sulphuric acid

3.3 Compressive strength change

Since the AAFG cubes were structurally intact after acid immersion, they were crushed to identify the influence of acid attack on strength change. Nine cubes in three groups were crushed after being immersed in acid for 7, 28 and 56 days. Fig. 3 shows the reduction of compressive strength of AAFG concretes as a function of immersion time. It should be stressed that the strength values at 56 days in Fig. 3 were calculated by the ultimate failure load and the measured section area. The influence of slight cracking has not been considered as it is difficult to quantify.

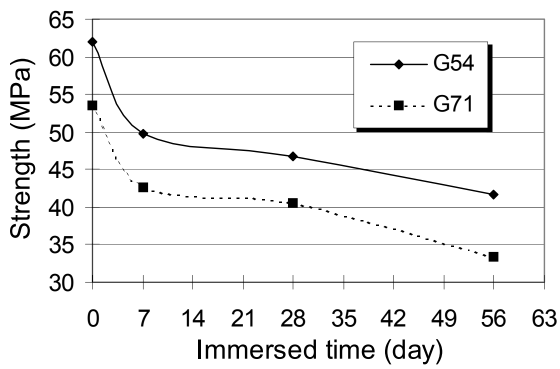

Figure 3. Compressive strength change of AAFG concretes in 10% sulphuric acid

3.4 Residual alkalinity

The residual alkalinity of concrete cubes after sulphuric acid attack was roughly determined by spraying 1% phenolphthalein on the freshly fractured surface [Rendell and Jauberthie 1999]. As shown in Fig. 4, AAFG concretes showed colourless while PC55 was still magenta indicating that the concrete in the centre of the specimen still has a pH above 9.

The pH values of AAFG concrete G54 and G71 were further investigated by powder method [Pavlik 1994]. Two powder samples were collected and identified by their location as the edge and the centre, the zones being separated by the brown line on the cross section (Fig. 4). The pH values of hardened Geopolymer concretes before and after acid attack are presented in Table 3. After acid attack for 56 day, the entire AAFG concrete cubes became acidic with pH value as low as 3.

Figure 4. Residual alkalinity indicated by the phenolphthalein (From left to right: PC55, G71, G54)

Sample	Before acid	After acid -- edge	After acid -- centre
G54	11.2	3.1	6.1
G71	11.0	2.9	8.0

Table 3: pH values of AAFG concretes before and after acid attack

3.5 Discussion the occurrence of fine cracks

The occurrence of very fine cracks on Geopolymer cubes in Fig. 1 was further investigated. It should be stressed that the fine cracks were not evenly distributed on the surface, they appeared only in local areas. As the exposure time progressed (e.g. 112 days), some white reaction products were found within cracks on one of three cubes used for long term exposure. The white products were observed under a Scanning Electron Microscopy (SEM -- Hitachi S4500) coupled with an Oxford Energy Dispersive Spectrometer (EDS). The crystals were observed to have a hexagonal section (Fig 5), which is similar to the SEM image taken by Rendell and Jauberthie [1999]. The EDS data clearly show the dominant elements of calcium and sulphur and hence the white products are gypsum.

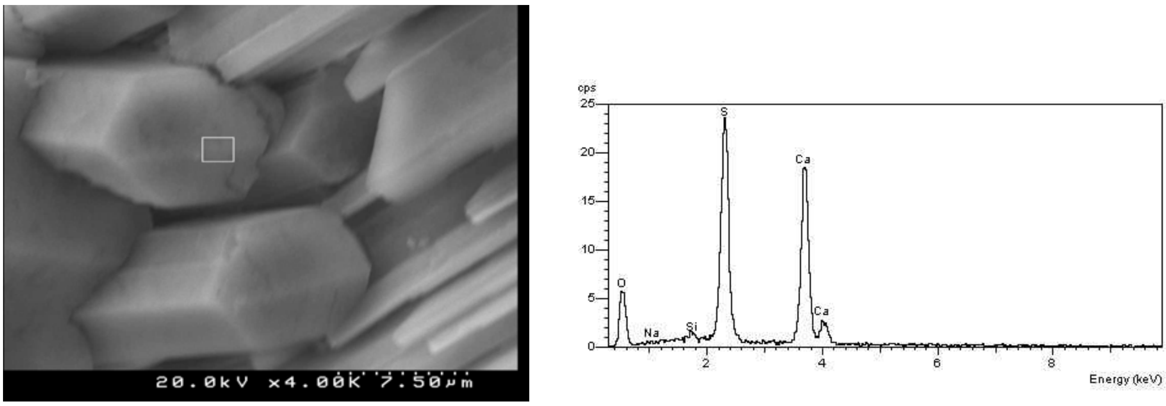

Figure 5: SEM of gypsum crystal and EDS data

The expansion of gypsum has been long known in sulphuric acid attack on PC concrete [Rendell and Jauberthie 1999], however the question remained where does the calcium come from? The class F fly ashes contain very low lime (Table 1). The solubility of sand and latite was determined by dissolving 50-g dry sample in 100-g 30% nitric acid solution for five hours. It is clear in Table 4 that the latite aggregates have a slightly higher solubility that the sand.

Though the solubility of latite aggregates is low, when they were examined carefully, a few pieces were found with a very slight white deposit. Several pieces of such aggregate were picked out to cast Geopolymer concrete Cr-3 whereas Geopolymer mortar Cr-2, without any latite aggregate, was made as the control. After exposure in 10% sulphuric acid for 70 days, the comparative results were significant (Fig. 6). Geopolymer mortar Cr-2 had no cracks whereas the Geopolymer concrete with latite aggregate once again exhibited visual cracking (Fig. 6, right). Clearly then, the cause of cracking, as shown in both Fig. 1 and Fig. 6, is attributed to the contamination the basalt aggregate, where the white stains reacted with sulphuric acid and formed gypsum. The volume expansion of gypsum caused the localised cracks. This test confirmed that the Geopolymer mortar did not swell at all under 10% sulphuric acid solution. Therefore, the selection of aggregate is critical in developing sulphuric acid resistant Geopolymer concretes.

Name	Fine sand	Coarse sand	Latite
By mass (%)	0.9	0.6	2.6

Table 4. Solubility of sand and Latite

Figure 6: Comparison of Geopolymer mortar (left) with concrete (right)

3.6 Discussion the sulphuric acid resistance of Geopolymer concretes

The appearance of the exposed samples (Fig. 1) clearly indicated that AAFG concrete is durable (regardless of the fine cracks) in 10% sulphuric acid up to 56 days. In the case of PC concrete, the hydration compounds were neutralised by sulphuric acid and gradually the binder disintegrated, thus exposing the aggregates.

The mass change agrees with the visual appearance in Fig. 1. The very low mass loss of Geopolymer concretes in this paper is consistent with the findings of Davidovits [1990] and Rostami and Brendley [2003]. Moreover, the trend of mass loss becomes essentially constant at longer exposure time. This indicates that the Geopolymer concretes presented in this paper are sulphuric acid resistant as they stabilized without further mass change.

The residual compressive strength has not been previously used to evaluate the acid resistance of PC concrete because of the rough surface and the exposed aggregate after acid immersion. In this investigation, however, Geopolymer concrete remains structurally intact. The compressive strength was used in this research to evaluate the impact of acid attack on mechanical performance. Although the strength reduction (Fig. 3) was significant within the first week of immersion, this trend then became stable with residual strength up to 33 ~ 42 MPa after 56 days acid exposure. The strength loss was measured in the range of 32 ~ 37%. The residual load capacity indicates that some bonds still exist even the entire section was neutralized by acid. On the other hand, however, the acidity presents a challenge for the use of steel reinforcement under acidic conditions.

In addition, it is very interesting to compare the acid resistance between G54 and G71. There is a significant difference in the 28 days strength development. As expected, G54 has higher compressive strength than G71 due to the effect of higher temperature curing. However, both of them have a very similar trend in resisting sulphuric acid attack, in terms of mass change (Fig. 2), compressive strength reduction (Fig. 3), and the residual alkalinity (Fig. 4). Therefore AAFG concretes are acid resistant regardless of curing conditions. It also seems that Geopolymer concretes have the potential to be used in the production of precast sewer pipes (high temperature curing) as well as in the repair of corroded pipes (ambient curing).

4 Conclusions

Based on above experimental data and discussion, the following conclusions can be drawn:

AAFG binders were found to exhibit much lower mass change than PC concretes. Moreover, Geopolymer cubes were structurally intact and still had substantial load capacity even though the entire section had been neutralized by sulphuric acid. However, steel reinforcement cannot be used in such low pH environments. Hence either alternate reinforcement needs to be used or the permeability of Geopolymer materials has to be substantially improved.

AAFG binders have high sulphuric acid resistance regardless of the curing conditions, at ambient condition (23˚C) or high temperature (70˚C). They have very similar degradation trends.

The requirement for materials selection, especially the presence of calcium, is critical in developing sulphuric acid resistant Geopolymer concrete. The high purity siliceous aggregates is strongly recommended.

5 REFERENCES

Davidovits, J., Comrie, D. C., Paterson, J. H., and Ritcey, D. J., 1990, 'Geopolmeric concretes for environmental protection', *ACI Concrete International*, 12, 30-40.

Malhotra, V. M., 1988, 'Sulphur concrete and sulphur-infiltrated concrete', in *New Concrete Material -- (Concrete Technology and Design volume 1)*, eds R. N. Swamy, Surrey University Press, London, pp. 13-21.

Pavlik, V., 1994, 'Corrosion of hardened cement paste in acetic and nitric acids part II: formation and chemical composition of the corrosion products layer', *Cement and Concrete Research*, **24**(8), 1495-1508.

Redner, J. A., His, R. P., Esfandi E., T., Sydney, R., 1998, '*Evaluation of Protective Coatings for Concrete*', County Sanitation Districts of Los Angeles County, Whittier, Calfonia, August.

Rendell, F., Jauberthie, R., 1999, 'The deterioration of mortar in sulphate environments', *Construction and building materials*, **13**(6), 321-327.

Rostami, H., Brendley, W., 2003, 'Alkali ash material: a novel fly ash based cement', *Environmental Science and Technology*, **37**(15), 3454-3457.

Song, X. J., Chang, Z. T., Marosszeky, M., Munn, R., 2003, 'Enhancing acid resistance of concrete by varying cement and aggregate types', the Concrete in the 3rd Millennium, Australian Concrete Institute, Brisbane, Australia, 17-19 July 2003, pp851-862.

Impact of Rice Husk Ash on the Performance of Durian Fiber-based Construction Materials

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ABSTRACT

The aim of this study was to evaluate the impact of rice husk ash on the performance of durian fiber-based construction materials. Rice husk ash is a by-product of power-plant. Physical and chemical properties of the Portland cement and rice husk ash were investigated. X-Ray diffraction analysis revealed the presence of quartz silicon dioxide in rice husk ash. Examination of the X-Ray fluorescence analysis of rice husk ash showed that silicon dioxide was the principal minerals of rice husk ash. Rice husk ash was detected by scanning electron microscopy (SEM). It revealed that rice husk ash have irregular and crushed shaped particle. In this study, mixing proportion conditions of durian fiber-based construction material production are as follows, durian fiber length of < 2 mm, cement: sand: durian fiber as 1: 1: 0.10 and 0.6 W/C ratio. Two sizes of sand commercially available were considered: < 0.71 mm and 0.855 ± 0.145 mm. Rice husk ash was used to replace Portland cement type I by weight of 0%, 10%, 20%, 30% and 40% to produce composite. Dimensions per spicemen were fabricated : 5: 5: 5 cm^3. Spicemens were incubated in water for 28 days. The reults revealed the replacement of Portland cement type I with 10%, 20%, 30% and 40% of ground rice husk ash produced high strength durian fiber based construction material and 30% and replacing portland cement type I by ground rice husk ash gave the highest compressive strength. Compressive strength of composites increased when its fineness was increased by grinding. The fineness of rice husk ash was the major factor affecting the strength of composites. The results also showed that the bulk density and thermal conductivity of composites are inversely proportional to the percentage of rice husk ash content and size of sand. In addition, chemical composition of durian fiber from dried durian peel revealed that durian fiber is composed of holocellose, hemicelluse and lignin. Replacing partial Portland cement by rice husk ash reduced alkaline pore water in the composite, thus, reduced breaking the link between the individual fiber cells. Rice husk ash can slow down the embrittelment process of natural fiber composite.

KEYWORDS

Compressive strength, Durability, SEM, X-Ray diffraction, X-Ray florescence

1 INTRODUCTION

For fiber cement composite, long term stability under various environmental and an exposure condition is the most important property which needs to be established. Although natural fibers appear to be prospective reinforcing materials, it has been found that after several years of its exposing to outdoor environment, the fibers grow brittle with time. The reason is that the alkali pore water in the matrix dissolves the lignin and hemicellulose in fibers so that they become decomposed and lose the reinforcing capacity Gram [1983].

The decomposition of cellulose in an alkaline environment can take place in accordance with two different mechanisms. One is the peeling-off mechanism which occurs at the end of the molecular chain. The end group, which is reductive, reacts with OH- and forms isosaccharin acid (CH_2OH) which is unhooked from the molecular chain. End groups are liberated in this way all the time. The probability of the end groups are liberated in this way all the time. The probability of the end group forming metasaccharin acid instead, which is not unhooked and which is stable in an alkaline solution. Since the degree of polymerization of cellulose of many natural fibers such as sisal is high, about 25000, the peeling-off mechanism is fairly harmless in itself. Peeling-off is said to occur within a wide temperature interval but the rate does not become markedly high until a temperature of about 75 °C or above is reached.

The other form of cellulose decomposition consists of alkaline hydrolysis. This caused the molecular chain to divide and the degree of polymerization decrease. Since the division of the molecular chain entails the exposure of new reductive end groups, the peeling-off mechanism can be started. Alkaline hydrolysis does not take place at a high rate until the temperature is in excess of 100 °C.

Lignin consists of aromatic substances, is easily broken down in an alkaline environment. Lignin begins to soften at 70ºC – 80ºC. At 120 ºC it is partly liquid.

The primary causes of the change in the characteristics of the natural fibers in concrete are due to the chemical decomposition of the lignin and the hemicellulose. The alkaline pore water in the concrete dissolves the lignin and hemicellulose and thus breaks the link between the individual fiber cells.

To produce good quality fiber reinforced composite materials for construction, many researches have been carried out to find appropriate method to utilize these natural fibers as effectively and economically as possible. A key concern with the use of cellulose fiber reinforced cement composites relates to the long-term durability of fibers in cement, particularly when the product is used in severse exposure conditions.

The alkaline pore water in concrete reacts with the lignin and hemicellulose existing in the middle lamellae of cellulose fibers, thus weakening the link between the individual fiber cells which constitute the natural fibers. Outdoor exposure conditions increase the moisture movements needed for alkaline pore water to reach and progressively decompose the natural fibers leading to the embrittlement of the composites. According to Shafiq [1988] showed that partial success in control of alkaline attack on cellulose fibers has been achieved through the reduction of the alkalinity of cement environment by the use of pozzolanic admixtures.

The alkaline of the composite pore water can be reduced by replacing some of the ordinary Portland cement with various pozzolanas. This is achieved when the calcium hydroxide, which is formed in connection with the cement hydration, reacts in parts with the silica present in the pozzolana. When the free calcium hydroxide has been completely consumed. The carbonation of the matrix is also facilitated, thus entailing a marked reduction in the pH value for the pore water. The following pozzolanas could be used as a substitute for cement.
In the hydration of Portland cement, Shafiq [1987]calcium silicate hydrates are formed liberating $Ca(OH)_2$. The later reacts with the reactive, silica in the RHA to further form calcium silicate hydrates. The chemical reactions can be represented by the following equations:

$$2(3CaO.SiO_2) + 6H_2O \rightarrow 3CaO.SiO_2.3H_2O + 3Ca(OH)_2$$
$$2(2CaO.SiO_2) + 4H_2O \rightarrow 3CaO.2SiO_2.3H_2O + Ca(OH)_2$$
$$3Ca(OH)_2 + 2SiO_2 \rightarrow 3CaO.2SiO_2.3H_2O$$

When high amount of cement is replaced by RHA, the alkalinity of the cement matrix is lowered and the carbonation process may go faster, Shafiq [1987].

The durability of mortars containing Rice husk ash has been studied by Spire *et a*l. [1999] who noted an increase in compressive strength. Rice husk ash has been used in many countries as a low cost concrete admixture because of its role as a filler and a pozzolan [Jauberthiea 2000]. It is a very fine pozzolanic material. It contains considerable amount of SiO_2. Investigations on the production of rice husk ash as pozzolana with high activity and the possible application of it in cement and concrete have been made by Nehdi [2003].

In order to improve performance and durability of the composites, it is necessary to find remedial solutions to counteract the embrittlement process of natural fiber reinforced composite materials. The objective of this study is to evaluate the impact of rice husk ash on the performance of durian fiber-based construction materials. The influences on the physical, mechanical and thermal conductivity properties of composite are discussed, and the results are compared with a control composite.

2 RESEARCH METHODOLOGY

2.1 Analysis of durian peel fiber

2.1.1 Chemical analysis of dried durian pee and dried durian peel fiber

When natural fiber reinforced composites are used, the durability question should be raised. According to Gram [1983] found that sisal fiber reinforced composites became brittle within a year when stored outdoor in the tropics. The reason for this was that the lignin and hemicellulose in fibers were dissolved in alkaline environment of composite and the fibers lost their reinforcing capacity. Therefore, it is necessary to know the basic and chemical composition of durian peel. Alpha-cellulose is the pulp fraction resistance to 17.5% and 9.45% sodium hydroxide solution under conditions of the test. It indicates undegraded, higher-molecular-weight cellulose content in pulp. Beta-cellulose is the soluble fraction which is reprecipitated on acidification of the solutions. It indicated that of degraded cellulose. Gamma-cellulose is that fraction remaining in the solution. It consists mainly of hemi-cellulose. Lignin represents what is called the “incrusting material” forming a part of the cell wall and middle lamella in wood. It is an aromatic, amorphous substance containing phenolic methoxyl, hydroxyl and other constituent groups; its chemical structure has not been fully elucidated. The carbohydrates in pulp are hydrolyzed and solubilized by sulfuric acid; the acid-insoluble lignin is filtered off, dried and weighed.

The results of dried durian peel and dried durian fiber chemical analysis performed following TAPPI standards are shown in Table 1. It can be seen that chemical compositions of dried durian peel and dried durian peel fiber are not much different as shown in Table 1. Dried durian peel and dried durian peel fiber have lignin and hemicellulose. The primary cause of the change in the characteristics of the natural fibers in composite is due to the chemical decomposition of the lignin and the hemicellulose. The alkaline pore water in the composite dissolves the lignin and hemicellulose and thus breaks the link between the individual fiber cells.

Chemical composition	*Dried Durian Peel* [%]	*Dried Durian Peel Fiber* [%]	*Standard*
Ash content	5.5	4.3	TAPPI-T211-om-93
Alcohol-benzene solubility	13.4	11.5	TAPPI-T204-om-93
Lignin (ash corrected)	10.9	10.7	TAPPI-T222-om-98
Holocellulose	47.1	54.2	Acid Chlorite's Browing
Alpha cellulose	31.6	35.6	TAPPI-T203-cm-88
Hemicellulose	15.5	18.6	-

Table 1. Chemical composition of durian peel and durian fiber

2.2 Analysis of Rice Husk Ash (RHA)

2.2.1 Physical properties and chemical analyses of the Portland cement and RHA

In this study, RHA was carried out from power plant. Ground RHA had been obtained by using steel ball mill for 3 hours. Physical and Oxide analyses for the RHA and Portland cement were conducted [Table 2].

Properties	*Unground RHA*	*Ground RHA*	*Portland cement*
Physical Tests			
Specific Gravity	0.25	2.4	3.14
Fineness			
- passing 45 µm, %	21.65	99.6	95.3
- nitrogen absorption, m^2/g	-	20.04	-
Chemical Analyses, %			
Silicon Dioxide (SiO_2)	91.6	88.2	12.7
Aluminium Oxide (Al_2O_3)	0.3	0.15	2.63
Phosphorous Oxide (P_2O_5)	0.37	0.47	0.049
Potassium Oxide (K_2O)	1.61	2.08	0.59
Calcium Oxide (CaO)	0.65	0.99	63.8
Titanium Oxide (TiO_2)	0.019	0.016	0.22
Manganese Oxide (MnO)	0.115	0.13	0.052
Ferric Oxide (Fe_2O_3)	0.19	0.89	2.37
Loss in ignition (LOI) (%)	5	5	13.68

Table 2 Physical properties and chemical analyses of the Portland cement and RHA

The physical properties of Portland cement type I is shown in Table 2 with different fineness. Ground RHA has high fineness because it passed 99.6 % on sieve No. 325. In addition, value of ground RHA's nitrogen absoption is 20.04. Values of specific gravity of unground RHA and ground RHA are shown in Table 2. It can be seen that unground RHA possesses lower specific gravity than ground RHA. This can be explained by the fact that ground RHA has high contents of Fe_2O_3, MgO and CaO since these oxides posses high molecular weighs. Whereas high alumina and silica tend to lower the molecular weighs.

Examination of chemical compositions was detected by using X-Ray fluorescence spectrometer. It can be seen that unground RHA, ground RHA and Portland cement composed of different amount of oxides. Table 2 presents that SiO_2 was the principle minerals of RHA. The high content of SiO_2 plays an important role on pozzolanic properties. In addition, SiO_2 content of ground RHA is lower than unground RHA and Fe_2O_3 content of ground RHA is higher than unground RHA. As a result, ground RHA is contaminated by steel ball in grinding process.

In the hydration of Portland cement, Shafiq [1987] calcium silicate hydrates are formed liberating $Ca(OH)_2$. The later reacts with the reactive, silica in the RHA to further form calcium silicate hydrates. When high amount of cement is replaced by RHA, the alkalinity of the cement matrix is lowered. The alkaline of the composite pore water can be reduced. It can slow down the embitterment process of natural fiber composite.

2.2.2 X-Ray diffraction analysis

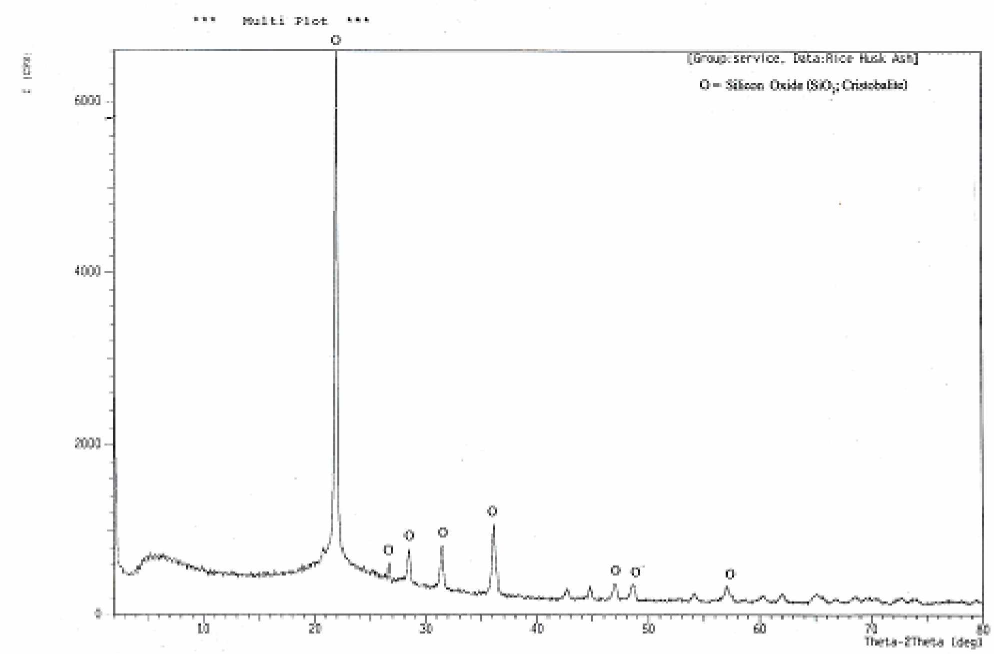

Figure 1. X-ray diffraction analysis of the RHA

The RHA was ground to a powder and analyses by X-ray diffraction. The diffraction diagram, Fig. 1, showed a difractogram of cristobalite. Cristobalite is the tetragonal polymorph of SiO_2.

2.2.3 Scanning electron microscopy and micro-analysis of RHA

The microscopic photographs of unground RHA and ground RHA are shown in Figs 2 and 3, respectively. Unground RHA presented the burnt husks and grain of quartz evident. It can be seen that unground RHA and ground RHA are not spherical. Rice husk ash have irregular and crushed shaped particle.

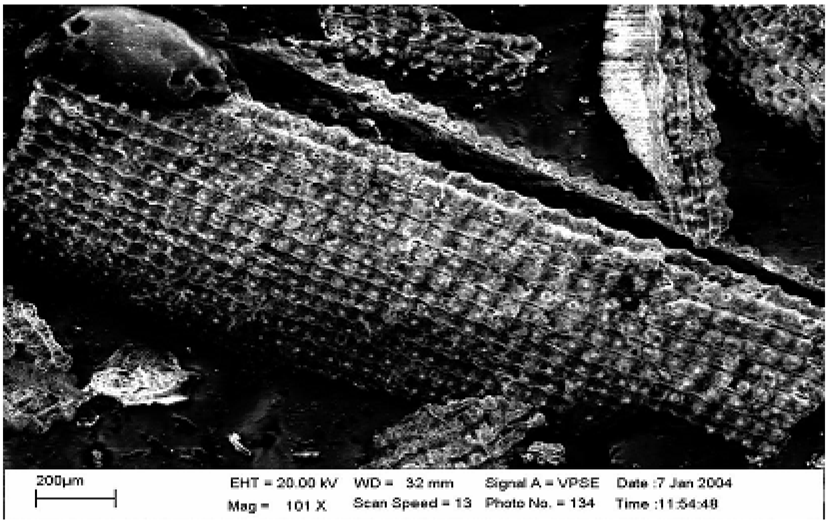

Figure 2. SEM observation of the unground RHA (Magnify 101 X)

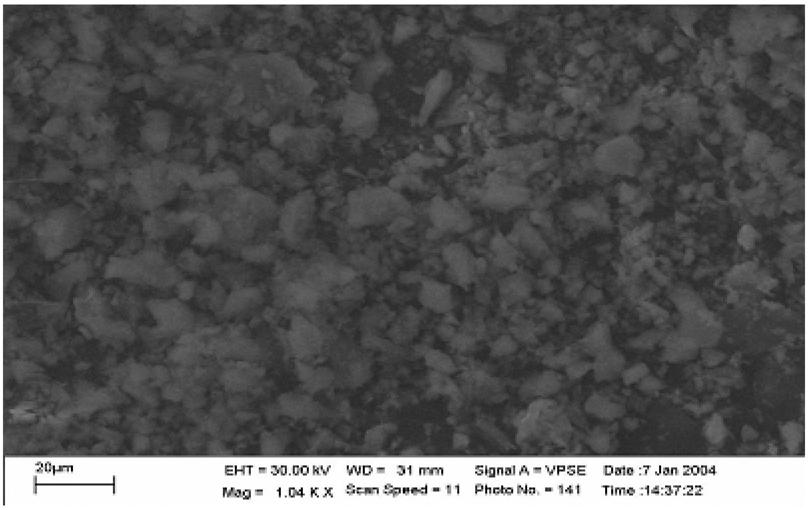

Figure 3. SEM observation of the ground RHA (Magnify 1.04 KX)

2.3 Composite Perparation

2.3.1 Fiber Preparation

Fiber preparation was made following this step; (i) Drying fresh durian peel, (ii) Grinding dried durian peel by Hammermill and (iii) Screening to remove excess fines by sieve machine. Durian fruit, fresh durian peel, dried durian peel, dried durian peel and dried durian fiber as shown in Fig.4.

2.3.2 Composites Preparation

Mixing proportion; cement: sand: fiber (1: 1: 0.1) and 0.6 Water/Cement ratio were used in condition of durian fiber based construciton material production. RHA was used to replace cement that varied from 0, 10, 20, 30 and 40 % by weight. Dimension per specimen were fabricated as 5 x 5 x 5 cm^3. Two sizes of sand commercially available were considered: < 0.71 mm and 0.855 ± 0.145 mm. Specimens were incubated in water for 28 days. The composite specimens were tested based on the following testing standard: (i) Physical property as bulk density performed according to ASTM C 134-88, (ii) Thermal property as thermal conductivity performed according to ASTM C 177 and (iii) Mechanical property as compressive strength performed according to ASTM C 109-95.

Figure 4. Durian fruit (Top-Left), fresh durian peel (Top-Right), dried durian peel (Bottom-Left) and dried durian fiber (Bottom-Right)

3 RESULTS AND DISCUSSION

To facilitate comparison, Table 3 groups all results of measurements.

Type of RHA	*Size of sand*	*RHA content* [%]	*Bulk density* [kg/m^3] *ASTM C 134-88*	*Compressive Strength* [Mpa] *ASTM C 109-95*	*Thermal conductivity* [W/m.K)] *ASTM C-177*
Unground RHA	< 0.71	0	2024	8.20	1.178
		10	1784	3.439	1.0878
		20	1534	4.373	0.928
		30	1386	5.914	0.819
		40	1220	3.138	0.689
	0.855 ± 0.145	0	2016	6.28	1.125
		10	1635	2.749	0.9864
		20	1413	3.949	0.8148
		30	1284	4.644	0.7152
		40	1183	2.894	0.5916
Ground RHA	< 0.71	0	2024	8.20	1.178
		10	2020	16.279	1.1754
		20	1962	18.614	1.0776
		30	1922	20.595	1.0524
		40	1841	16.769	1.0245
	0.855 ± 0.145	0	2016	6.28	1.125
		10	2016	10.572	1.1142
		20	1811	17.309	0.987
		30	1813	18.554	0.865
		40	1736	14.749	0.798

Table 3 Composites specimens specification for unground and ground RHA

According to the experimental results, conclusions can be presented as follows: Replacing cement with 0%, 10%, 20%, 30% and 40% of ground RHA produces high strength durian fiber based construction material and 30% cement replacement by ground RHA gives the highest compressive strength as shown in Fig. 5. It was found that compressive strength of durian fiber based construction materials containing unground RHA are lower than ground RHA. The degree of fineness achieved was not sufficient to make high strength. Two sizes of sand commercially available were considered: < 0.71 mm and 0.855 ± 0.145 mm. With sand size < 0.71 mm, the compressive strength is higher than with 0.855 ± 0.145 mm. Fine sand can insert into matrix between fibers. Therefore, it reduces voids and get high compressive strength.

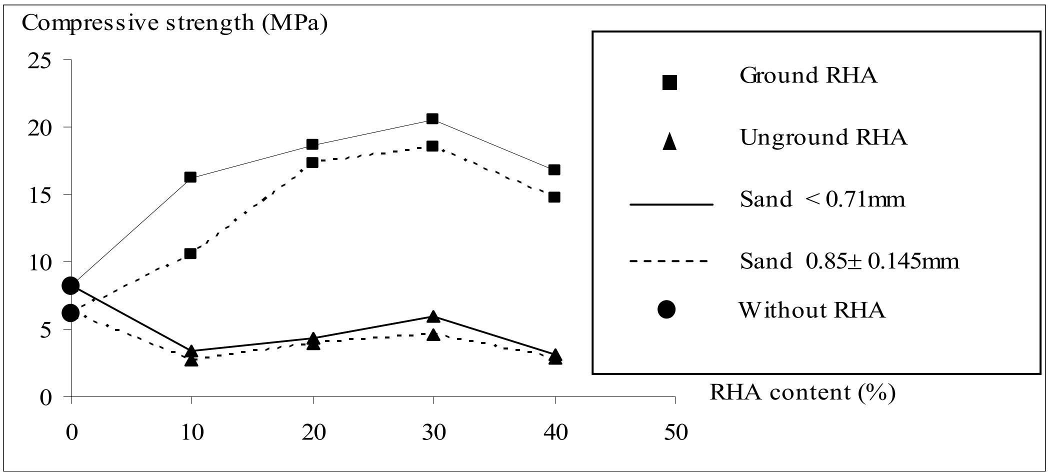

Figure 5. Compressive strength of composites vs. RHA content for different RHA and sizes of sand

The bulk density of composites is inversely proportional to the percentage of RHA content as shown in Fig. 6. Specific gravity of unground RHA and ground RHA are lower than Portland cement. In addition, Portland cement has high degree of fineness. It was found that replacing Portland cement type I by RHA get more voids' ratio, the lighter the composite and the lower its thermal conductivity.

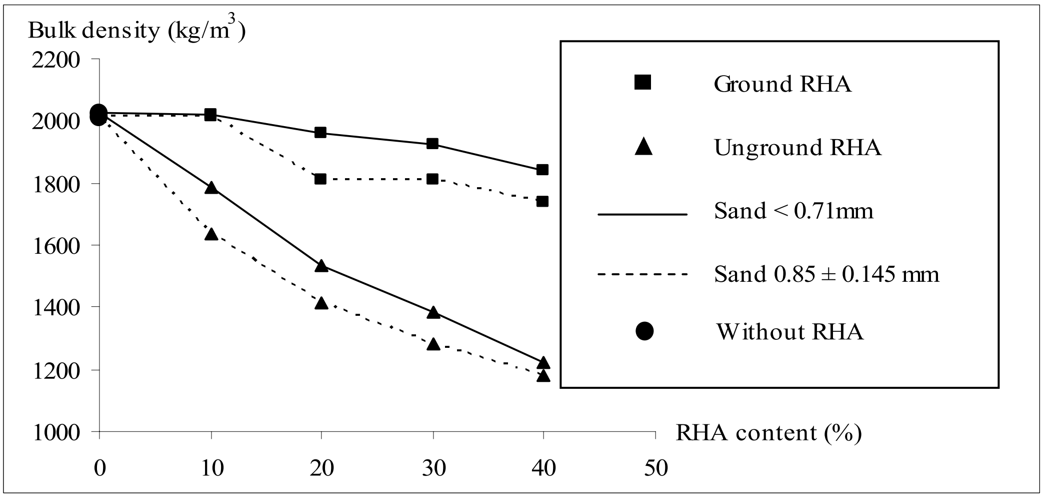

Figure 6. Bulk density of composites vs. RHA content for different RHA and sizes of sand

The thermal conductivity of composites is inversely proportional to the voids in the specimen. With sand size 0.85 ± 0.145 mm, the thermal conductivity of composite is lower than that with < 0.71 mm as shown in Fig. 7. Increasing the size of sand create void and get low thermal conductivity and low density as shown in Figs. 6 and 7.

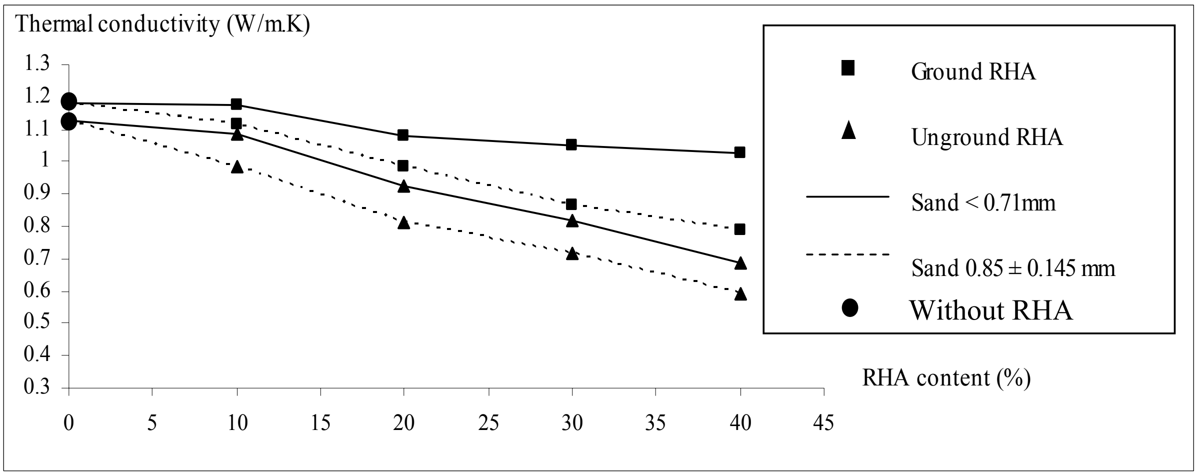

Figure 7. Thermal conductivity of composites vs. RHA content for different RHA and sizes of sand

4 CONCLUSIONS

According to the experimental results, conclusions can be drawn as follows: (i) The replacement of Portland cement type I with 10%, 20%, 30% and 40% of ground RHA produces high strength durian fiber based construction material and 30% Portland cement type I by ground RHA gives the highest compressive strength. (ii) The fineness of rice husk ash was the major factor affecting the strength of composites.The ground RHA with high fineness is a suitable pozzolanic material to improve performance of durian fiber based construction materials. (iii)The size of sand is significant to the structure of composite. It seems that increasing the size of sand decreases compressive strength and thermal conductivity and bulk density.

5 ACKNOWLEDGMENTS

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6 REFERENCES

Gram, H. E. 1983, *Durability of Natural Fibres in Concrete*. Swedish Cement and Concrete Research Institute, S – 100 44, CBI Fo 1:83, pp. 1-255, Stockholm, Sweden.

Jauberthie, R. 2000, 'Origin of the pozzolanic effect of rice husks', *Construction and Building Materials,* **14**[8], 419-423.

Nehdi, M., Duquette, J. & Damatty, A.E. 2003, 'Performance of rice husk ash produced using a new technology as a mineral admixture in concrete', *Cement and Concrete Research,* **33**[8], 215-273.

Shafiq, N.1988, 'Durability of Natural Fibers in RHA Mortar', *Journal of Ferrocement.,* **18**[3], 249-262.

Spire, S. Elefthriou K. & Siludom, S. 1999, 'Durability of concrete containing rice husk ash in an additive', Proc. of Exploiting wastes in concrete, Creating with Concrete, Dundee, United Kingdom, pp. 283-290.

Durability and service life of CFRPs Externally Bonded to Concrete structures

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TT3-59

ABSTRACT

Composite materials find their application in civil engineering in the strengthening of reinforced concrete and pre-stressed concrete structures. The design method of these reinforcement is based on classical methods used for reinforced concrete structure, taking into account the slip phenomena at the interface. The mechanical behavior of each material and particularly the damage states of steel, concrete and composite are also considered. If short term and long term behavior law of steel, concrete and composite has been well identified for, it is different for concrete composite interface. Varastehpour has brought out the necessity to determine the shear mechanical law behavior of the concrete/composite interface. The environmental action on concrete-composite adhesive layer should modify this law. It is necessary to identify the influence of combined effects "time-temperature-water-strength" on composite reinforcement.

KEYWORDS
carbon epoxy reinforcement, creep, thermo-stimulate test, moisture, temperature.

1 INTRODUCTION

Externally bonded carbon-epoxy fibre reinforced polymers (CFRPs) have been used extensively to restore or to increase the capacities of reinforced concrete beams (Meier et al. [1]; Neale and Labossière, [2]; Varastehpour and Hamelin [3]). FRPs combine high strength unidirectional fibres with an epoxy matrix that cure at temperatures ranging between 5°C and 30°C. FRP strengthening materials for field applications have been commercialized by various manufacturers. Under static loading they are proved to be sufficiently reliable, as long as appropriate anchorages of the FRP are provided. Several authors have shown that, when failure occurs by debonding, it is always the concrete cover above the FRP-concrete interface that shears-off. Most of these studies have been conducted under static loadings.
Ferrier and Hamelin [6, 7]) have clearly shown that FRP-strengthened structures present better performance than unstrengthened ones. In most cases, it has been observed that failure of the structure is initiated by successive yielding of the reinforcing steel in tension in one or several locations. When the debonding of the FRP laminate occurred, it was considered to be a secondary failure mode following the reinforcing steel rebars yielding and failure. Technical literature on the subject relates that durability of structure strengthened by carbon epoxy composite depends on the environmental exposure of the structure [8, 9, 10]. Most of authors test the durability of reinforced structure under static loading by exposing specimen to a specific environmental condition (frost, high temperature, moisture). The aim of this paper is to determine the influence of combined effect of loading-temperature-moisture on the behavior of the beam. This behavior depends on the stress transfer between the composite and the concrete structure. So, the shear mechanical law evolution of the adhesive layer is predominant on the behavior of all the reinforced structure.We have carried out a set of tests on reinforced concrete specimens. The tests consist in applying shear stress during six months, with two environmental conditions. The first samples are exposed to 60°C environmental condition and the second are placed in water (20°C). For all the time of exposure, a constant stress is applied on the interface. Composites reinforcements made of two-carbon fabric strips with dimension of 425.50 mm^2. The strip assembles two concrete blocks on 200 mm length. The fabric has been designed to avoid failure of the composite. The failure mode is delamination of the interface. The level of long term loading is specified with thermo-stimulated tests for several levels of shear stress. When the stress level is determined, the creep test allows to set up, the durability of the repair, and the creep of the composite reinforcement under combined effect of stress and environment. The thermo-stimulated test is compare to a six-month-long test period. The use of thermo-stimulated tests allows the prediction of creeping using the master curve construction with the time temperature principle. The identification of mechanical behavior law allows to determine the level of shear stress to be applied to the reinforcement in the case of a flexural behavior. We have studied several epoxy polymer with three different values of glass transition temperature (40°C, 50°C, 85°C). The result allows to select the most appropriate polymer to use for the strengthening of a structure. The durability of the repaired or reinforced structure depends on the behavior of the adhesive layer between concrete and composite.

2. MATERIAL AND EXPERIMENTAL PARAMETERS

2.1 FRP laminates and adhesive

As recommended by Karbhari and Seible [11], several epoxy systems have been investigated, and three CFRP laminates were tested. In order to evaluate the performance of a composite repair, we propose to determine the evolution of the polymer shear modulus (G) of the composite according to the time, temperature or other environmental conditions (moisture). A long test, at 60°C and less than 20 % of humidity, and a short term thermo-stimulated test have been carried out in order to determine the evolution of the shear modulus. These two tests allow the determination of high temperature and time influences on the evolution of the mechanical characteristics. The main characteristics of the composite reinforcements are given by table 1. The average thickness of the composite, measured after a plate strengthening is nearly 1 mm. The thickness of the adhesive layer is 0.40 mm. The most common method for bonding the FRP sheets on the concrete surface is by a hand-applied wet lay-up procedure, after proper sandblast cleaning (air blasting) of the concrete surface.

Table 1 : Material Properties

Material	A	B	C
Fibers	Carbon	Carbon	Carbon
Young Modulus (GPa)	235	235	235
Ultimate Strength (MPa)	3500	3500	3500
Resin	Bi component Epoxy	Bi component Epoxy	Bi component Epoxy
Glass Temperature Tg (°C)	46	55	80
Composite			
Young Modulus (GPa)	70±6600	75±6800	88±1600
Ultimate Strength (MPa)	540±35	885±55	1000±45
Elongation (%)	0,77±02	1.4±02	1.13±02

2.2 Double-lap FRP-concrete joints

A review of the literature reveals that several techniques exist for characterizing the behaviour of FRPs bonded to concrete. The first type of test consists of a single-lap shear test while the second type involves double-lap joint specimens Several authors have investigated in detail conditions at the end of the FRP laminate. They have taken into account the influence of application methods, size effects, interface stress concentration effects, as well as anchorage conditions. A carbon fabric and an epoxy polymer connect two concrete blocks separated by 20 mm. The displacement of the two blocks is due to the lengthening of the reinforcement element (Δl_2), and to the slippage (Δl) at the interface between the concrete and the composite. The value of ΔL is given by Δl_1-Δl_2. The reinforcement elements bonded on the two opposite faces present a surface of 50 x 415 mm^2. A constant shear load is applied. This load corresponds to 50 % of the failure load determined by a static test at 60°C (Fig. 2). This failure (under static loading) occurs by delamination of the reinforcement element. The reinforcement lengthening is measured by gauges (350 ohms) bonded on the composite central part. The space between the concrete blocks is measured by two displacement sensors (types RDP D5 ±20 mm). The displacement sensors present a temperature of use ranging from -20°C to 120°C. The displacement induced by the lengthening of the composite is subtracted from the spacing of blocks (Fig. 1). The global slipping of the composite plates is measured. The shear modulus is calculated in a first approach with the relation (1).

$$G = \frac{\tau_{moy}}{\Delta l2 - \Delta l1} \cdot s \qquad (1)$$

with : G : shear modulus (MPa); τ_{moy} : mean shear stress (MPa), Δl_2 : concrete bloc displacement (m), Δl_1 : composite displacement (m); Lc : polymer thickness (m)

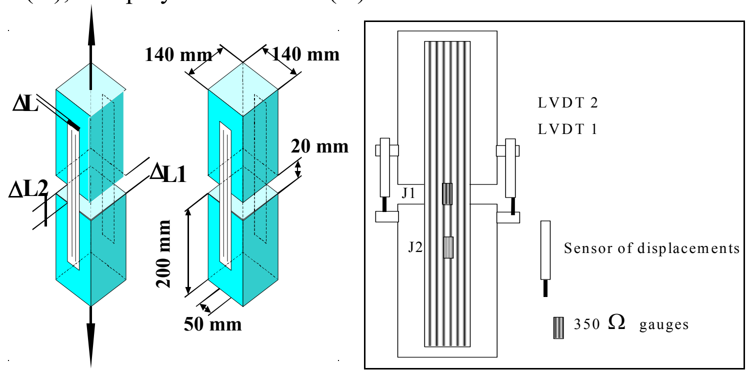


Figure 1: Adhesive Shear Test Setup

2.4 Experimental parameters for creep tests

Once direct tensile-shear tests are carried out, we determine the creep load corresponding to 50 % of the failure load at 60°C. This creep load is 4.5 KN for composite A, and 9 KN for composite B and C. This creep level corresponds to a linear behavior of the polymer (Fig. 2). The polymer thickness is 0.40 mm.

3. EXPERIMENTAL RESULTS

3.1 Study of temperature effect on the creep of concrete – composite adhesive layer

3.1.1 Thermo-stimulated Tests, Construction of the Shear-Creeping Master Curve
The test specimens are placed in a thermo-regulated room at 20°C and a load of 4.5 KN is applied, i.e. an average shear stress of 0.20 MPa. The slipping is measured during three-hour-periods, up to 60°C. The results of these tests allow to determine the long-term creep behavior of the composite by using the principle of time-temperature superposition given by WLF [12]. The construction of the master curve can be done step by step with the superposition of each temperature curve. This construction allows assessing the value of the shift factor for each temperature. The result of this construction (Fig. 3) shows that the shift factor (a_t) can be calculated using the master curve construction following a bilinear law. As a first step, we neglected the vertical shift factor, considering that the polymer does not have any volume variation or aging. The results of these tests shows on composite A that the shear modulus decreases from 400 down to 100 MPa during the first month that follows the loading (Fig. 4a). On composite B from 1200 MPa to 820 MPa (Fig. 4b). On composite C, the result shows that the shear modulus decreases from 2000 MPa to 1500 MPa during the same time (Fig. 4c).

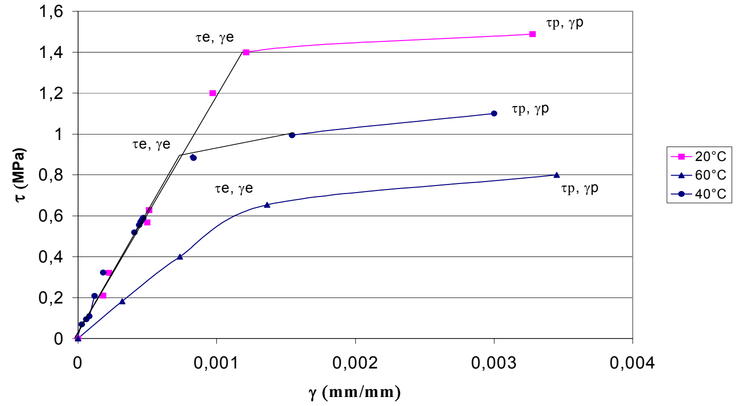


Figure 2: Mechanical shear behavior of the concrete–composite interface (composite B)

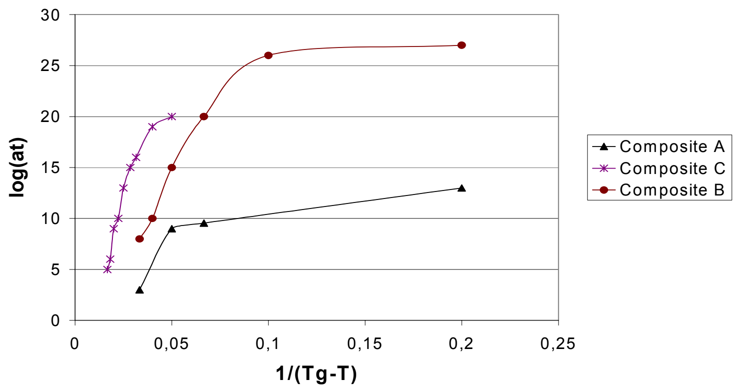
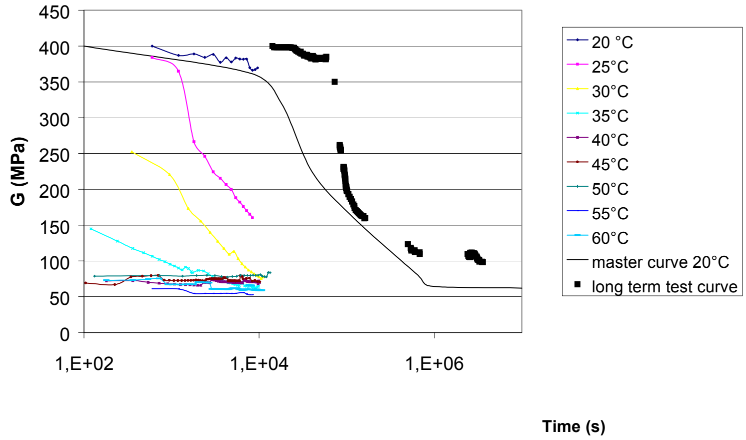


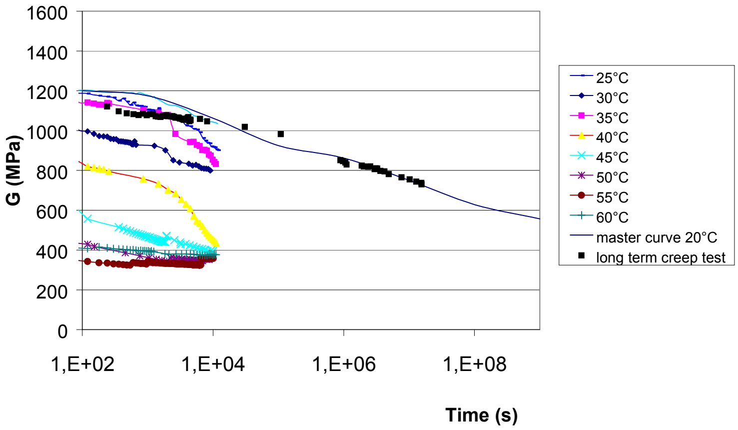
Figure 3: Shift factor for each temperature at 20°C

3.1.2 Long term Shear Creep Tests at 60°C

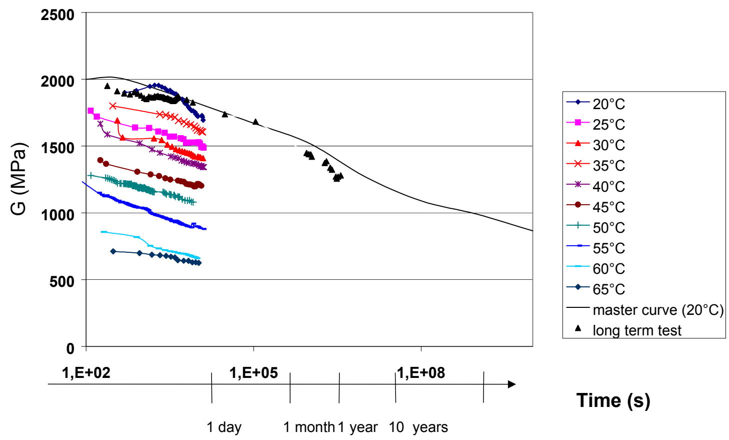
To verify the master curve construction, we use a six-month-long-term-test under the same creep level. We have carry out this test at 60°C to evaluate the higher creep effect on the adhesive layer. To compare with the master curve construction at 20°C, we plot the curve at the same scale of shear modulus. A stress of 0.20 MPa is applied on composite A for one-month period, under 60°C and dry environmental conditions. A stress of 0.50 MPa is applied on composite B and C in the same condition. The evolution of the shear modulus is represented by the curve in Fig 9a and 9b. On composite A, we observe that the decrease of the shear modulus is important. The value of shear modulus at 60°C is 70 MPa, a decrease of 70 % during the first month is observed. On composite C, the value of shear modulus at 60°C is 1200 MPa, it decreases of 20 % during the first month, on composite B, the value of shear modulus at 60°C is 800 MPa, it decreases of 35 % during the same period. The comparison between the two tests is assessed by superposition of the long-term test and the master curve. This long-term test curve is fitted from it initial values toward the initial value of the master curve, then each value of the long-term test is fitted with the same factor. The comparison between the two tests illustrates the opportunity to obtain values of the long-term shear modulus from a thermo-stimulated test. Those tests have shown that thermo-stimulated test can be used to identify the long term creep behavior of epoxy polymer tested (Fig. 4).



(a) composite A



(b) composite B



(c) composite C

Figure 4: Evolution of the shear modulus at 60°C for composite A and B and C

3.1.3 Study Of Moisture Effect On Creep Of Concrete – Composite adhesive layer

To identify the effect of moisture on the concrete composite interface the tensile shear test is used at 20 °C with 100 % of moisture. The shear modulus determined shows that the interface properties decrease (Fig. 5), during the first 2 months of loading. During the test, moisture absorption is measured on the composite. A decrease of 30% of mechanical properties has been observed for a total absoption of water of 0.60 %.

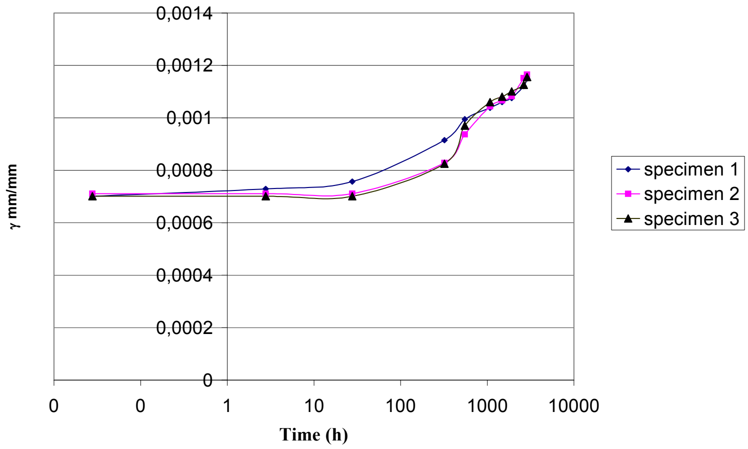


Figure 5: Slipping by length of anchorage function as time exposure

The result of this study has shown that the polymer specimen absorb more water and less rapidly than the composite specimen. This can be explained by the thickness of the specimen. The comparison between the long-term creep curve and the master curve shows that the influence of the water absorption on the mechanical behavior is not so important. To validate this result tensile tests on aging specimen are under-going to evaluate the influence of water absorption on the mechanical law behavior. It seems that the behavior is not so dependent of moisture absorption. This justifies the fact that for the master curve construction the vertical shift factor can be neglected with our polymer.

4. RHEOLOGY MODEL FOR CREEPING FUNCTION IDENTIFICATION

The evolution of shear modulus is given for all the test curves assessed in this study. The shear modulus given by the eqn 2, results from the rheological model. The evolution of the shear modulus of the double lap joint is given by the rheological model:

$$G(t) = G_1 + G_2 \cdot \exp(\frac{-t}{\eta_1}) + G_3 \cdot \exp(\frac{-t}{\eta_2}) - \frac{G_4}{t} \qquad (2)$$

The parameters of the rheological model are fitted with experimental data. The test curves are determined with a correlation factor of 0.90. The parameter G_4/t is added to the rheological relation to modify the beginning of the creep curve. For all the tests or master curves, the parameters are given in the table 2. The use of rheological model allows determining the long-term shear modulus of the interface with any level of shear stress (Fig. 6). The result of this study shows that the polymer with a high T_g and a high shear modulus presents a lower decrease of its mechanical behavior.
The polymer with a glass transition temperature lower than 50 °C should be avoid in civil engineering application because of a high decrease of mechanical characteristics and because of its high creep. In the case of structural application structure, the mechanical law assessed with this method is used to determine the long-term displacement of a beam. The rheological relations are used to determine the slipping in the adhesive layer, this shear-strain in the adhesive layer brings out a loss in strength in the composite during long term loading.
The loss in strength is then recalculated with the stress equilibrium of the section. This iterative calculation method assessed the mid span displacement of the bearing structure, and with suitable failure criteria, the safety factor to apply to the structure is then determined. With the tensile shear tested, and the rheological relation, the influence of any formula of polymer can be tested. With the thermo-stimulated test the polymer with the most accurate behavior can be chosen. With this method, it is possible to use any aging mechanical law (creep, fatigue….) for each material, and the strength evolution in each material can be evaluated.

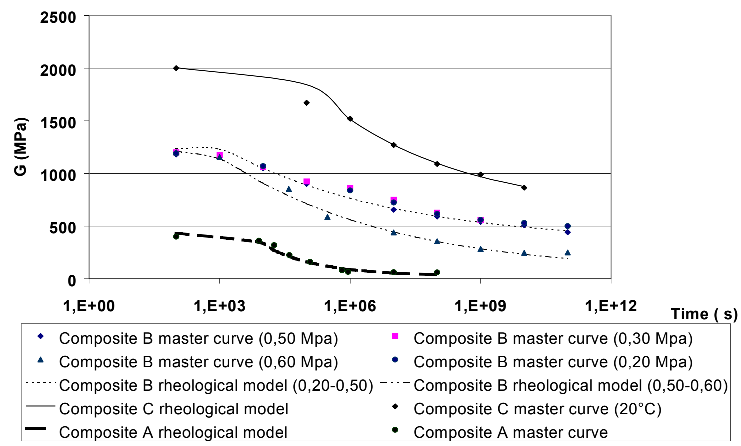


Figure 6: Theoretical and long term experimental creep compliance for adhesive A, B and C

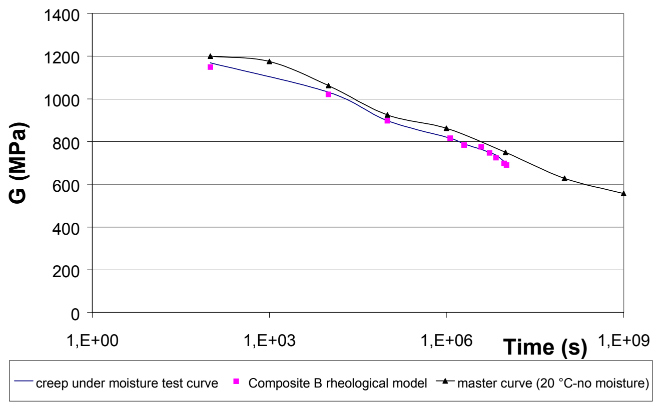


Figure 7: Theoretical and long-term experimental creep under moisture effect

Table 2: Rheological parameters

	E_1	E_2	E_3	E_4	η_1	η_2
Composite A	27.80	4442	1.19	4442	1.19	1.19
Composite B τ=0.20-0.50 Mpa	334	994	3.92	994	3.92	3.92
Composite B τ=0.60 MPa	55	1221	3.81	1221	3.81	3.81
Composite B water exposure τ=0.80MPa	-158	2039	4.47	2039	4.47	4.47
Composite C	618	3021	3.50	3042	3.50	2.78

5. CONCLUSION

The mechanical characteristics of the polymer at the interface between the concrete and carbon fibers are evaluated by performing thermo-stimulated creep tests of. These tests have brought out the following points:

- the shear modulus in dry environmental conditions at 20°C, decreases of 80 % during the first month of loading for a polymer with a T_g of 40°C,
- the shear modulus in dry environmental conditions, at 60°C, decreases of 70 % during the same period for the same polymer,
- the shear modulus in dry environmental conditions at 20°C, decreases of 33 % during the first month of loading for a polymer with a T_g of 85°C,
- the shear modulus in dry environmental conditions at 60°C, decreases of 20 % during the same period for the second polymer,
- the 100 % moisture effect on the shear modulus consists in a decrease of 35% for two months of exposure.

In conclusions, it seems essential to use high shear modulus polymers with high glass transition value, and to evaluate the modulus decrease as a function of time to determine the durability of the beam. The repaired concrete structure quality depends on the adhesive layer performance. The bending design of structure can take into account this slipping phenomena. This prediction can be done with the non-linear calculus method by taking into account the rheological behavior of each material (concrete, composite), but also by taking into account the shear behavior law identified in this study. The identification of concrete-composite shear behavior law with various environmental conditions allows to take into account the coupled effect of loading – temperature – moisture of a reinforced concrete structure.

6. REFERENCES

[1] Meier, U., Deuring, M., Meier, H., Schwegler, G. "Strenthening of structures with CFRP laminates: research and applications in Switzerland.", *Proceedings of ACMBS'1*, Canada, (1992), 243-251.
[2] Neale, K.W., and Labossière, P., *"State of art report on retrofitting and strengthening by continuous fibre in canada.",* Non-Metallic Reinforcement for Concrete Structure, *Japan Concrete Institute, (1997), 25-39.*
[3] Varastehpour, H., and Hamelin, P., "Strengthening of concrete beams using fiber-reinforced plastics.", *Materials and Structures*, RILEM, **30,** (1997), 160-166.
[4] Barnes, R.A., and Mays, G.C., , "Fatigue performance of concrete beams strengthened with CFRP plates.", *Journal of composites for construction*, ASCE 3 (2), (1999), 63-72.
[5] Shahawy, M., and Beitelman, T.E., "Fatigue performance of RC beams strengthened with CFRP laminates." *Proceeding of Durability of fibre reinforced polymers (FRP) composites for construction,* Benmokrane B. , Rahman H. Eds, Université de Sherbrooke, Sherbrooke, Québec, (1998), 169-178.

[6] Ferrier, E., and Hamelin, P., "Fatigue behavior of composite reinforcement for concrete structure. " *ACI scientific publications*, **SP 188-48**, (1999), 535-545.
[7] Ferrier, E., and Hamelin, P., "Concrete beams reinforced by fibre-reinforced plastics: the effect of temperature on the adhesive layer."*, Composite Science and Technology,* **61**, (3), (2001), 425-431.
[8] H.Varastehpour, P. Hamelin, "Structural behavior of reinforced concrete beams strengthened by epoxy bonded FRP plate", *Second international symposium on non metallic (FRP reinforcement of concrete structures)*, Ghent (Belgique), 23 August 1995, p 559-567.
[9] M. Green, K.A. Soudki, "FRP strengthened concrete structures in cold regions", *Recent advances in bridge engineering,* UR Meir and Raimondo Betti (eds) July 1997 p 219-226
[10] Mark.F. Green, Luke A. Bishy, Y. Beaudion, P. Labossière, "Effects of freeze-thaw action on the bond of FRP sheets to concrete", *CDCC 98,* University of Sherbrooke 1998 Canada page 179-190
[11] Karbhari, "Effect of environmental exposure on the external strengthening of concrete with composites- short term bond durability*"*, *Journal of reinforced plastics and composites*, vol 15 – december 1996.
[12] Williams, American Chemistery Society, vol. 77, 3701 1955.

Durability on The Fracture Toughness of Crack-repaired Various Concrete

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ABSTRACT

One of the issues related to maintenance of the domestic stock of concrete structures is the evaluation of the degree of performance recovery and subsequent long-term performance retention brought about by repair. If the long-term performance retention of a concrete structure is not anticipated, then it can be more effective to render the structure to raw materials for structural concrete by reasonable recycling techniques for subsequent reuse as new components. In this light, when a structure is to be repaired, an optimum repair method should principally selected according to the degree of deterioration to ensure restoration of its performance equal to or higher than that before deterioration and long-term retention of the performance. However, few studies have been conducted on the evaluation of mechanical performance retention under deterioration action and crack propagation resistance of repaired concrete.

In this study, the performances on the recovery of fracture toughness and on the effectiveness of durability maintenance for crack-repaired various concrete were investigated. The relation of air content, matrix strength, fiber properties, crack repairing and freeze-thaw durability test to the fracture properties of concrete were evaluated by using a tension softening evaluation system. The specimen of on which the Repairs of cracks using a polymer-based resin on non-fiber concrete showed extremely low toughness in the freeze-thaw durability tests. Toughness and durability performance of the specimen were improved by increasing air entrainment and the addition of vinylon fiber, because of the effect of reducing the stress by inner water pressure. Information was obtained on improving the fracture toughness and durability of crack-repaired concrete. This has direct positive influence on the reliability and maintainability of such systems.

KEYWORDS

PVA fiber, crack-repaired concrete, epoxy resin, fracture toughness, preserving durability

1 INTRODUCTION

One of the issues related to the maintenance of the domestic stock of concrete structures is the evaluation of the degree of performance recovery and subsequent long-term performance retention brought about by repair[Collins & Roper 1990]. If the long-term performance retention of a concrete structure is not anticipated, then it can be more effective to render the structure to raw materials for structural concrete by reasonable recycling techniques for reuse as new members [Basunbul *et al.* 1990]. In this light, when a structure is to be repaired, an optimum repair method should principally selected according to the degree of deterioration to ensure restoration of its performance equal to or higher than that before deterioration and long-term retention of the performance. However, few studies have been conducted on the evaluation of mechanical performance retention under deterioration action and crack propagation resistance of repaired concrete.

To deal with such an issue, the authors repaired various cracked concrete samples by epoxy resin injection and subjected the repaired concretes to repeated freeze-thaw cycles to investigate their toughness retention and resistance to freezing and thawing.

2 OUTLINE OF EXPERIMENT

2.1 Experiment plan

The materials are given in Table 1. Table 2 gives the factors and levels of experiment. Table 3 gives the basic mixture proportions of concrete samples. Polyvinyl alcohol (PVA) fibers were selected as short fibers to be blended in concrete for its excellent bonding with cement paste and energy absorbing capability, as well as its light weight and low price. To investigate the effects of the fiber lengths (L and S), diameters (40 to 660 μm), and fiber contents (0 and 1.2%), five (5) types of fiber phases were prepared. As well, two (2) levels of concrete strengths (high strength with a W/C of 30% and normal strength with a W/C of 60%) were selected, each with the mortar phase composition being kept constant regardless of the volumetric fiber content.

<table>
<tr><th colspan="2">Name</th><th>Mark</th><th colspan="5">Contents</th></tr>
<tr><td>Cement</td><td>Normal Portland</td><td>C</td><td colspan="5">Density3.16(g/cm^3)</td></tr>
<tr><td>Fine Agg.</td><td>Crushed SandStone</td><td>S</td><td colspan="5">Density2.58(g/cm^3)
Absorption1.71(%), Mas Diameter 5.0(mm)</td></tr>
<tr><td rowspan="4">Fiver</td><td rowspan="4">Vinyron</td><td>V</td><td colspan="5">Density 1.30(g/cm^3)</td></tr>
<tr><td colspan="2">Diameter (μm)</td><td>40</td><td>200</td><td>400</td><td>660</td></tr>
<tr><td colspan="2">Length S:12mm</td><td>Sss</td><td>Ss</td><td>Sm</td><td>Sl</td></tr>
<tr><td colspan="2">Length L:30mm</td><td>---</td><td>---</td><td>Lm</td><td>---</td></tr>
<tr><td>Repaired</td><td>Epoxy</td><td>E</td><td colspan="5">JIS A 6024 Low Viscosity Type(130±20 (mPa□s)
Tensile More Strength 20(N/mm^2)e</td></tr>
</table>

Table 1. Materials in the experiment

Factors	Levels□Mark□
Water Cement Ratio(%)	60(N), 30(H)
Installed Fiber Volume(%)	0(00), 1(01), 2 (02)
Fiber Length (mm)	12(S), 30(L)
Fiber Diameter(μm)	660(l), 400(m), 200(s), 40(ss)
Freezing Thaw(Cycle)	0(0c), 60(60c),120(120c)
Type of concrete	Normal (N), Repaired (R)

Table 2. Factors and levels of experiment

<table>
<tr><th rowspan="2">Type</th><th rowspan="2">W/C (%)</th><th colspan="3">Unit Weihgt (kg/m^3)</th><th rowspan="2">Fiber Volume (%)</th><th rowspan="2">Additives (C×%)</th></tr>
<tr><th>Water</th><th>Cement</th><th>Fine Agg.</th></tr>
<tr><td>□</td><td>30</td><td>229</td><td>799</td><td>1326</td><td>0,1,2</td><td>1.5</td></tr>
<tr><td>□</td><td>60</td><td>388</td><td>647</td><td>1069</td><td>0,1,2</td><td>0</td></tr>
</table>

Remarks: These mixtures contain no fibers

Table 3. The basic mixture proportions of concrete samples

2.2 Fabrication of specimens

Table 4 gives the types and properties of specimens. A 20-liter biaxial forced action mixer was used for mixing. After dry-mixing materials other than fibers for 60 sec, water was added and further mixing of the components was continued for 90 sec. Fibers were then added in small portions to achieve a uniform dispersion during the mixing process that lasted a total of 5 min. As-mixed concrete samples were subjected to air content and unit weight testing in accordance with JIS A 1116 and flow testing in accordance with JIS R 5201. Three compression specimens (100 mm, diameter; 200 mm, height) and three toughness specimens (100 by 100 by 120 mm) were fabricated from each sample, demolded at an age of 2 days, and subjected to standard-curing.

Mark		W/C (%)	Fiber Volume(%)	Air Contents(%)	Flow Value	Unit Weight (kg/L)	Compressive Strenght (N/mm^2)
□	00	30	0	5	224×234	2.29	98.3
	Sm01		1	2.9	300×300	2.32	84.3
	Sm02		2	5	275×285	2.25	75.3
	Lm01		1	2.1	273×293	2.35	94.5
	Lm02		2	5	250×256	2.28	86.8
N	00	60	0	2.2	274×279	2.17	42.9
	Sm01		1	2.6	280×288	2.17	36.8
	Sm02		2	2.3	219×226	2.13	35.9
	Lm01		1	0.8	234×260	2.19	44.8
	Lm02		2	1.4	202×224	2.15	40.8
	Sss01		1	1.7	133×135	2.14	39.4
	Ss01		1	1.7	241×253	2.20	47.9
	Sl01		1	1.7	279×295	2.20	47

Table 4. Basic properties of specimens

2.3 Fracture toughness testing

Figure 1 shows a sketch of a wedge-splitting type of fracture toughness test. Original and repaired concretes were subjected to wedge splitting tests [RILEM 1994] for fracture toughness. A wedge was inserted in the notch in the center of the top surface with a ligament depth of 50 mm to induce tension failure, while measuring the load-crack-mouth-opening displacement (L-CMOD) curve. This test began at an age of 28 days for original concretes. Repaired concretes were prepared by epoxy-repair of cracks of original concretes after wedge testing. These were air-cured at 20°C and 60% R.H. until an age of 6 months and then exposed to the specified number of freeze-thaw cycles (0, 60, and 120) before being subjected to the same wedge testing. In order to measure stable L-CMOD curves, loading rates of 0.04 mm/min and 0.02 mm/min were applied to original and repaired concretes, respectively.

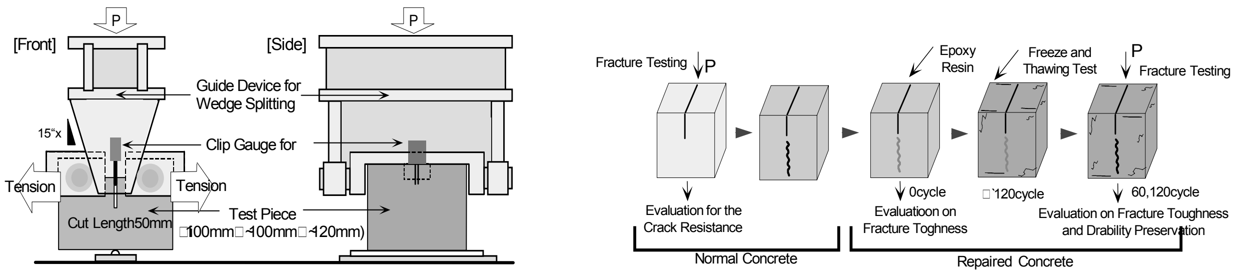

Figure 1. Wedge-splitting type fracture toughness testing

2.4 Method of evaluating toughness

The fracture parameters were calculated from the tension-softening diagram (TSD) determined from the L-CMOD curve by multilinear approximation [Kitsutaka 1997]. A TSD is an index of the relationship between the cohesive force and the CMOD when the progress of fracture is represented by a cohesive force model. Also, the area under the TSD to a CMOD of δu = 0.5 mm was defined as the effective fracture energy, G_F^u, for evaluation in terms of energy (Eq. (1)) [Kitsutaka *et al.* 1994].

$$G_F^{\ u} = \int_0^{\delta u} \sigma(\delta) d\delta \tag{1}$$

where, δ = crack opening displacement[mm], $\sigma(\delta)$ = cohesive stress[MPa].

2.5 Method of crack repairing

An epoxy resin with a low viscosity suitable for fine cracks, to which normal resin is not readily applicable, was used for repairing cracks. The repair was carried out as follows: a silicone sealant was applied to both sides and ends of the cracks (to prevent spilling of resin when it is applied in the crack opening). After confirming that the sealant had completely hardened, the epoxy resin was then injected at the top of the notch using a syringe. By carrying out the injection process twice, repaired specimens were produced with openings completely filled with epoxy resin.

2.6 Freezing and thawing tests

Freezing and thawing tests for evaluating durability were conducted in accordance with JIS A 1148 (Method A). Three repaired specimens for each type of concrete were subjected to 3-hour cycles of freezing and thawing. At the end of 0, 60, and 120 cycles, each specimen was again subjected to fracture toughness testing. Also, the mass loss ratio was measured at the end of every 30 cycles to further estioimate the degree of deterioration and evaluate the durability.

3 RESULTS AND DISCUSSION

3.1 L-CMOD curve

Figure 2 (a to c) shows typical L-CMOD curves from the normal strength (N) series. In regard to the original concrete (Fig. 2(a)), the post-peak reductions in the load-bearing capacity of short fiber series (N-S) are more ductile than those of N-00 with no fibers, showing improvement in toughness. The slow reduction is particularly significant for specimens designated S-ss having a small fiber diameter (large apparent number of fibers). It is therefore confirmed that the tendency toward large peak load and high toughness is proportional to apparent number of fibers.

The peak loads of all repaired concretes (Fig. 2(b), 0 cycles) are higher than those of original concretes, presumably because epoxy permeated not only into cracks but also in areas near the cracks, thereby increasing the structure's resistance to tension. It should be noted that new cracks mostly developed outward into unrepaired portions in Series N specimens, whereas most new cracks developed within the repaired portions in Series H specimens.

The peak loads of repaired specimens after 60 cycles of freezing and thawing (Fig. 2(c)) are significantly lower than those of original concretes. Specimens designated N-00, made of normal concrete with no fibers, and specimens designated N-Sl, having large fiber diameter, were fractured by the freeze-thaw action before being subjected to loading. The L-CMOD curves for these specimens were therefore immeasurable.

Note that the properties of all types of concrete samples inferred from the L-CMOD curves were confirmed as similar tendencies in TSDs.

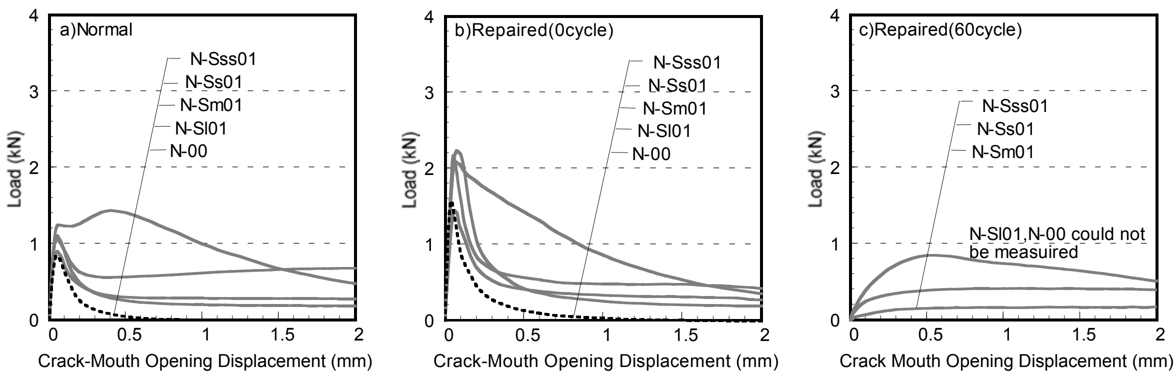

Figure 2. Example of L-CMOD curves of normal strength (N) series

Figure 3 (a to d) shows typical measured L-CMOD curves for fiber reinforced concretes. In regard to the long fiber series (Lm, 30 mm) in Figs. 2(a) and 2(b), the repair increased the peak load 1.5 times, while improving the toughness. This tendency is noticeable in Lm02 specimens with a high fiber content.

As shown in Fig. 2(c), the load-bearing capacities of repaired concretes subjected to freezing and thawing (60 cycles) tend to be lower than those of repaired concretes. However, Lm02 specimens with a high fiber content show strain-hardening type deformation, indicating their high toughness.

The peak loads of all specimens in the short fiber series (Sm, 12 mm) in Fig. 2(c) and 2(d) tend to increase similarly to Lm series, but these specimens are found to show no marked improvement in terms of toughness.

Note that the properties of all types of concrete samples inferred from the L-CMOD curves were confirmed as similar tendencies in TSDs.

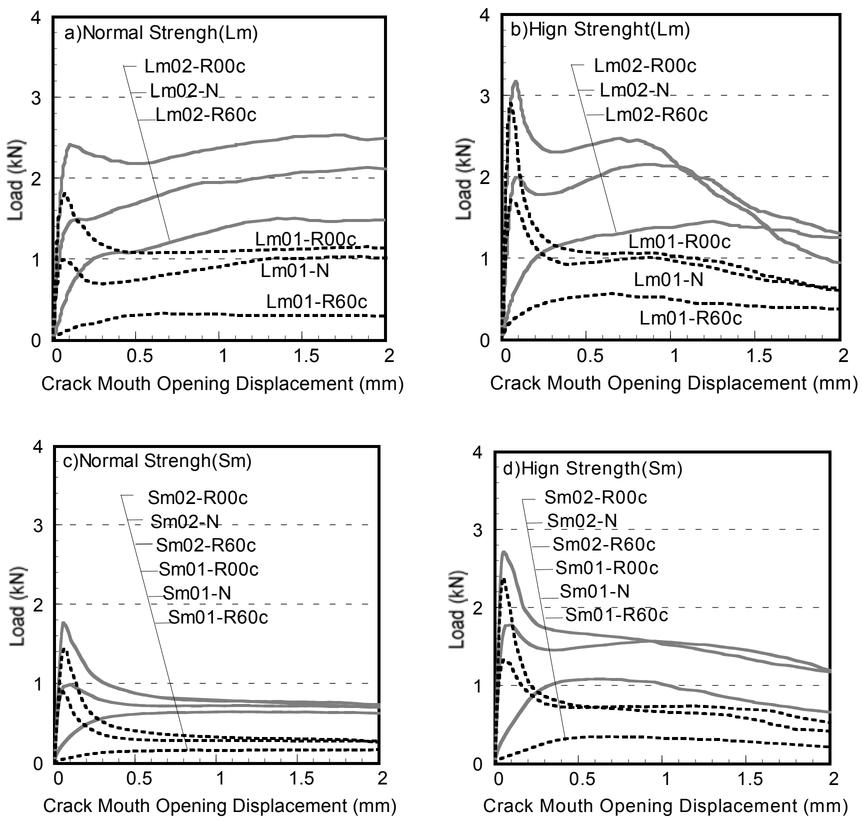

Figure 3. Examples of measured L-CMOD curves for fiber reinforced concretes

3.2 Initial cohesive stress

Initial cohesive stress, which refers to the cohesive stress when the crack opening displacement is 0 mm on the tension-softening diagram, is a value for the assessment of cracking strength. Figure 4 (a and b) shows the initial cohesive stress ratio related to the fiber content.

Figures 4(a) and 4(b) focusing on the fiber length (L and S) compare the effects of the fiber length, fiber content, repair, and freezing and thawing action with respect to the initial cohesive stress of the original concretes with no fibers (N-00 and H-00). The initial cohesive stress ratios of repaired specimens without freezing and thawing action (Rep0c) are found to increase to 1.5 to 2 times those of the original concretes regardless of the fiber content. After being subjected to freezing and thawing action (60 and 120 cycles), however, the values substantially decrease similarly to L-CMOD curves regardless of the fiber content. It is therefore inferred that it is difficult to retain the initial cohesive force, a force that serves to resist freeze-thaw action, by means of crack repair using epoxy resin.

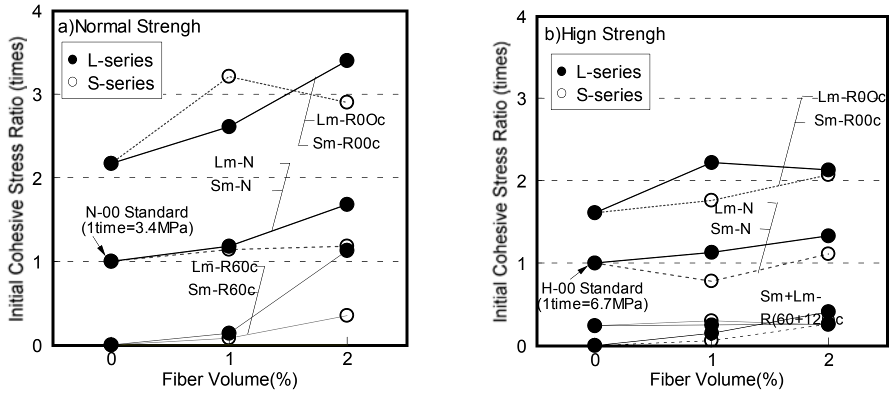

Figure 4. Initial cohesive stress ratio related to the fiber content

Figure 5 (a and b) shows the initial cohesive stress ratio related to the number of freeze-thaw cycles. These results show the strong effects of mortar strength, fiber length and fiber content on the decreasing tendency of the initial cohesive stress ratio as the number of cycles increases.

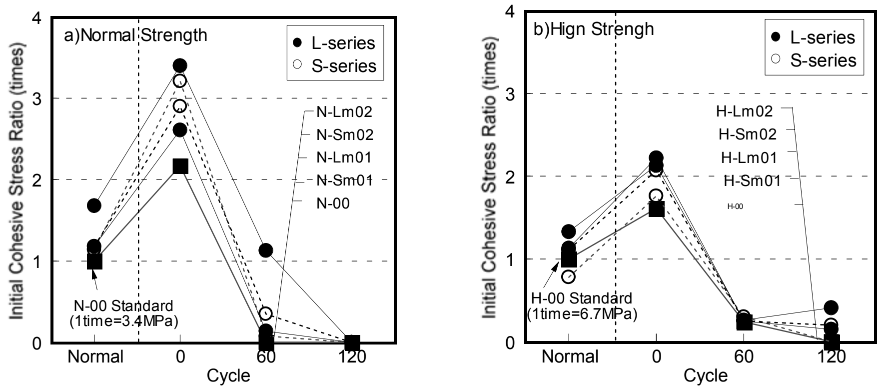

Figure 5. Initial cohesive stress ratio related to the number of freezing and thawing cycles

3.3 Effective fracture energy

Figure 6 (a and b) shows the relationship between the effective fracture energy ratio (G_F^u ratio) and the fiber content. In these figures focusing on the fiber lengths (L and S), the effects of fiber length, fiber content, repair, and freezing thawing action are compared in the form of G_F^u with respect to the effective fracture energy of the original concretes with no fibers (N-00 and H-00) similarly to the initial cohesive stress ratio. The G_F^u ratios of repaired specimens without freezing and thawing action (Rep0c) are found to be extremely higher than those of the original concretes, being nearly 20 times higher at the maximum with long fibers (Lm02) at a fiber content of 2%. This tendency is more significant with high strength concrete. This indicates the high potential for energy absorption of fibers in concrete. Energy absorption is also recognized accordingly with the fiber content even after freezing and thawing cycles. It can therefore be said that the toughness recovered by repair was retained through repeated freezing and thawing.

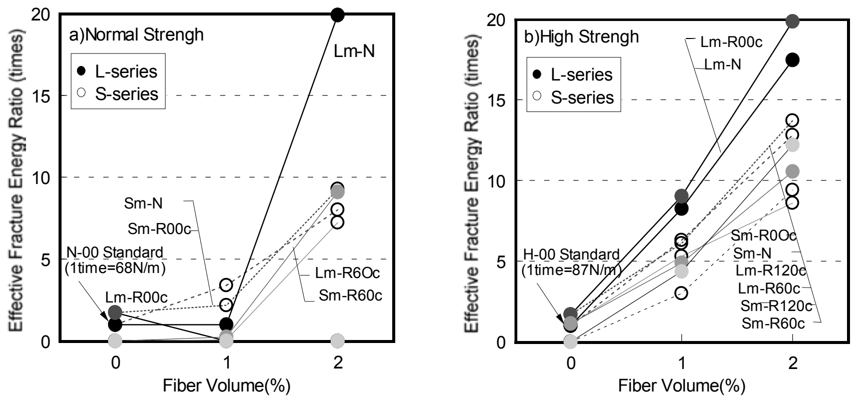

Figure 6. Relationship between the effective fracture energy ratio (G_F^u ratio) and fiber content

Figure 7 (a and b) shows the relationship between the number of freezing and thawing cycles and the G_F^u ratio. The fracture energy-retaining effect through the increasing number of cycles is found to be related to the strength, fiber length, and fiber content, and the values tend to decrease moderately when compared with the reductions in the initial cohesive stress ratio.

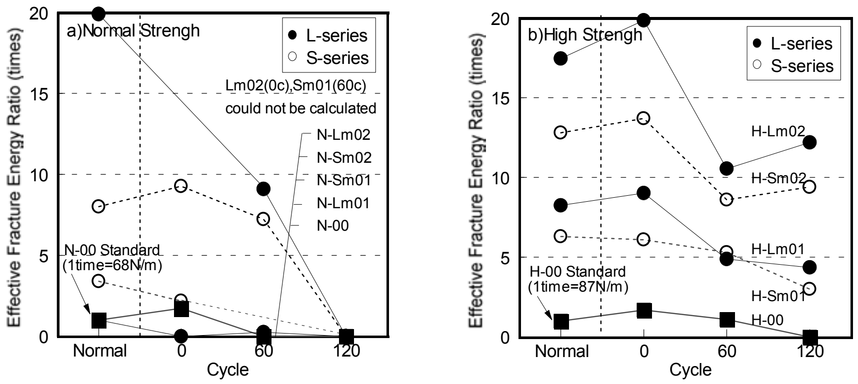

Figure 7. Relationship between the number of freezing and thawing cycles and G_F^u ratio

3.4 Durability-retaining effect

Figure 8 shows the mass loss ratios by freezing and thawing testing. In Fig. 8(a), the mass loss ratio is retained low in Series H regardless of the fiber length and content, but it begins to increase in Series N, with the durability decreasing accordingly. Since the mass loss ratio increases more significantly with longer fibers and higher fiber content, the fiber parameters are considered to affect the durability-retaining effect. Figure 8(b) focusing on the fiber diameter shows the durability-retaining effect of fibers to a certain extent regardless of the air content, which ranges from 1.7 to 2.6%, when compared with N-00. However, the mass losses become significant at around 60 cycles. It is therefore considered difficult to sustain the durability-retaining effect against freezing and thawing by changing the fiber diameter and content.

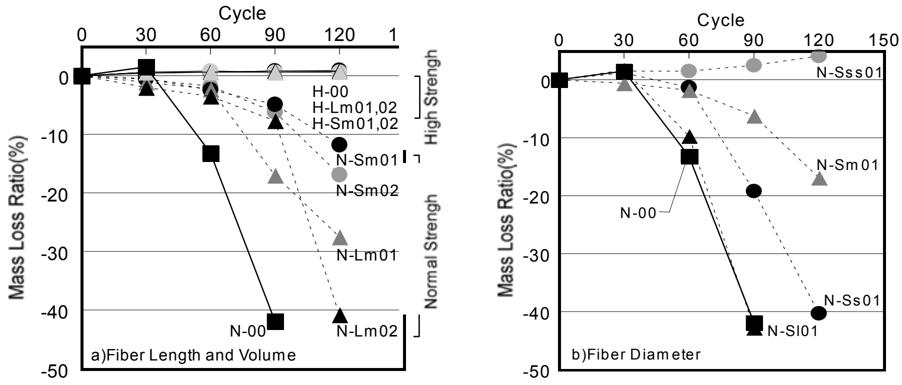

Figure 8. Mass loss ratios by freezing and thawing testing

4 CONCLUSIONS

The following has been elucidated from this study:

(1) Strength recovery by repair is satisfactory regardless of the inclusion of fibers. Toughness is also significantly improved by the addition of fibers. The degree of improvement to toughness becomes greater as the volumetric content of fibers increases, fiber diameter decreases and as fiber length increases, .

(2) Repair of concrete improves its initial cohesive stress regardless of the presence or absence of fibers, and certain improving effects are accordingly anticipated with the fiber content.

(3) Inclusion of fibers drastically improves the effective fracture energy of repaired concrete and strong improving effects are anticipated accordingly with the fiber content.

(4) When exposed to freeze-thaw action, the durability of repaired concrete is found to decrease concomitant with losses in its strength and mass, but its toughness is affected by the presence of fibers. A high content of long fibers produces a toughness-retaining effect.

ACKNOWLEDGMENT

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5 REFERENCES

Collins, F. & Roper, H. 1990, 'Laboratory investigation of shear repair of reinforced concrete beames loaded in flexure', *ACI Materials Journal*, **97**[2], pp.149-159

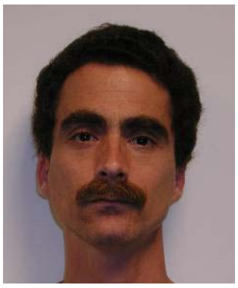
Basunbul, I. A., Gubati, A. A., Al-Sulaimani, G. J. & Baluch, M. H. 1990, 'Reinforced concrete beames', *ACI Materials Journal*, **87**[4], pp.348-354

RILEM AAC13.1 1994, 'Determination of the specific fracture energy and strain softening of AAC', RILEM Technical Recommendations for the Testing and Use of Construction Materials, pp.156-158

Kitsutaka, Y. 1997, 'Fracture parameters by poly-linear tension softening analysis', *Journal of Engineering Mechanics*, ASCE, **123**[5], pp.444-450

Kitsutaka, Y., Kamimura, K. & Nakamura, S. 1994, 'Evaluation of aggregate properties on tension softening behavior of high strength concrete'. High Performance Concrete, ACI SP149-40, pp.711-pp.727

Conservation Repair Approaches in the United States: Don't Close the Door to the Future



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ABSTRACT

Private individuals or corporation own and maintain much of the historically significant architecture in the United States. For these owners, conservation of historic buildings is often a secondary concern after economic considerations. As the significance of commercial building stock in the United States increases, conservation specialists must address issues of conservation that satisfy the owner's requirement, as well as, conservation theory.

All too often repairs to historic buildings have been or are being designed and executed without consideration for future conservation. While numerous examples of aesthetically insensitive repairs exist, less noticeable, but equally significant, are repairs that are installed without consideration for future evaluation, repair and maintenance. Buildings in private ownership are more likely to change ownership or management. Private owners rely on external experts to direct building evaluation, repair and maintenance more often than governmental agencies or institutions. When a change in management occurs, it is often associated with a change in conservation experts directing the maintenance and conservation of the building. In addition, the drawings and specifications related to previous repair work may be discarded or not transferred to the new management. As a result, if unusual repairs are made or can not be understood without removal and loss of historic fabric by future conservation professionals, the repair must be replaced even if the repair may be technically sound; since, the new conservation professional responsible for the care of the building and public safety is unable to verify the repair.

The following paper will provide an overview comparision of preservation philosophy in the United States and Europe. A case study will be presented of the ongoing restoration of a culturally significant 1870s building located in Chicago, Illinois. This case study will demonstate the impact of previous interventions and the current approach for restoring the facade of the building.

KEYWORDS

conservation, preservation, masonry, cast iron

1 INTRODUCTION

While conservation professionals in Europe may address issues of buildings that are hundreds or even thousands of years old, in the United States few western influenced buildings are more than 200 years old. These modern buildings often have more complicated structural systems mixing materials. Further, the culture of conserving, or at least repairing old buildings, in Europe has existed for hundreds of years while in the United Stated conservation gained significant public support only in the 1960s and 1970s. With some notable exceptions, repair of significant buildings was often insensitive, inappropriate and inconsistent. Worse yet, many of the uniquely American buildings were demolished following World War Two for the machine aesthetic of the modern movement.

The case study building presented below provided a range of issues and conditions which had to be evaluated and addressed by the authors in the ongoing restoration of the building's facade.

2 CULTURAL HERITAGE CHARTERS AND STANDARDS

The first attempt to establish a coherent and logically defensible philosophy for building conservation is found in the Society for the Protection of Ancient Building's (SPAB) Manifesto of 1877 [Morris]. The Manifesto consists principally of a plea to "put protection in place of restoration". This manifesto marked the starting point for the many later policy statements in which the underlying theme of the manifesto is adopted and developed rather than being significantly amended.

> *For what is left we plead before our architects themselves, before the official guardians of buildings, and before the public generally, and we pray them to remember how much is gone of the religion, thought and manners of time past, never by almost universal consent, to be Restored; and to consider whether it be possible to Restore those buildings, the living spirit of which, it cannot be too often repeated, was an inseparable part of that religion and thought, and those past manners.* [Morris 1887]

The Athens Conference of 1931 [US/ICOMOS 1999], established basic principles for an international code of practice for conservation. The Second International Congress of Architects and Technicians of Historic Monuments, which met in Venice in May 1964 [US/ICOMOS 1999], approved the Venice Charter. The Venice Charter stresses the importance of setting, respect for original fabric, precise documentation of any intervention, the significance of contributions from all periods to the building's character, and the maintenance of historic buildings for a socially useful purpose. These charters and standards are intended to provide guiding principles towards defining an appropriate response to particular conservation issues, not as instant and all-inclusive prescriptions.

In the United States, the Society for the Preservation of New England Antiquites promoted a philosophy similar to SPABs philosophy. Unfortunately, these ideals were not widely embraced by the public or design professionals. In the United States, the federal government wrote and promoted the Secretary of Interior's Standards for the Treatment of Historic Properties [National Park Service 1992]. While these standards reference previous domestic and international guidelines, the Standards were established to regulate the conservation of federally funded public projects or private projects that receive tax incentives. These standards established an implied priority of interventions with the most desirable approach being one of conservation/preservation, followed by rehabilitation, restoration and the least desirable being reconstruction.

"Preservation is defined as the act or process of applying measures necessary to sustain the existing form, integrity, and materials of an historic property." [NPS 1992]. Preservation respects the historic material and enables the structure to continue to age. "Rehabilitation is defined as the act or process of making possible a compatible use for a property through repair, alterations, and additions while preserving those portions or features which convey its historical, cultural, or architectural values."

[NPS 1992]. Rehabilitation alters the continuum by the addition of elements. "Restoration is defined as the act or process of accurately depicting the form, features, and character of a property as it appeared at a particular period of time by means of the removal of features from other periods in its history and reconstruction of missing features from the restoration period. " [NPS 1992]. Restoration moves the structure back to a previous condition and therefore also alters the timeline. "Reconstruction is defined as the act or process of depicting, by means of new construction, the form, features, and detailing of a non-surviving site, landscape, building, structure, or object for the purpose of replicating its appearance at a specific period of time and in its historic location ." [NPS 1992].
Reconstruction effectively eliminates the original time line and introduces a completely new element which is a gesture to a past element [Murtagh 1997].

Perhaps the interpretation (or misinterpretation) of the Secretary of Interior's Standards for Rehabilitation has in part contributed to the approach in the United States of matching and blending replacement materials with the adjacent original material:

> *Deteriorated architectural features shall be repaired rather than replaced, wherever possible. In the event replacement is necessary, the new material should match the materials being replaced in composition, design, color, texture and other visible qualities.*[NPS 1992]

This contrasts to Venice Charter which states:

> *Replacements of missing parts must integrate harmoniously with the whole, but at the same time must be distinguishable from the original so that restoration does not falsify the artistic or historic evidence.* [US/ICOMOS 1992]

Both address intervention programs in a similar manor:

> *If an interpretation program involves the introduction of new materials or change to the physical setting of a cultural heritage site, these alterations should be reversible and removable without leaving permanent traces* .[US/ICOMOS 1992]

European and United States philosophical approaches to the actual interventions are consistent as they relate to reversibility, retreatability, and use of traditional techniques. All repair approaches should be designed to be removed or replaced in the future when implementing better techniques and materials is possible if and when they become available [Croci 1998]. The use of traditional techniques and materials is desirable but also a recognition of the tested ideas and systems.

In the United States, even buildings recognized as National Landmarks are provided no protection unless they are also locally designated. Many local landmark ordinances require owners approval prior to landmarking. As a result, some of the most important buildigns have no protection or oversite when it comes to protection their integrity. Also, in the United States there is a lower emphasize placed on the age value of buildings. As a result, a greater emphasis on restoration over a conservation approach is often demanded by private clients. As a result, materials are largely choosen to match and blend into the existing and only sometimes upon close inspection can new material be identified.

In the conservation of heritage buildings, the conservation professionals responsible for decisions and public safety often change especially for buildings in private ownership. Each conservation professional involved with a building must feel certain that the conditions of the building do not threaten the occupants or public. To ensure this level of comfort, each new responsible conservation professional must understand the theoretical approach taken, the technical detailing, and the scope and scale of previous repairs. Each effort to gain this knowledge may lead to inspection openings and corresponding destruction of historic fabric. Repairs should be well documented and as transparent as possible. The following case studies offer repair choices adopted and how it is hoped these repairs will be obvious to future conservation professionals.

3 CAST STUDY: PAGE BROTHERS BUILDING, CHICAGO ILLIONIS

Located at the intersection of two of Chicago's major commercial streets, the Page Brothers Building, shown in its current state in Figure 2 is located on the former site of Stiles Burton's City Hotel. After the fire of 1871 destroyed the building, Burton immediately redeveloped the lot. For this task, Burton hired John Mills Van Osdel, among Chicago's first professional architects, to design the Page Brothers Building. Van Osdel (1811-1891) was responsible for designing many buildings in nineteenth century Chicago [Chicago Landmarks 1996].

The Page Brothers Building was originally five stories in height, clad with cast iron on the north facade, and brick masonry on the remaining facades. The west facade faces State Street and the north facade faces Lake Street. At the time of construction, Lake Street was the predominant commercial street in Chicago, and the architecture of the building illustrates this by having the north facade much more ornate than the utilitarian west facade. As originally designed, the building was of semi-mill construction consisting of load bearing masonry walls and cast iron interior columns supporting a heavy timber floor structure [National Register Nomination 1975]. A cornice, frieze and ornamentation accented both the west and north facades. An engraving shown in Figure 1, illustrates the original configuration of the building.

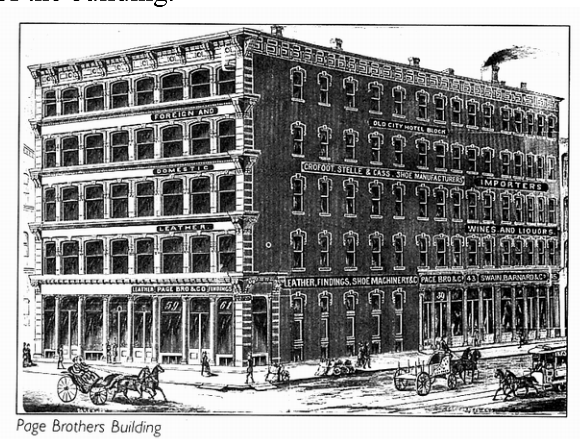

Figure 2. Original engraving of Page Brothers Building circa 1880s viewed from the northwest.

3.1 Alterations

3.1.1 1901-1902 Renovation

In 1901–1902, the building underwent a significant remodeled by the architecture firm Hill & Woltersdorf [Chicago Landmarks 1996]. Alterations of the building included the addition of a sixth floor and a redesigned west facade. The building changed from commercial to office uses and was reoriented to make the State Street facade its primary facade.

A new cornice and frieze was installed so that it extended over both the north and west facades. An entirely new facade supported by steel framing was constructed on State Street along with extensive interior alterations. The windows on the west facade were significantly enlarged. WJE's investigation tends to indicate that the original face brick on the west façade was removed and replaced with the current buff colored bricks.

In additional to the exterior work, significant structural alterations were made to the internal structure of the building. A new office entry and lobby was installed at the extreme southern end of the State Street facade, and a central elevator and staircase was installed. All original wood staircases from original construction were probably removed during the 1901–02 renovation work and replaced.

3.1.2 1986 Renovation

A major rehabilitation of the Page Brothers Building was again performed in 1986 in conjunction with the rehabilitation of the Chicago Theatre that abuts the south and east facades of the Page Brothers Building. The work on the Page Brothers Building included cleaning, repairing, and repainting the existing cornice, frieze and cast iron, cleaning; repairing and repointing all masonry surfaces; installing new window sashes into the existing frames; and adding a mechanical penthouse above the sixth floor. The first floor was also significant altered with the installation of new large storefront windows. New Corinthian column capitals were made to replace the existing or missing capitals and metal clad wood panels were installed at various areas. A new metal fascia was also to be added between the floors. This work also included replacement of the remaining heavy timber floor framing elements with a new cast-in-place reinforced concrete floor system supported by reinforced concrete columns. The new concrete system was placed directly on the original structure using the original wood framing as a formwork for the concrete. The original timber members were subsequently removed.

3.2 Assessment of Previous Repairs and Alterations

Based on the age of the building as well as the complexity of the facade systems much of the repair work that has been performed over its 140 years of service has not been performed with an overarching goal of preserving the cultural heritage of the building. Much of the exterior work has been performed as short-term solutions to address visible problems and public safety. The interior work has also been shortsighted solutions to tenant needs and code requirements.

The recent evaluation of the Page Brothers facades was initiated to comply with facade inspection requirements defined by the Chicago Building Code for buildings over six stories tall. The intent of the inspection is to assist building owners in maintaining their building and prioritizing required repairs and maintenance. Conditions observed and determining appropriate sensitive recommendations were complicated by the previous alterations and maintenance programs.

Previous repairs to the cast iron façade generally were stopgap approaches. While some of the repairs were readily apparent, such as strapping cast iron components, other repairs were much more difficult to detect such as filling cast iron decorative components with concrete or adhering loss pieces with sealant. Further, previous repair programs or routine maintenance had included removal of presumably deteriorated cast iron belt courses at the intermediate floors with no documentation of the components that were removed.

Masonry repairs were typically smear coats of cement-based mortars over deteriorated joints and sandblasting cleaning techniques. These repairs made an already soft brick much more susceptible to moisture infiltration and associated damage. The interface condition and detailing between the face brick façade, installed in 1903, and the original backup masonry were difficult to evaluate since the repair work was not documented. Connections between the face brick and backup brick, based on the inspection and probe openings appeared to be limited to intermittent inconsistently spaced paired header bricks pocketed into the back up brickwork.

The 1986 restoration resulted in loss of the historic structural system and is difficult to evaluate as it relates to the overall behavior of the building facades. The interface of the new cast-in-place concrete and the cast iron and masonry facades were not clearly detailed in the architect's drawings and could not be readily assessed without removing significant portions of the façade. Fortunately, however the architect placed the repair documents into a public archive. Though limited in detail, the information that was included in the drawing was very helpful in gaining a general understanding of the scope of repairs.

Figure 3. View of building from northwest prior be beginning current restoration program.

3.3 Current Restoration Program

Conditions observed during the inspection prompted developing a phased restoration for the façades of the building. The primary goal of the repair and restoration work was to address public safety concerns while maintaining, retaining or replacing as much of the historic fabric of the facades as possible or necessary. The project will be phased over a two to three year period based on available funds. Prioritization of the repair work will obviously focus first on public safety followed by maintenance and finally aesthetics with an overarching philosophy of preserving as much of the historic fabric as possible and replacing material in kind or with appropriate substitute materials as recommended by the Secretary of Interior Standards.

3.3.1 Masonry Facade Repairs

The scope of recommended repair work on the main walls is intended to repair existing areas of distress; repoint mortar joints; install new stainless steel anchors in selected areas of the exterior wall to supplement existing anchors; and improve the long-term performance of the facade.

Steel Lintels: The current condition of the steel lintels supporting the brick masonry above window openings is generally good. However, because the steel is unprotected except for the primer that was applied during original fabrication, it is susceptible to corrosion. To minimize the visual impact of protecting the steel members, steel angles will be exposed and all surfaces will be treated with a high quality protective coating system. In addition, a flashing system will be installed to protect the angle and direct moisture to the exterior of the building.

Lateral Anchorage: Based on the investigation, the lateral anchorage of the existing face brick to the backup masonry did not comply with current code requirements or historical standards. Header bricks were found to be intermittently and inconsistently spaced. Several in-situ anchorage systems were considered including the installation of stainless steel helical anchors and epoxy anchors from either the interior or exterior. The installation from the interior was ruled out due to tenant disruptions and inaccessibility to the masonry in front of structural beams and columns. The installation of the system from the exterior proved to be unfeasible due to the butter mortar joints and the condition of the spandrels necessitated disassembly and reconstruction due to instabilities that resulted when repointing the butter joints was attempted. The first phase of spandrel repairs included reconstruction of the spandrels using the original brick, lime putty mortared butter joints and new stainless steel lateral anchors. The new anchor system is readily detectable from the exterior with a metal detector and did not necessitate drilling through the historic brick.

Replacement Brick: A limited number of brick were damaged to a point that nessitated procuring replacement brick. The philisophical dilema of matching sandblasted brick or matching undamaged brick was exhaustively discussed. Generally in the United States, the goal of replacement material is to match clean original material. The difficulty was in deciding to match the sandblasted clean brick or the unsandblasted clean brick. The sandblasting had signficantly changed the appearance of the brick, thus replacement material which matched undamaged brick would have been too noticable for the client and the public in the United States. The decision was made to group all new brickwork together on spandreds which were completely rebuilt with replacement brick at the top of the building. By not mixing the historic brick with the new brick, the old and new will be more easibily distibguished upon a close inspection of the building. Excess original brick was not mixed with replacement brick and was saved for future replacement of damaged brick at other areas of the façade.

Repointing Mortar: Given the age of the facade, the compressive strength of the historic brick is low and variable when compared to modern brick. Further, the previous cleaning that has abraded the surface of the brick has made the surface much more fragile than a brick that has not been damaged by abrasive cleaning techniques.There were also multiple non-matching portland cement based pointing mortars visible at the building. The preferential deterioration of the mortar joints rather than the brick masonry units is desirable by allowing deterioration to occur in a material that is easily maintained.

Given the historic significance of the building and the complexity of material and aesthetic issues described above, prepackaged lime putty and sand mortar was used for masonry work on the masonry facade of the building. Use of a traditional lime putty mortar formulation was determined to be consistent with the history of the building. Use of a premixed prepackaged mortar provided greater quality control and consistency of the mortar color. Further, since lime putty is softer than conventional cement based mortars, it is better able to accommodate small movements in the brick masonry and should minimize cracking in the narrow butter mortar joints.

*Crack Repairs:*Cracking observed that was not directly attributed to the corrosion of the steel lintels over windows will be repointed with lime mortar putty. Rather than installing conventional expansion joints to accommodate thermal movements of the exterior masonry, the cracks will be monitored for future cracking as evidence of on-going building movements. Filling the cracks with sealant was not considered due to the negative visual impact. In addition, the physical properties of the lime putty mortar will accommodate more movement than cement based mortars which should also minimize minor cracking.

3.3.2 Cast Iron Facade Repairs

The scope of recommended cast iron repair work on the main walls is intended to repair existing areas of distress, repair damaged areas, and replace missing or significantly damaged elements.

Patching and Repairs: Joints between individual cast iron pieces were typically caulked to prevent water infiltration. This tended to trap water within the cast iron and cause rusting from the inside out. All sealant joints between panels and adjacent materials will be removed and replaced with new silicone sealant. Installation of sealant should be adequate to repair non-structural cracks; however, brazing was used to repair some of the cracked decorative pieces since the operation could be performed off site reducing the fire potential. At the time of our inspection, no significant structural cracks were observed in the panels that remained on the building. All structural cracks will be repaired by in-situ brazing and non-structural cracks will be filled with sealant. Additionally, there are isolated holes in portions of the cast iron. These holes will be filled with epoxy-based putties and painted. All pieces that were removed were repaired and all surfaces were painted with a corrosion inhibiting system.

Replacement: Some of the smaller pieces from the building were severely damaged or missing and had to be replaced. All fasteners used to anchor the small cast iron pieces to the larger panels will be replaced with new stainless steel anchors. New holes will be drilled and tapped into the cast iron. The new fasteners will be galvanically isolated from the cast iron to reduce the potential for galvanic interaction. By using a threaded fastener rather than a toggle type anchor, the decorative components, if necessary, can be removed in the future and reinstalled without significant damage caused by cutting.

The non-original fiberglass decorative column capitals at the first floor will be removed and replaced with new cast iron capitals. Though the condition of the capitals at this time is serviceable, the long-term durability of the current capitals is unlikely. Further, the use of fiberglass elements is generally not considered a good substitute material for historically significant buildings.

3.3.3 Sheet Metal Cornice, Frieze, and First Floor Storefront Repairs

The scope of recommended sheet metal repair work on the main walls is intended to repair damaged areas and replace missing or significantly damaged elements.

New ornamental sheet metal will be attached to the existing galvanized steel backing and new supplemental elements with stainless steel rivets. This method of attachment is standard in the sheet metal industry today and is much simpler for field application than soldering.

The existing color of the cornice and frieze were historically painted with the same color as the cast iron portion of the façade, which was an off-white color, intended to replicate limestone. Subsequent painting campaigns have changed the color scheme primarily to a dark green color. The final phase of the restoration work will include repainting the sheet metal and cast iron components the original off-white color.

4 CONCLUSION

Limited funds necessitated implementation of a multi year phasing of the restoration of the Page Brothers building. Initial interventions focused on addressing significantly distressed on the west masonry facade and life safety issues on the north cast iron facade. Subsequent phases will address returning the building facades to the 1900s aesthetics. This will include recreating historic cast iron ornamental elements, replacing missing sheet metal elements in the cornice, masonry repairs and finally replication of the cast iron storefronts and replicating the ‘original’ color scheme of the facade. The Page Brothers building provides a wide range of conservation approaches that have been implemented over the past 100 years and reflect many significant philosophical issues when considering a comparison of approaches used for a building in the United States verses a European approach.

Both standards in the United States and Internationally emphasize the role of documentation. As litigation risks continually increase in the United States, there is more of an incentive to destroy drawings, specifications, and reports after the ten year statute of limitations has expired rather than donating these documents to a public archive. This is true for both the Owner and the design professionals. Further, with the ever increasing security risks few owners will allow drawings and reports detailing the floor plan and construction of their buildings in a public archive. As neither of these trends appear to be flexible or changing, there is an increased need to make repairs transparant for the longterm conservation of privately owner histroic buildings.

5 REFERENCES

Commission on Chicago Landmarks. Chicago Historic Resources Survey, 1996.

Croci, Giorgio 1998, *The Conservation and Structural Restoration of Architectural Heritage*, Computational Mechanics Publications, UK and Boston, pp 80-81.

TT3-93, Conservation Repair Approaches in the United States:Don’t close the door to the future
E.Gerns and J.Freedland

National Park Service 1992: Secretary of Interiors Standards for the Treatment of Historic Properties; Introduction: Choose an Appropriate Treatment for the Historic Building. http://www.cr.nps.gov/hps/tps/standguide/overview/choose_treat.htm [Accessed 22 February 2005]

National Register of Historic Places Nomination, 1975.

Morris, William 1877, Society for the Protection of Ancient Buildings (SPAB): The Manifesto. http://www.spab.org.uk/whatis_manifesto.html [Accessed 22 February 2005]

Murtagh, William 1997, *Keeping Time: The History and Theory of Preservation in America*, John Wiley & Sons, New York, pp. 184-188.

US/ICOMOS Scientific Journal, Volume 1, Number 1, 1999, ICOMOS Charters and Other International Doctrinal Documents, pp. 7-8 (Charter of Venice). http://www.icomos.org/venice_charter.html#conservation [Accessed 22 February 2005]

Study of the Quality Improvement of Fly Ash Concrete with Durability Improving Admixture

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ABSTRACT

The purpose of this research was to improve the durability of fly ash concrete. We focused on NonAE concrete, a non-air entraining (NonAE) type admixture that improves durability by reducing shrinkage in the drying process and is freeze-thaw resistant, and examined the impact of this admixture on the improvement of durability, presence or absence of air content, water-binder ratio and impact on fly ash replacement ratio. Results showed that by making fly ash concrete from non-air entraining concrete the defects of fly ash concrete—reducing initial compressive strength and increasing carbonation speed—are greatly improved. Also, taken into consideration were the effects of changes in the curing methods and curing periods of preliminary curing in the accelerated carbonation test. In addition, by using the durability improving admixture, drying shrinkage was reduced about 60% compared to not using the durability improving admixture, as well as resolving the freezing-thawing problem with NonAE concrete where ordinarily stripping 24 hours after casting, concrete specimens were under a constant temperature of 20°C and relative humidity of RH60%, initially cured in water and immediately afterwards subjected to a freezing-thawing test. We confirmed that by allowing the mixture to cure in air before conducting the freezing-thawing test, even NonAE concrete freezing resistance remarkably improved. We examined the impact of durability improving admixture from the aspect of the air void system of hardening concrete and we presumed that the action mechanism reduces drying shrinkage and improves freeze-thaw resistance.

Finally, by making fly ash concrete from NonAE concrete, the difficulty of controlling air content in fly ash concrete is reduced and quality management is simplified.

KEYWORDS

Fly Ash, Air Content, Durability Improving Admixture, Durability

1 INTRODUCTION

For a stable power supply, as an alternative to petroleum following nuclear enargy, Japan uses coal because it is excels in dependability and economy. In the year 2006, coal-fired power plants are planned to produce 38,480,000 KW of electric power, increasing to 43,940,000 KW in 2011, and is expected to release into the atmosphere a corresponding output of 10,000,000 tons of coal ash by the end of 2007. In order to form recycling society, over a long period many studies have been made to find ways to make efficient use of this fly ash from coal in the construction field, but in actuality Japan is still not using it much in building construction. The chief reasons for this are the decreased level of the initial compression arising from the low activity in fly ash and increased carbonation velocity due to a decrease in pH from the self-carbonation of pozzuolanic action. Also, because of the large amount of AE agent absorbed by the unburned carbon in fly ash it is difficult to regulate air quantity, and maintaining serial stability of air content is a problem. In Japan, to ensure resistance to freeze-thaw action it is necessary to mix the proper amount of air in the concrete. The chief reason for the difficulty of quality management of fly ash concrete is the influence exerted by fly ash on the quantity of air needed to maintain quality. Against the above background, in our experiments, with the purpose of improving fly ash concrete durability we focused on reduced drying shrinkage, improved freeze-thaw resistant non-air entraining type durability improving admixture (Glycol ether derivative) in NonAE concrete, and examined the extent of its influence on durability improving admixture,the presence of air, water-binder ratio and fly ash replacement ratio.

2 EXPERIMENT OVERVIEW

Explanations are given in the following tables: Table 1, materials used; Table 2, quality of fly ash; Table 3, experiment factors and standards; Table 4, concrete composition; Table 5, symbols; and Table 6, experiment items. In this experiment, fly ash (FA) was used to replace the internal mass of the basic mixture and 10 kg per 1m^3 of durability improving admixture was added to the final mixture. AE water reducing agent was added to the total mixture, making 1% of the binder weight. Standard air entraining concrete (ST) using air entraining admixture for FA regulated a fixed air content. Materials with durability improving admixture (D), because of strong anti-foam reaction in the admixture itself, become NonAE and other NonAE materials (PL) regulate a fixed air content by anti-foam reaction. The targeted performance for both ST and NonAE concrete was set at slump 18±1 cm.

Table 1. Materials used

Cement	Ordinary portland cement Density:3.16g/cm^3, Blaine's value:3283cm^2/g
Binder	Fly ash Type□(JIS A 6201)
Fine aggregate	Mountain sand from Kakegawa Specific gravity under oven-dry:2.56g/cm^3, Coefficient of water absorption:1.72%, F.M.:2.67
Coarse aggregate	Crushed stone from Oume Specific gravity under oven-dry:2.64g/cm^3, Coefficient of water absorption:0.51%, F.M.:6.76
Admixture	Air entraining and water reducing admixture:Normal type Class□ Air entraining admixture for fly ash:Non ionic surface active agent Durabirity improving admixture: glycol ether derivative(Non-water solution)

Table 2. Quality of fly ash

Ignition loss [%]	*Density [g/cm^3]*	*Blaine's value [cm^2/g]*	*MB absorption [mg/g]*
1.4	2.24	4160	0.51

Table 3. Experiment factors and standards

Experiment factors	*Standards*
Fly ash replacement ratio	0%, 15%, 25%
Water-binder ratio	60%, 50%, 43%
Air content	1.5±0.5%(D and ST), 4.5±0.5%(ST)
Durability improving admixture	0kg/m^3, 10kg/m^3

Table.6 Experiment items

Experiment items	*Method of test*	*Curing methods*
Compressive strength	JIS A 1108 - 1999	Curing in water* [7 and 28 and 91days]
Static modulus of elasticity	JIS A 1149 - 2001	Curing in water* [28days]
Splitting tensile strength	JIS A 1113 - 1999	Curing in water* [28days]
Length change of concrete	JIS A 1129-1 - 2001	After initial curing in water* for periods of 7 to 28days. The specimens(10×10×40cm) were under constant temperature of 20□and relative humidity of RH 60%.
Accelerated carbonation test	JIS A 1152 -2002 JIS A 1153 - 2003	After initial curing the specimens(10×10×40cm) were laid in enviroments of CO_2 cocentrations 5%(accelerated carbonations). Measurement of carbonation depths was done by splitting specimens ,and spraying the surface with 1% phenolphthalein solution and taking the uncolored areas as carbonated portions. Initial curing methods □:After water*curing 28days,curing in air**28days.(JIS A 1153) Initial curing methods □:After curing in water*28days, curing in the seal***63days , Finally curing in air*28days. Initial curing methods □:After water*curing 28days.curing in air**91days. Initial curing methods □:After water*curing 7days.curing in air**91days.
Resistance of concrete to rapid feezing and thawing	ASTM C 666-97 JIS A 1148-2001 Procedure A	Measurement of fundamental transverse frequency:300cycles Specimens size: 10×10×40cm Initial curing methods A :Water*curing 28days. Initial curing methods B :After water*curing 28days.curing in air**14days. Initial curing methods C :After water*curing 28days.curing in air**28days.
Microscopical determination of parameters of the air-void system in hardened concrete	ASTM C 457-98	After initial curing in water*28days, curing in air**28days.

* Under constant temperature of 20□,** Under constant temperature of 20□and relative humidity of RH 60%.
*** After sealing put specimens into Plastic-film bag, curing in air**.

Table 4. Concrete composition

symbols	*W/B*	*W/C*	Unit content[kg/m³]					
			W	*C*	*FA*	*S*	*G*	*D*
Af0-60PL	0.60	0.60	182	303	0	856	1014	-
Af0-60ST			170	283	0	813	1014	-
Af15-60D		0.71	166	235	42	880	1014	10
Af15-60ST			166	235	42	815	1014	-
Af25-60D		0.80	164	205	68	879	1014	10
Af25-60ST			167	209	70	801	1014	-
Af0-50PL	0.50	0.50	182	364	0	806	1014	-
Af0-50D			174	348	0	814	1014	10
Af0-50ST			170	340	0	766	1014	-
Af15-50PL		0.59	174	296	52	823	1014	-
Af15-50D			166	282	50	831	1014	10
Af15-50ST			166	282	50	766	1014	-
Af25-50PL		0.67	172	258	86	820	1014	-
Af25-50D			164	246	82	829	1014	10
Af25-50ST			167	251	84	751	1014	-
Af0-43PL	0.43	0.43	189	440	0	726	1014	-
Af0-43ST			178	414	0	684	1014	-
Af15-43D		0.51	173	342	60	752	1014	10
Af15-43ST			176	348	61	673	1014	-
Af25-43D		0.57	171	298	99	748	1014	10
Af25-43ST			177	309	103	655	1014	-

W/B:Water-binder ratio, *W/C*:Water-cement ratio
W:Water, *C*:Cement, *FA*:Fly ash, *S*:Fine aggregate
G:Coarse aggregate, *D*:Durability improving admixture

Table 5. Symbols

Af 15 – 50 D ← Sort of concrete[PL, D, ST]
Water-binder ratio[%]
Fly ash replacement ratio[%]

3. HARDENED CONCRETE EXPERIMENT RESULTS AND CONSIDERATIONS

3.1 Compressive Strength Experiment Results

Fig.1 shows the manifestation of W/B=50% compressive strength. Regardless of the amount of air, whether or not there is durability improving admixture or W/B, with the addition of FA replacement ratio the initial compressive strength deteriorated but based on a 28 day criterion for material aging, in 91 days the compressive strength rose. Fig.2 shows the relation between binder-water ratio and compressive strength, and cement water (C/W) ratio and compressive strength. Since there was no difference noted between the compressive strength of PL and D NonAE concrete, in the diagram both are shown by the same approximation line. Until day 28 of aging, a high correlation was shown between C/W and compressive strength, regardless of FA replacement. After 28 days, pozzuolanic action occurs and FA contributes to the strength of the material and C/W correlation weakens. There was high correlation between B/W and compressive strength over a longer period of one year, but this was not realized when assessing its strength in the shorter span of 91 days. Also, the effect of the presence of air, NonAE of the same standard C/W ST, compressive strength ratio, was 10% higher, and to produce the same compressive strength ratio, NonAE W/C was 5% higher than hardened concrete W/C. the correlation between B/W and compressive strength in the same replacement ratio is high, and in the same B/W there is less difference in strength depending on the presence or absence of air water-binder ratio and at W/B=50 or 43%, NonAE is 10-20% stronger.

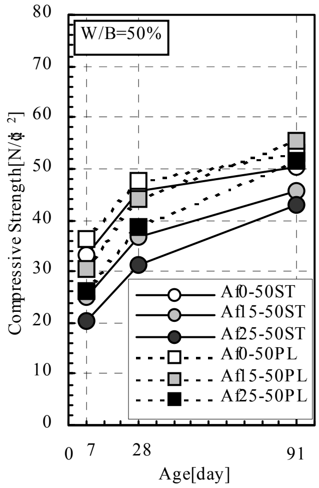

Fig.1 Compressive strength

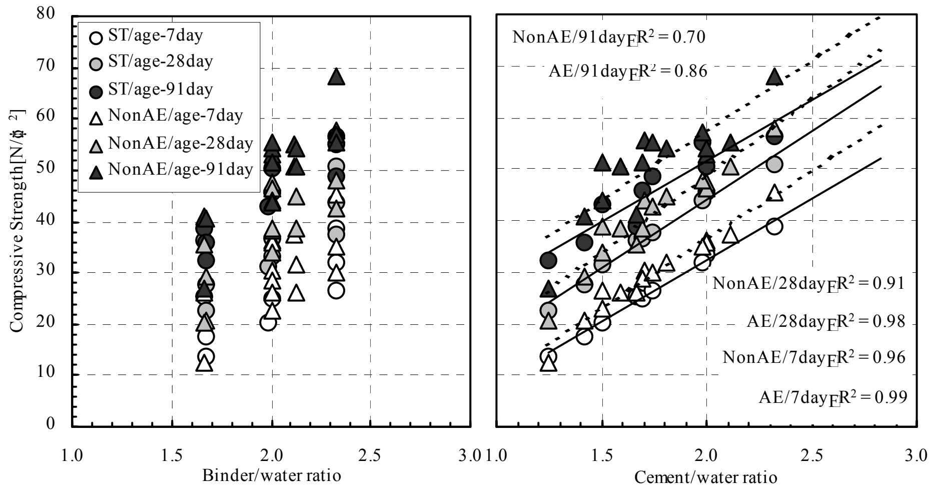

Fig.2 Binder cement ratio and cement water ratio, and compressive strength

3.2 Results of Experiments with Tensile Strength Binder and Static Elasticity Coefficient

Fig.3 shows the relation between compressive strength and tensile strength, and Fig.4 shows the relation between compressive strength and the modulus of elasticity coefficient. The influence of FA replacement ratio and the presence of air and durability improvement mixture are not seen in either tensile strength or the modulus of elasticity coefficient but both depend on comprehensive strength.

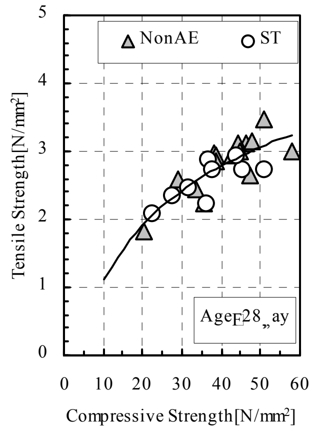

Fig.3 Compressive strength and tensile strength

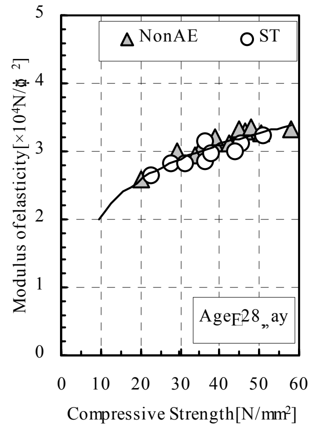

Fig.4 Compressive strength and modulus of elasticity

3.3 Drying Shrinkage

In Fig.5, the change in the material length of 13 week-old materials (water cured for 1 week), and in Fig.6, the change in length in 50% water-binder ratio are shown. Upon examining Fig.5 and 6, because of the difference in the amount of air given during the drying shrinkage period in Non AE materials there is slightly less shrinkage. From Fig.6, it is observed that the longer the pre-curing period the less shrinkage. On the other hand, when durability improving admixture is used it has a significant effect in reducing drying shrinkage, repressing change in material length by 60% compared to materials where the admixture is not used. Fig.7 shows the relation between deterioration ratio in quality mass and drying shrinkage. This is shown in the unit volume percentage of cement paste used in patching up defects accompanying escaping moisture. Also, a clear difference can be seen in the drying shrinkage cause of deterioration in the same mass . Glycol ether derivative which is the main ingredient of the capillary void water used to fuse cavities and reduce surface tension, is low in capillary tension and greatly reduces shrinkage. Furthermore, when comparing the deterioration of PL and D, the fact that deterioration is less in the initial stage in the case of D prompts one to conjecture that, since only 1 % of glycol ether derivative is soluble in water, even when drying

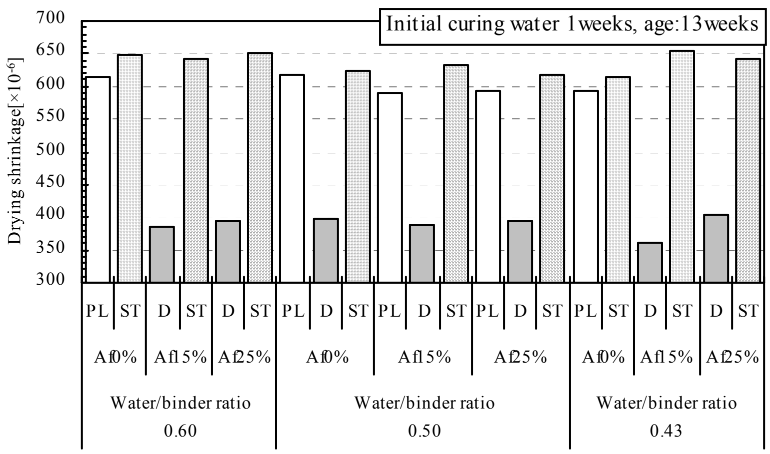

Fig.5 Change in the material length

proceeds, for the most part it acts to prevent drying of water in cavities remaining in the concrete. It is assumed that the admixture itself remains in capillary and gel voids. As Fig.15 illustrates, there are reports that in hardened concrete where durability improving admixture is used, through capillary tension, stress increases to a radius of 100 □ reducing the number of pores, Presumably this influences drying shrinkage (capillary and gel cavities), increasing the percentage of durability improving admixture, reducing drying shrinkage.

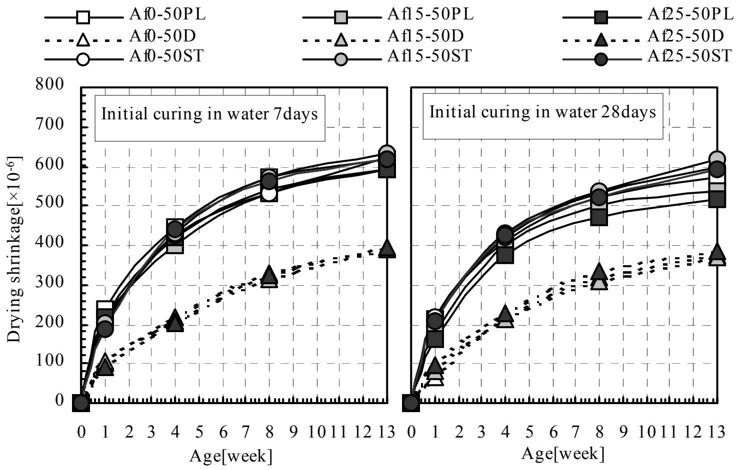

Fig.6 Change in length in 50% water-binder ratio

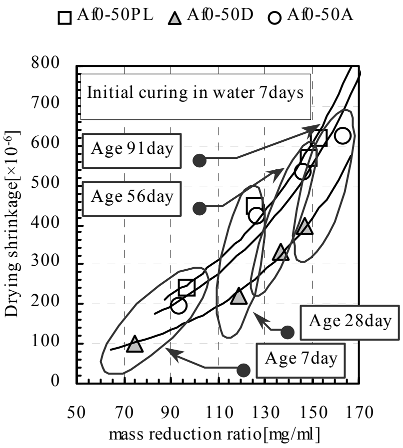

Fig.7 Mass reduction ratio and drying shrinkage

3.4 Carbonation Acceleration Experiment

In Fig.8, the relation between the square root of each water-binder ratio acceleration period and the depth of carbonation is shown. The regression line in the diagram, carbonation depth (x), is in proportion to the square root of the period of acceleration (t), or √t based on the regression line. Carbonation depth increases with mixture of FA, and the lower the water-binder ratio becomes the less the depth of carbonation. Also, generally when ordinary portland cement is used, if AE materials or AE water reducing agent is used, the carbonation velocity is increased somewhat compared to PL,but in this experiment, irrespective of the water-binder ratio, there was a notable difference in the amount of air. It is conjectured that when applied to NonAE, the composition of the concrete would become finer and carbonation controlled. Fig.9 shows the relation between carbonation velocity coefficient (A) derived from the regression formula x=A√t+B (B: carbonation depth at the start of carbonation acceleration) of Fig.8 and water cement ratio. In ST, PL and D, all lines converge, and from our experiment of pre-curing 4 weeks in water and 4 weeks in air, we assume that self carbonation due to FA pozzuolanic action can be disregarded, that there is no relation to FA replacement, and that the carbonation velocity is the same for water cement ratio. Ratios for

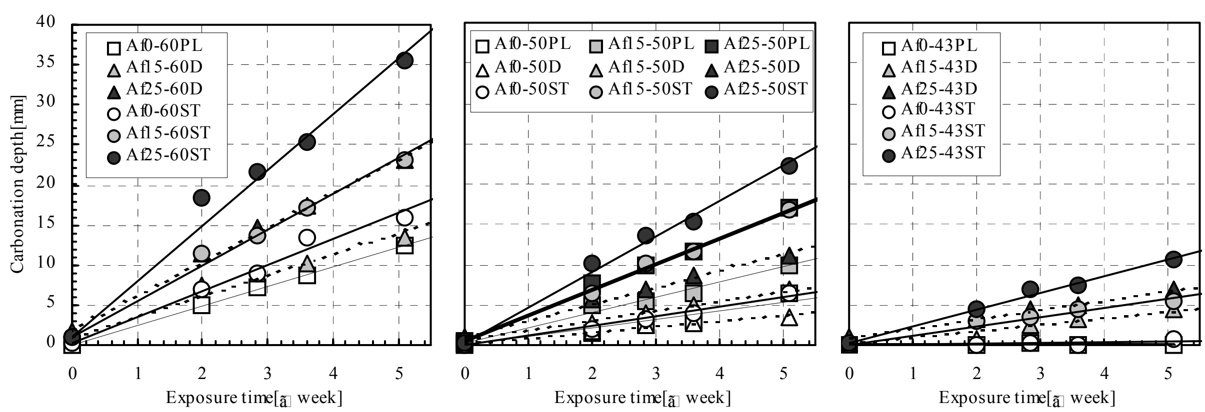

Fig.8 Exposure time and carbonation depth(Initial curing methods Ⅰ)

carbonation velocity coefficients of FA concrete are ST: PL: D = 1.0: 0.7: 0.6; for NonAE the carbonation velocity decreases markedly.The carbonation acceleration experiment was an experiment in raising the carbon dioxide concentration in the surrounding atmosphere, causing carbonation in the concrete to accelerate, and by measuring the depth of carbonation resulted in making a carbonation resistance index. Table.6 shows 4 types of carbonation acceleration pre-curing experiments in 50% water-binder ratio that were conducted and their influence was confirmed. Fig.10 shows the relation between FA replacement ratio and the carbonation velocity coefficient. From Fig.10 we can observe the difference in the carbonation velocity coefficient of both PL and ST caused by pre-curing, from the great effect of FA non-replacement pre-curing method and period, FA 25% replacement regardless of pre-curing method and period, where both PL and ST showed the same rate of carbonation velocity. In other words, not mixing FA in pre-curing results in a change in carbonation velocity coefficient, but cement mixed with 15% or more FA results in self-carbonation of FA which has a greater influence on carbonation velocity than changes in the carbonation velocity coefficient. The experiment on carbonation acceleration with 15% + FA replacement depends on the carbonation depth at the start.

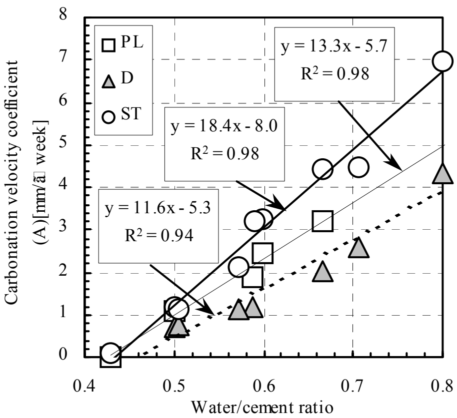

Fig.9 Water-cement ratio and carbonation velocity coefficient(Initial curing methods Ⅰ)

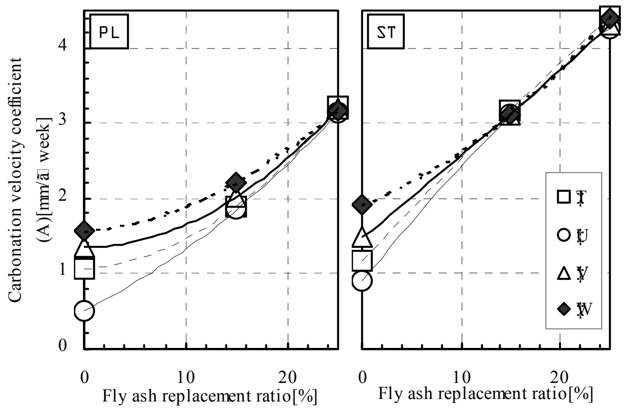

Fig.10 Fly ash replacement ratio and carbonation velocity coefficient

3.5 Freeze-Thaw Resistance

DF=(P N)/M
DF:durability index
P:relative dynamic elasticity coefficient of N cycles
N:either a cycle quantity where the relative dynamic elasticity cofficient is 60%,or 300 cycles,whichever is smaller
M:300 cycles

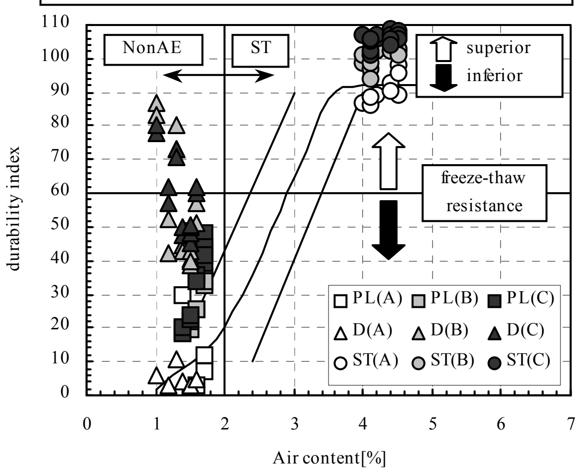

Fig.11 Air content after hardening and the durability index

Fig.11 shows the relation between air content after hardening and the durability index, and Fig.12 compares water cement ratio to the durability index. As shown in Fig.11, regardless of water-binder ratio and FA replacement ratio, ST showed favorable resistance to freeze damage. On the other hand, NonAE in this experiment also, with only 4 weeks of water immersion in the pre-curing stage was below 40%, but regardless of water-binder ratio, when durability improving admixture was used, by conducting the drying process for 2 weeks in the pre-curing stage, resistance from freeze damage improved even in NonAE. Also, from Fig.12 it can be observed that the influence of water cement on resisting freeze damage, regardless of cement type, the lower the water level in the cement the better the resistance. Cement using durability improving admixture, regardless of the length of the drying process in the pre-curing stage, performed at W/C = 60% to fulfill the the freeze-damage index of 60% or more. The curing period in actual construction work is very short, so taking into consideration stripping 24 hours after casting and exposing to air, FA concrete can be made freeze resistant by using durability improving admixture to ensure the proper amount of water in the cement, even when using NonAE material.

3.6 Characteristics of Air Bubbles

Fig.13 shows air bubble diameters and air bubbles quantity, and Fig.14 shows the relation of air-void spacing factor to the durability index. it can be seen that the number of air bubbles is greater when durability improving admixture is used and the average diameter is smaller.

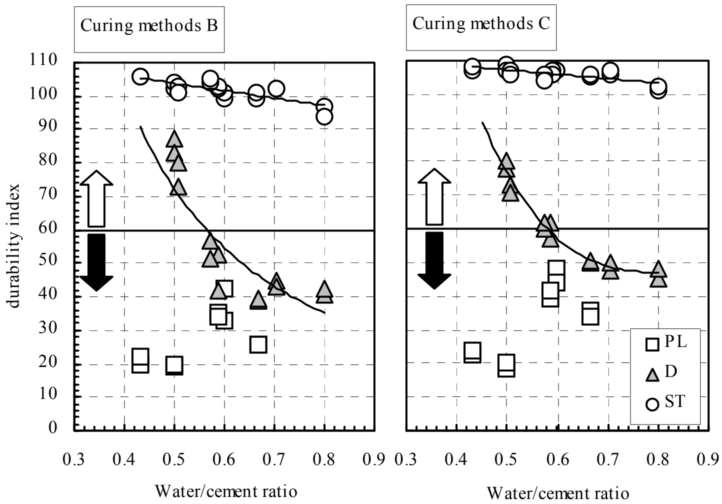

Fig.12 Water cement ratio and durability index

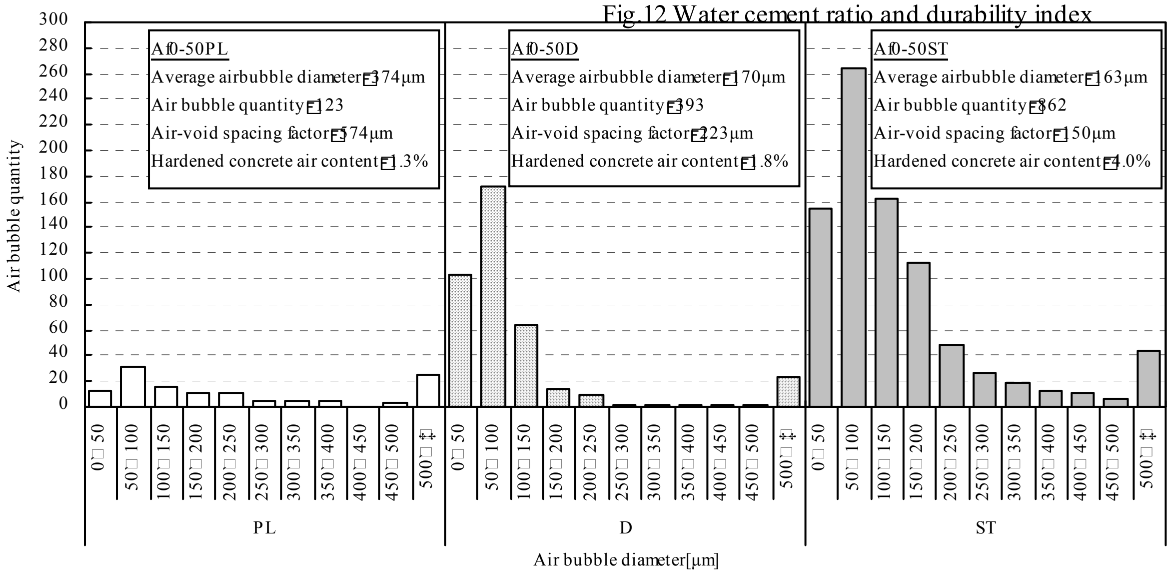

Fig.13 Air bobble diameter and air bubbles quantity

The influence of mixing in FA, without regard to the amount of air or the use of durability improving admixture, in keeping with the extent of FA replacement there is a tendency for the number of air bubbles to increase and the air bubble interval coefficient to become smaller. Also, as Fig.13 shows, there is a greater number of air bubbles when durability improving admixture is used and their average diameter is smaller. The air bubble interval coefficient is lower than the 200~250µm anti-freeze damage index shown in Fig.14, and in addition there is a strong de-foaming reaction to durability improving admixture driving out coarse air bubbles over 500µm. When the cement is fresh the quantity of air is the same in PL and D, but after hardening the quantity of air in D becomes 0.5~1.0% greater. This is thought to be because of the presence of minute grease specks, with a grain size averaging less than 150µm, left in the hardened concrete used in the experiment specimen by the glycol ether derivative of the durability improving admixture that ran through the material when polishing the air bubble system during the experiment.

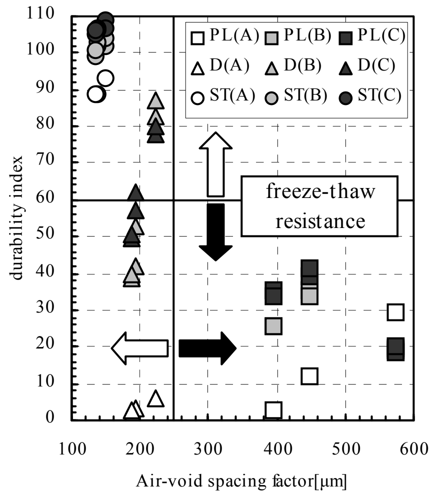

Fig.14 Air-void spacing factor and durability index

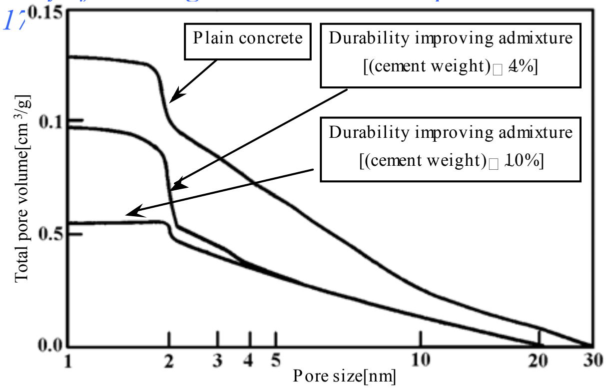

Fig.15 Concrete cavity distribution when durability improving admixture is used

Concerning the effect of durability improving admixture in preventing freeze damage, a significant improvement was noted when it was introduced in the 2 week drying process during pre-curing. In the concrete, the glycol ether derivative acts to reduce drying shrinkage just as was conjectured; i.e., when void water is dried by injecting the glycol ether derivative into minute capillary and gel air cavities, it performs the same function as the air bubbles it replaces, reducing freeze expansion and damage. Cavity distribution in concrete, when durability improvement admixture is used, as shown in Fig.13, by its strong bubble elimination action, the admixture reduces the entrapped air and replaces it with the same amount of entrained air bearing 250 µm or less of oil specks into the hardened material. Furthermore, accompanying the drying of void water, by injecting a portion into minute capillary cavities, shrinkage is lowered. It is also believed that the same reaction occurs in gel cavities, which are the lamellar crystal left in lamellar cavities formed while C-S-H hydrate is produced as the material matures. Through these processes hardened concrete is formed and durability is improved. Fig.15 shows concrete cavity distribution when durability improving admixture is used.

4. CONCLUSION

Compressive Strength: Making FA concrete into NonAE increases the material's strength 10~20% compared to materials of the same water-binder ratio, resulting in reducing initial compressive strength defects..

Tensile Strength and Static Elasticity Coefficient: Both ST and NonAE tensile strength and static elasticity coefficient depend on compressive strength..

Drying Shrinkage: By using durability improving admixture, regardless of whether or not the material is W/B or FA replacement ratio, drying shrinkage is controlled at 60% compared to when the mixture is not added.

Carbonation Acceleration Experiments: In our experiment of pre-curing 4 weeks in water and 4 weeks in air, the carbonation of FA concrete compared to the water cement index, regardless of W/B or FA replacementratio, for carbonation velocity coefficient ratio of FA concrete are ST: PL: D = 1.0: 0.7: 0.6. Also, in the experiment of carbonation acceleration where FA replacement ratio was 15% or more, regardless of pre-curing method or time period, the results depended on the carbonation depth at the beginning of acceleration.

Freeze-Thaw Resistance: By using durability improving admixture for 2 weeks curing in air (drying process), keeping NonAE a water cement level at 60% or less, the freeze damage resistance improves.

Not only is durability improved by making FA concrete into NonAE, the regulation of air content is facilitated making quality management easier. In accordance with use and application, in the manufacture of FA concrete, NonAE and durability improving admixture is valuable option.

6. References

Izumi, Kasami, *et al*, (1985), 'Cement Types, Blends and Curing Conditions Influencing Concrete Carbonation,' Concrete Engineering Annual Lecture Meeting Collected Papers, [30], 177-120.

Sugiyama,*et al*, (1986), 'Basic Properties of Concrete Using Admixtures to Decrease Drying Shrinkage,' Concrete Engineering Annual Lecture Meeting Collected Papers, [71], 117-284.

Sugiyama, *et al*, (1986, Aug.), 'Development Research in High Durability Concrete (1-3),' Architectural Institute of Japan Academic Lecture Synopsis Papers.

Kunio Yanagibashi, (2000, Nov.), 'Development of Super-high Durability Concrete,' Cement Concrete, No. 645.

Coconut Coir Cement Board

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ABSTRACT

Natural organic fibers exist in reasonably large quantities in many countries of the world. Coconut coir fibers are widely available in Thailand. They are the seed-hair fiber obtained from the outer shell (endocarp) or husk of the coconut. In addition, coconut coir has low thermal conductivity. Using coconut coir cement board as a construction component with low thermal conductivity will reduce heat transferred into building which decrease energy consumption of building facilities (air condition). This will reduce the utility cost. Coconut coir cement boards are composites to alternative option to save energy consumption of building. The main parameter was investigated namely mixture ratio of coconut coir fiber, cement and water. Three mixing ratio by weight were varied 1:2:1, 1:1:1, and 2:1:2, respectively (cement : coconut coir : water). In this research, coconut coir fiber length of 0.5-1 cm. was used in coconut coir cement board production. In addition, boiled and washed pretreatment of coconut coir was obtained in fiber preparation. The specimens were cast in 350 mm. x 350 mm. x 10 mm. steel molds. After the cool pressing, the board was stacked and stored for 28 days in order to be completely cured, and then trimmed and cut into various test specimens. The investigation revealed that the optimum of mixture ratio is 2:1:2. The corresponding composite properties are as follows: thermal conductivity of 0.40 w/m K, MOR(Modulus of Rupture) of 19.94 MPa MOE(Modulus of Elasticity) of 5315 MPa internal bond of 0.73 MPa, thickness swelling of 3.64 MPa and 9.13 % moisture content. Examination of chemical composition analysis indicated that boiled and washed coconut coir fiber have high lignin and cellulose. High amount of lignin and cellulose would increase of the strength of composites.

Comparison of commercial board composite confirmed that the coconut coir-based lightweight composite have a low thermal conductivity. That is extremely interesting for energy saving with use as ceiling and wall material.

KEYWORDS

Coconut coir, Cement board, Fiber, Energy, Building

1 INTRODUCTION

Thailand located in the tropical zone. There are a huge amount of young coconut (*Cocos nucifera*). It is the most interesting product as it has the low thermal conductivity [Khedari 2000]. In additions, using coconut coir cement boards as a construction component with low thermal conductivity will reduce heat transferred into building which decrease energy consumption of building facilitied (air condition). This will reduce the utility cost [Khedari 2002] . Energy has a significance influence in developing government, industrial and business sector of worldwide. Nowadays, energy price trends to be high. Thailand is the one of the country facing to this problem [Oranratnachai 1976]. Coconut coir cement boards are composites to alternate option to save energy consumption of building. In this work, investigation of difference mixture ratio between coconut coir and cement of low thermal conductivity cement composite board was the main objective.

2 EXPERIMENTAL SETUP

In this research, three board densities were considered for investigation : 0.8, 0.9, 1 g/cm^3 for coconut coir board. Three ratios used by weight were 1:2:1, 1:1:1, 2:1:2 (cement : coconut coir : water). The boards were made into three replicates for each combination. So there were 27 specimen boards (three board density x three ratios by weight x three replicates).

2.1 Raw materials

Chemical composition	*Coconut Coir fibre* [%]	*Boiled Coconut Coir fibre* [%]	*Standard*
Ash content	2.8	0.8	TAPPI-T211-om-93
Alcohol-benzene solubility	3.0	5.0	TAPPI-T204-cm-97
Lignin	32.1	32.7	TAPPI-T222-om-98
Holocellulose	68.9	73.9	Acid chlorite's Browing
Alpha-cellulose	34.9	41.8	TAPPI-T203-cm-88
Hemi-cellulose	16.8	19.4	-

Table 1 Chemical composition of not boiled coconut coir and boiled coconut coir fiber [4]

Coconut coir is the seed-hair fiber obtained from the outer shell (endocarp) or husk of the coconut. Chemical composition of coir is given in table 1 It can be seen that the coconut coir fiber contain a high lignin, holocellulose, alpha-cellulose, hemi-cellulose ratios. That makes fibers stiffer and tougher. The stiff and tough fibers are difficult to beat, do not conform and collapse against each other so well. It is necessary to know the basic and chemical properties of coconut coir before preparing sample. Information on the chemical compositions of coconut coir is important in order to understand cement board' physical and mechanical properties. Actually, cement boards that have a high lignin content will be rather stronger with a high water-resistance.

2.2 Fiber preparation

Coconut coir fibers were cut into the length of 1-13 cm. They were boiled for 2 hours to extract the following water soluble chemicals : sugar, starch, fat, tannins, resin, quinines phenols and hemi cellulose. They are the coconut coir endo-substances. And then the fibers were washed with tap water. After they had been dried in solar radiation for 2 days, the fibers were cut into 0.5-1 cm. size.

2.3 Board preparation

Fiber sample were mixed with cement and water in the ratios of cement : fiber sample : water, 1:2:1, 1:1:1, and 2:1:2 until homogenous. To obtained the coconut coir fiber cement boards, the mixture were compressed by cooling method with the pressure of 560 kg/cm^2 for 24 hrs. Cement, fiber and water ratio was varied to obtain boards of different density.

2.4 Specimen preparation and testing

After the cool pressing , the board were stacked in order to be completely cured for 28 days and then trimmed and cut into various test specimens. The specimens were conditioned in a conditioning room until they reached equilibrium for at least 2 weeks at room temperature. Afterwards, testing specimens were carried out according to JIS A 5908-1994 (Japanese Standard Association, 1994) for physical and mechanical properties (density, moisture content, water absorption, thickness swelling, modulus of rupture (MOR), modulus of elasticity (MOE) and internal bond). The thermal conductivity of the cement boards was measured by using JIS R 2618

2.5 Experimental apparatus according to JIS A 5908-1994

In this section, a brief description of testing equipments and method is presented. Universal testing machine (Instron type): was used for bending strength test (MOR): the testing method implies to apply a load of approcximately 10 mm/min at a mean deformation speed from the surface of the test piece and to measure the maximum load (P). Next, calculate the bending strength of individual test piece from

$$\text{Bending Strength } \left(N/mm^2\right)\left\{kgf/cm^2\right\} = \frac{3P_m L}{2bt^2} \quad (1)$$

Where P_m is the maximum load (N), (kgf), L the span (mm),(cm), b the width of test piece (mm),(cm), t the thickness of test piece (mm),(cm)

Breading load test (MOE): From the bending strength test, a graph which relates the maximun load (P) to the bending distance was plotted. The value of bending strength tests were calculated using the formula (2) when increasing load and bending distance using the liner line of the graph:

$$MOE = \frac{L^3 \Delta W}{4bd^3 \Delta S} \quad (2)$$

Where L is the span (mm), ΔW the increasing load in the range of linear line of graph (N), ΔS the increasing bending distance in the range of linear line of graph (mm), b the width of test specimen (mm), d the average thickness of specimen (mm).

Test of expansion ratio in thickness due to water absorption: First, measure the thickness in the center of a test piece (t_1) to the nearest 0.05 mm with a dial gauge or a micrometer. Next immerse it in water at 20 ± 1° horizontally about 3 cm below the water surface for 24 h, take it out, wipe off the water and measure the thickness (t_2) again in the same manner as above. Then ,calculate the expansion ratio in thickness due to water absorption from the formula (3) below:

$$\text{Expansion ratio in thickness} = \frac{t_2 - t_1}{t_1} \times 100 \quad (3)$$

Internal bond Measurement (Wolpert) for internal bond test: Adhere a test piece to steel or aluminum blocks. Apply a tension load vertically to the board face, measure the maximum load (P′) at the time of failing force (breading load of perpendicular tensile strength to the board), and calculate the internal bond using the following equation (4). In this test, the tension loading speed shall be approximately 2 mm/min:

$$\text{Internal bond} = \frac{P'}{(b \times L)} \tag{4}$$

Where, P' maximum load (N) {kgf} at the time failing force, b : width (mm) {cm} of test piece, L : length (mm) {cm} of test piece

3 Results and discussion

Mixture ratio by weight Cement : Coconut coir : Water	*Mixture ratio by Weight* [g] Cement : Coconut coir : Water	*Density* [g/cm^3]	*MC* [%wb]	*MOE* [Mpa]	*MOR* [Mpa]	*TS* [%]	*WA* [%]	*IB* [Mpa]	*Thermal conductivity* [W/mK]
1 : 2 : 1	360 : 720 : 360	0.79	11.26	698.20	6.37	40.04	86.99	0.29	0.23
1 : 1 : 1	540 : 540 : 540	0.98	11.01	2266.03	13.35	13.70	40.38	0.51	0.28
2 : 1 : 2	720 : 360 : 720	1.16	10.12	4792.56	15.91	5.25	24.22	0.64	0.33
1 : 2 : 1	405 : 810 : 405	0.84	11.19	915.13	9.23	28.23	66.31	0.35	0.24
1 : 1 : 1	607 : 607 : 607	1.03	10.98	2884.39	16.89	10.66	35.99	0.59	0.30
2 : 1 : 2	810 : 405 : 810	1.19	9.63	5031.14	17.60	4.04	21.25	0.69	0.40
1 : 2 : 1	450 : 900 : 450	0.87	11.02	1238.44	10.74	27.06	64.72	0.38	0.24
1 : 1 : 1	680 : 680 : 680	1.03	10.65	2905.15	17.35	10.20	35.20	0.68	0.32
2 : 1 : 2	900 : 450 : 900	1.22	9.13	5419.33	20.34	3.64	19.65	0.73	0.40

Table 2 Physical and mechanical properties of cement boards from coconut coir fibers .

Note: MC = Moisture Content, MOE = Modulus of Elasticity, MOR = Modulus of Rupture, TS = Thickness Swelling, WA = Water Absorption, IB = Internal Bond

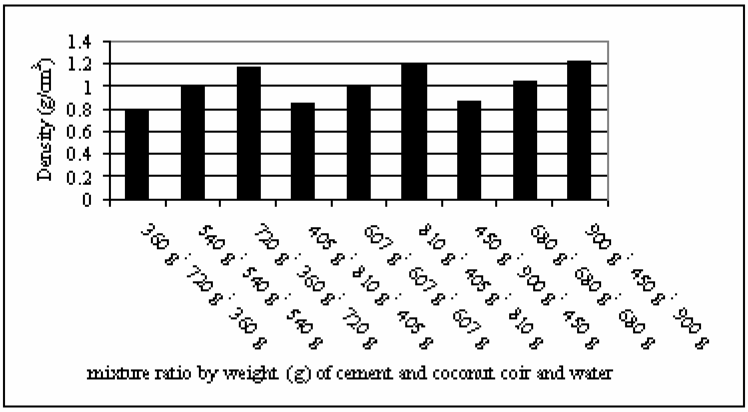

Figure 1. Density of cement boards VS mixture ratio of cement and coconut coir and water

Figure 1. presents results of the mixture ratio of cement, coconut coir and water and total mass on board density. For the same total board mass, the density were changed with the increase of cement content in direct proportion. It was also increased with in increase total board mass. For the low total mass or light cement board (low density) have more space and void than high total mass or heavy cement boards (high density).

Figure 2. presents the effects of the total mass and cement mixture ratio on moisture content . The moisture content is inversely proportional to the total mass and cement content. It shows that high content of coconut coir provides high moisture content.With high content of cement, the internal bond between cement and cememt can increase. The space and void of cement board decrease. A lower density yielded higher porosity, space and voids. Consequently, the moisture content increase when the pororosity was increased.

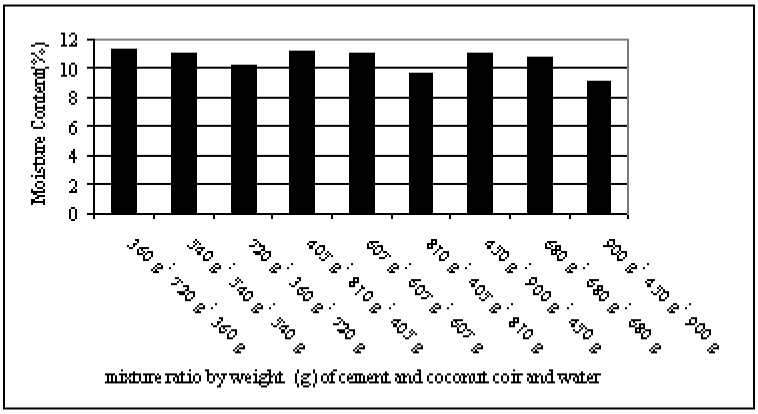

Figure 2. Moisture content of cement boards VS mixture ratio of cement, coconut coir and water

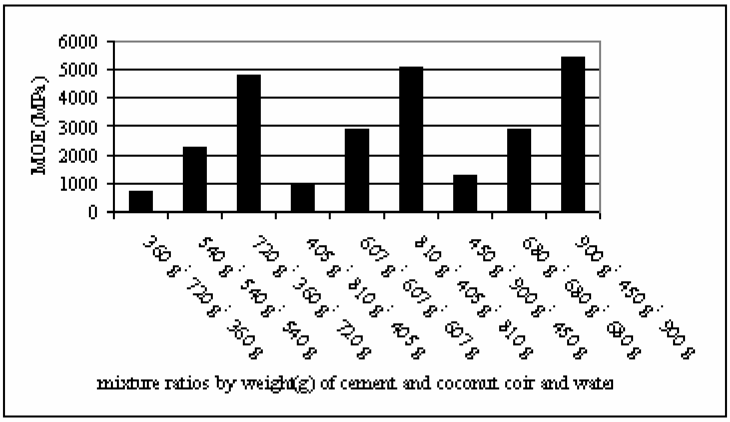

Figure 3. MOE (Modulus of Elasticity) of cement boards VS mixture ratio of Cement, coconut coir and water

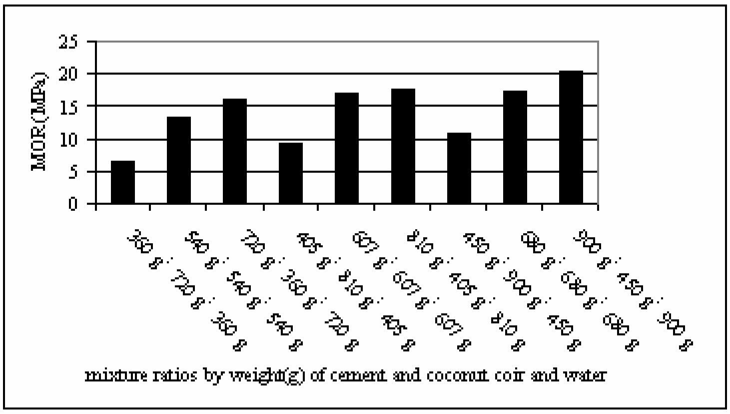

Figure 4. MOR (Modulus of Rapture) of cement boards VS mixture ratio of Cement, coconut coir and water

MOE and MOR were varied when the content of cement in the mixture ratio and total mass was increased as shown in Fig3. and Fig 4. This probably that the internal bond of cement in cement boards act as an adhesive and increase the strength of the cement bond. In addition, the high total mass or the high density board are stronger and stiffer than low total mass or low density cement boards.

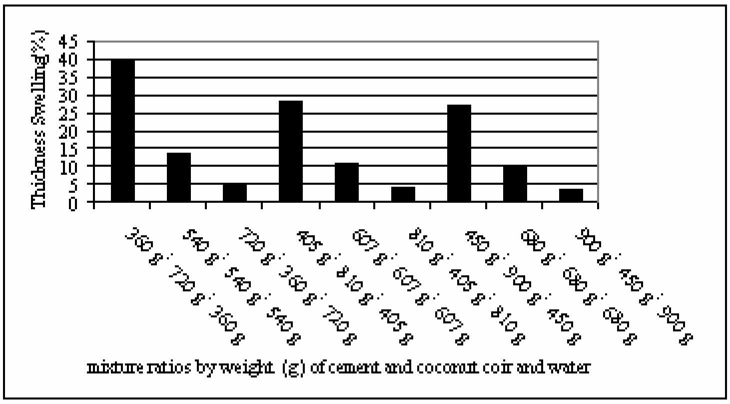

Figure 5. Thickness swelling of cement boards VS mixture ratio of cement, coconut coir and water

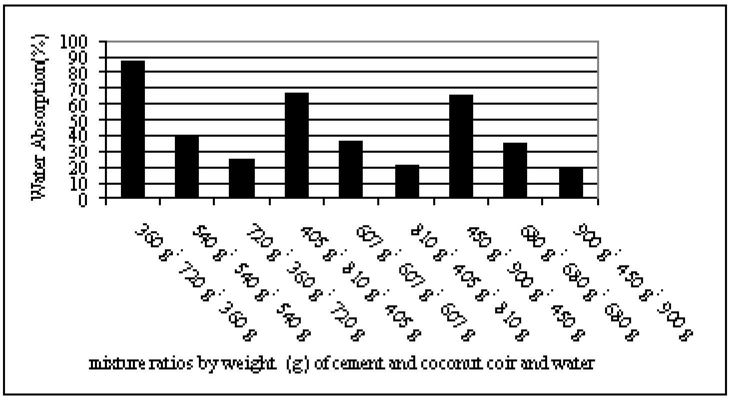

Figure 6. Water absorption of cement boards VS mixture ratio of cement, coconut coir and water

The thickness swelling is inversly porportional to the increase of the cement content and total mass or density as shown in Fig 5. For the low total mass or low density have more space and void than high total mass or high density. Consequently, low total mass or low density of cement boards had a high thickness swelling while high total mass or high density of cement boards had a low thickness swelling value.

The water absorption is inversly proportional to the increase of the cement content, total mass of cement board or density of cement board as shown in Fig 6. Consequently, the higher proportion of cement, the lower of water absorption . This phenomena is caused by the low void space and higher internal bond between cement and cement than internal bond between cement and coconut coir fibers.

Figure 7. presents the effects of the mixture ratio and total mass on internal bond . It can be observed that the internal bond was changed when the cement mixture ratio and total mass of cement board or density were increased. Hence, an increase of total mass of cement board or density of cement boards accommodate a higher internal bond value. In addition, by considering the mixture ratio, if cement boards have a high content of cement, they will also have a high internal bond value. Consequently, the low total mass or low density of cement boards have more space and void than high total mass or density of cement board.

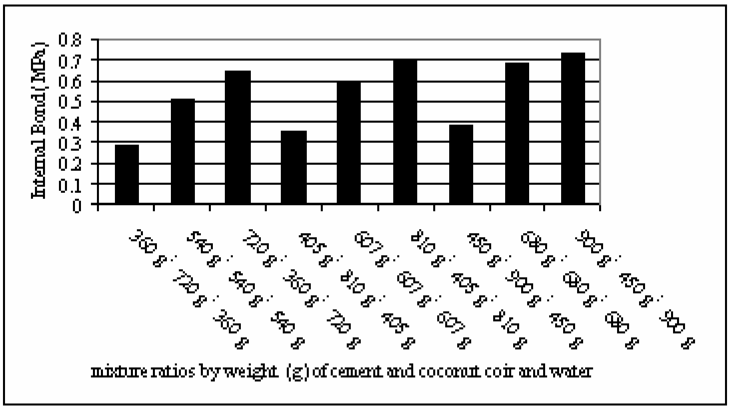

Figure 7. Internal bond of cement boards VS mixture ratio of cement, coconut coir and water

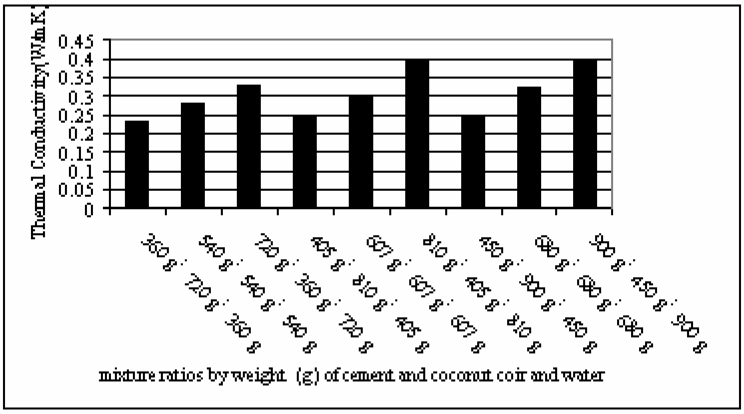

Figure 8. Thermal conductivity of cement boards VS mixture ratio of cement, and coconut coir and water

Figure 8. presents results of mixture ratio or the total mass on thermal conductivity . High total mass or high density cement boards have high thermal conductivity because space and void in cement board are decrease. For the same total mass, when the cement content is high, the thermal conductivity of the cement board is higher than that of the high coconut coir content. This is caused by the structure of board, whose porosity is closely dependent on the cement content.When measured thermal conductivity of the tested boards (0.4 W/mK) were compared to commercial board (Eterpan board 0.68 W/mK), it can be seen that the thermal conductivity of the boards tested was lower than that of commercial board, but it exceeded standard for Thailand (0.155 W/mK). Therefore, cement boards made from coconut coir have potential for development for using as an insulating component of building materials.

4 CONCLUSION

The development of randomly distributed short coir fiber reinforce cement composite with low thermal conductivity is the main purpose in this study. Main parameter was investigated namely mixture ratio of coconut coir fiber, cement and water. The optimum of mixture ratio is 2:1:2. The corresponding composite properties are as follows: thermal conductivity of 0.40, modulus of rupture (MOR) of 19.94 MPa, modulus of elasticity of 5,315 MPa, internal bond of 0.73 MPa, thickness swelling of 3.64 MPa and 9.13 % moisture content. These value showed the good properties of this cement board. Examination of chemical composition analysis indicated that boiled and washed coconut coir fiber have high lignin and cellulose. High amount of lignin and cellulose would increase of the strength of composites.

Comparison of commercial board composite confirmed that coconut coir-based lightweight composite have a low thermal conductivity. That is extremely interesting for energy saving with use as ceiling and wall material.

5 ACKNOWLEDGEMENTS

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6 REFERENCES

Khedari, J., Suttisonk, B., & Pratinthong, N. 2000, 'New lightweight composite construction material with low thermal conductivity', *CECO Journal*, **23**[1], 1-6.

Khedari, J., Charoenvai, S., & Hirunlabh, J. 2002, 'New insulating particleboards from durian peel and coconut coir', *Building and Environment Journal*, 38[3], 435-441.

Oranratnachai, A. 1976, *Mechanical and Physical Properties of Coir-Fibre Boards*, Master of Engineering Thesis, Structure Engineering Program, Asian Institute of Technology, 60 p.

Polymer Coatings for Concrete Structures Exposed to Corrosive Environment

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ABSTRACT

The question of corrosion protection of reinforced concrete structures is dependent upon the corrosive environment to which these structures are exposed. For concrete protection different methods and procedures are applied. The measures for the protection and minimization of premature damage of a structure should be proposed in a way that is more advantageous for the proprietor, from the economical point of view, than the structure without protection. Application of surface coating treatment is one of the possibilities. This paper describes the development and testing of special protective coatings for concrete structures exposed to very corrosive environments. However, surface protection of concrete structures through the use of coatings is an expensive secondary measure. A significant part of production costs is the price of the materials. One of the possibilities to reduce costs is the choice of less expensive base materials (e.g. binder and fillers), but one that, nonetheless, provides sufficient protection to the structure. Another possibility is to replace part of the binder with filler given that the cost of filler is typically much less than that of the binder. The utilization of industrial waste materials as filler in coatings is suggested as a means to achieve reductions in production costs and also provide indisputable ecological benefits. The primary objective of the research was to compare the expected performance of different coatings in relation to the both types of polymer-based binder and type of filler as well as the degree filling. The exploitation of waste materials is advantageous for environmental reasons and also potentially provides economic savings in production costs. Three types of industrial wastes were selected: "washing wastes" (waste from washing of crushed aggregate); ground blast furnace slag and; power plant fly ash.As well, three types of polymers were tested as binders for protective coatings: Vinilester (VE);) Polyester (UP), and; Polyurethane (PUR). One of the main goals of the research was to find the optimum quantity of individual waste materials that would not change the functional properties of a particular coating. In this regard, the key coating properties evaluated in selected tests included:: water tightness, , permeability to water vapor and CO_2, degree of adhesion to concrete substrates and an entire group of tests to assess the performance of the protective coatings when subjected to exposure to different corrosive media (e.g. acids, oils). This paper reports on partial results of the performance of the coatings when subjected to corrosive media..

KEYWORDS

Concrete, corrosive media, protection, polymer coatings.

1 INTRODUCTION

The lifetime of reinforced concrete construction work is substantially limited by the amount of deterioration and chiefly by the corrosion of reinforcement. The causes of reinforced concrete deterioration can be various [Matousek & Drochytka 1998]. In general, aggressive media can be divided according to the types that affect concrete:

- Aggressive gases and vapors with acidic character (CO_2, SO_2, N_xO_y, H_2S etc.),
- Aggressive waters and solutions,
- Hygroscopic solid substances,
- Microbiological effects,
- Mineral fats and oils,
- Stray currents that affect reinforcement

2 STRATEGY OF PROTECTION

The question of protection of reinforced concrete structures depends on the corrosive surroundings to which these structures are exposed. The protection of concrete structures can be realized in different ways [Drochytka & Petranek 2002]:

- Change of operational and exposure conditions
- Improvement of physical properties of repair materials for original concrete
- Influencing the electrochemical behavior
- Application of different types of surface treatment

Several effective methods for sufrace treatment and proctection of concrete are currently used including, for example: coatings, membranes, and impregnation paints. However, protecting the surface of concrete structures is an expensive secondary measure, with a significant part of the production costs is the price of the raw material (e.g. binder and fillers). One of the possibilities to reduce costs is the selection of less expensive raw materials, that will nonetheless provide sufficient protection to the structure. Another possibility is to replace part of the binder with filler. The utilization of industrial waste materials as filler in coatings is suggested as a means to achieve reductions in production costs and also provide indisputable ecological benefits. In this paper, the development of surface treatment, specifically protective coatings, is discussed.

3 THE METHODOLOGY OF TESTS

At the Institute of Technology of Building Materials and Components, Faculty of Civil Engineering, Brno University of Technology, Czech Republic, in the framework of research, the problems of the investigated protective coating types were tested. Those that utilize waste materials instead of usual fillers. Tested coatings can be divided into two main groups; silicate coatings and polymer coatings. Polymer coatings are based on polyester, vinyl-ester and polyurethane. They are designed for special applications where the concrete is exposed to a strong corrosive medium. The selection of materials was directed by local accessibility, by urgency of processing or liquidation, and by current known limits of utilization possibility. Admittedly, the waste materials are from local sources and their actual technical properties differ from all other materials, but on the basis of this research it is possible to deduce generally applicable dependences for the same waste materials from other localities.

3.1 Description of individual waste materials used in paints

Three types of waste materials were used as fillers for the protective coating materials, of which brief descriptions of their physical and chemical properties are given below.

Fly ash - Fly ash from electrostatic precipitators (Power plant Chvaletice, east Bohemia) was used in this work. The fly ash consists of two main components: mullite and β-quartz. The chemical composition is relatively stable, consisting of the following compounds:

SiO_2 57%, Al_2O_3 29%, Fe_2O_3 6,2%, TiO_2 2% , CaO 1,7%, MgO , K_2O 1,8%.

Values for the bulk density and specific surface area are 2060 kg/m^3 and 270 m^2/kg respectively.
Slag - Ground blast furnace slag is a granulated material formed by quick cooling of blast furnace slag during the production of iron. The blast furnace slag used in this study was obtained from steel plant Trinec (north-east of the Czech Republic) The chemical composition is variable, consisting of:

CaO (30-50%), SiO_2 (30-43%), Al_2O_3 (5-18%), MgO (1-15%)

Values for bulk density and specific surface area respectively are: 2810 kg/m^3 380 m^2/kg.

Wastes from washing crushed aggregates - In brief, this waste material, referred to as "washing waste", is formed in the quarry from washing crushed aggregate. It is mainly the production of washed standard sand. The slurries formed in this manner accumulate and then are stockpiled. The washing waste has the same mineralogical and chemical composition as washed sand. The mineralogical and chemical composition necessarily depends on the locality of the source. The grain size composition depends on the kind of mineral, the crushing technology, and the method of treatment and separation. The chemical compositions of two samples of washing wastes are given in Table 1.

Compound	*Sample 1 Amount %*	*Sample 2 Amount %*
Insoluble material	59.5	91.47
CaO	0.82	1.19
Al_2O_3	0.4	1.39
Fe_2O_3	1.17	1.92
pH	8.5	8.2
Minerals determined by X-ray analysis	β-quartz, feldspar, muscovite, kaolinite	β-quartz, feldspar, muscovite, kaolinite, gehlenite, montmorillonite

Table 1 Chemical and mineral analysis of waste washings

Properties of two kinds of waste washings having different properties:
Bulk density: Sample 1 = 2170 kg/m^3
Sample 2 = 2690 kg/m^3 ,
Bulk density (jarred): Sample 1 = 1470 kg/m^3
Sample 2 = 1420 kg/m^3.

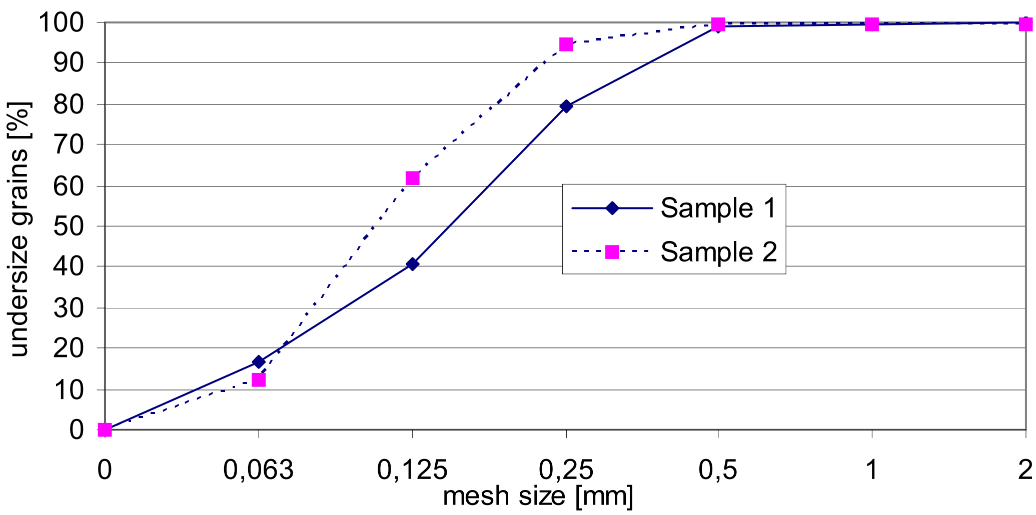

Figure 1 Sieve analysis of two types of washing wastes

3.2 Binders

The experimental work was carried out using three types of polymer-based binders:

- Vinyl-ester (VE)
- Polyester (UP)
- Polyurethane (PUR)

3.3 Application of coatings and proposed proportions

All coatings were applied with a paint brush on vibro-pressed concrete paving bricks (200 x 200 mm). The composition of the vibro-pressed concrete paving bricks corresponds to requirements for concrete as a substrate for testing surface treatment of building structures (min. Rb 50 Mpa).
The methodology used for assessing the polymer coating performance was chosen to describe the changes in coating behavior as a function of filler loading. It is divided into two phases:

Phase 1. Selection of most preferable mix proportions in which proportions were adjusted by the progressively increasing the quantity of filler incorporated in the mix in increments of 5%, [wt.]. This permitted determining the effect of filler quantity on specific coating properties for constant quantities of binder. Hence it also includes testing basic properties and the selection of most preferable mix proportions by means of an optimization process. This led to further modification of the coatings' mix proportions.

Phase 2. Determination of selected coating properties. Tests were performed to determine functional properties on coatings following optimization of mix proportions.

4 TEST RESULTS

The consistency of freshly applied coating (as measured in terms of time of outflow) is an important property. It has been found that consistency is a crucial property that relates directly to the ease of application of coatings on concrete structures [Petranek & Hudec 2003].

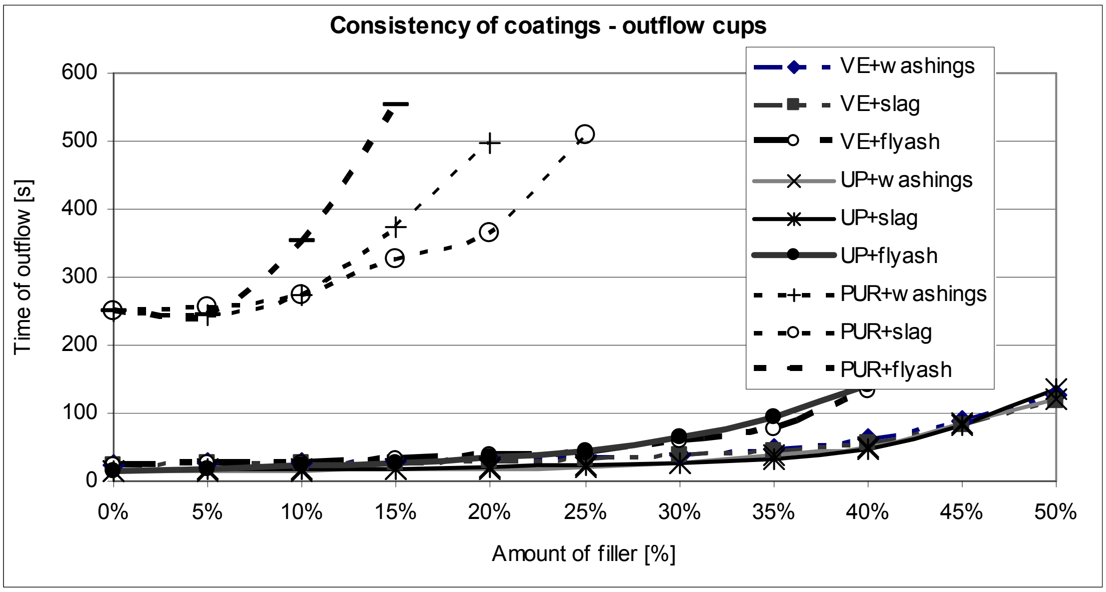

Figure 2Consistency of polymer coatings in the fresh state. Outflow cups was the method used to determine the "consistency" of the coating. The graph shows the change of consistency in relation to the amount of filler [wt%] incorporated in the coating. PUR coatings had a higher consistency that other coatings even when no filler was present in the blend.

Consistency is influenced by the properties of the binder as well as the amount of filler loading. Figure 2 provides test results for consistency of different polymer coatings having varying amounts of filler waste materials.

An essential property of coatings is bond to the substrate. All coatings tested had, due to high binder quality, very good results as shown in see Figure 3.

As might be expected, the characteristics of polyurethane coatings in this respect significantly differs from that of either polyester or vinyl-ester resins. The high viscosity of the binder considerably limited the degree of filling. When applied, this caused an excessively thick coating. From another point of view such thick coatings offer additional protection since they tend to cover small unevenness of the substrate surface. With increasing quantities of filler incorporated in the blend, coatings quickly lose their self-leveling properties after application. For this reason the maximum filler loading for coatings filled with slag was only 30% [wt.], for washing wastes, a 20% [wt.] and for fly ash 20% [wt.]. Optimal filler loadings had lower values as provided in Figure 2.

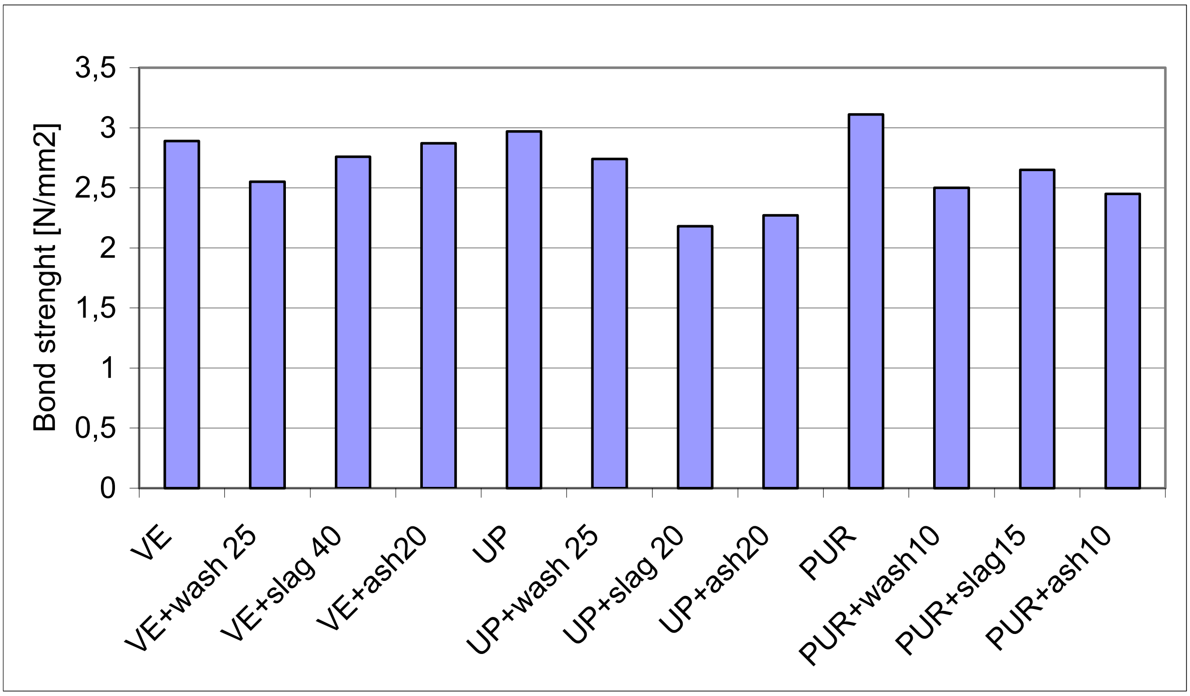

Figure 3 Bond strength of polymer coatings to concrete surface. VE+wash25 – VE binder filled with 25 % [wt.] of waste washings. Other abbreviations are analogous.

The polyester and vinyl-ester based-coatings enabled a maximum filler loading of 50% [wt.] owing to their low viscosity. In the case of transparent vinyl-ester this meant that it became non-transparent at high filler loadings. , The micro-hardness of both polyester and vinyl-ester resins significantly exceeds the micro-hardness of polyurethane. Results from tests on a both resins and the PUR often exceeded the values of the reference samples. The vinyl-ester resin had self-leveling properties even with high percentage of filling with all types of binders. Additionally, no brush strokes were formed after application

It was found that if polyester and vinyl-ester were used without filler the gelatinization takes place as a rapid exothermic reaction. In this instance, temperature increases of up to 60ºC occur with very rapid curing taking place. Curing in these conditions brings about considerable shrinkage of the resin. However, when fillers are added, the curing reaction is not so rapid and the shrinkage is reduced. In thin-layer materials, the importance of shrinkage effects is not as significant as in bulk samples, and the shrinkage is further limited by the addition of fillers.

5 CONCLUSION

At the present time filler materials commonly used in coatings are fine ground limestone or ground pure quartz sand. The use of industrial waste materials such as a substitute for these fillers was proven and verified for the development of protective coatings. Besides the environmental benefits, this would also have a positive economic effect.
The result of the work verified that the filling with selected raw waste materials has no significant effect on the properties of the coating itself up to a relatively high degree of filling. All test result values exceeded the values required for coatings by relevant standards or technical conditions. The limiting factor for filling the binder is the workability of the fresh coating. The research of long term durability of these coating applications on real structures is in progress.
For the most advantageous utilization of wastes the following proportions were selected:

- Vinilester + 40% slag, with similar properties as VE + 25% washing wastes
- Polyester + 25% washing wastes
- Polyurethane + 15% slag

In contrast with silicate coatings, the noted proportions of polymeric coatings have more general validity because the polymer binders are not as sensitive concerning the type and the properties of fillers, except for significant changes of pH value. The mentioned proportions for polymer coatings can be used even for other fillers with similar grain size distribution.

6 ACKNOWLEDGMENTS

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7 REFERENCES

Drochytka, R., Petranek, V. 2002 'Atmospheric Concrete Deterioration'. Proc. 1st FIB Conference In Concrete Structures in the 21st Century. Japan Concrete Institute, 1st ed. Osaka, Japan 20 24 November 2002, vol. 2, pp.10 19

Petranek, V., Hudec, P. 2003 'Silicate Coatings with Waste Materials'. Proc. 25th WTA International Conference, 1st ed., Brno, Czech Republic, 23 November 2003, pp 92 98. ISBN 80-02-01538-X

Matousek, M., Drochytka, R.: Atmosférická koroze betonu (Atmospheric deterioration of concrete), IKAS, Praha, 1998. IBSN 80 – 9902558-0-9. (in Czech)

Moisture Related Properties of Oriented Strand Board (OSB)

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ABSTRACT

The objective of this Ph.D. research project is to determine how various Oriented Strand Board (OSB) production parameters and physical panel properties affect the moisture-related performance of OSB. The study also looks at the various component layers of OSB individually since the material is non-homogenious and each layer behaves quite differently. The permeability of individual layers as well as the composite section were evaluated under various moisture conditions including following cyclic wetting and drying. The trial panels for this project were specifically manufactured in a commercial OSB mill, under controlled conditions. The panels were then tested in the laboratory for water vapor permeance and sorption through five relative humidity gradient steps. The outcome of this work will serve to facilitate the design of durable, healthy, energy efficient homes with OSB, and to allow the OSB manufacturer to produce a product with optimum moisture performance properties.

KEYWORDS

OSB, moisture, durability, permeance, relative humidity.

1 INTRODUCTION

Oriented Strand Board (OSB) is a structural wood composite panel product, made from wood strands, bonded together with a synthetic resin under heat and pressure. OSB comes in the form of sheets, most commonly 4-foot (1220 mm) by 8-foot (2440 mm) and a range of thicknesses from ¼" (6.5 mm) up to 1.5"(38 mm), and is used primarily for residential or lightweight construction.

Although North American wood frame houses perform very well in a wide range of extreme climates, they do occasionally experience moisture-related problems. The problems, which may cause health problems, can range from mould and mildew infestation, to buildings having extensive structural damage (Straube, 1998). Much of the residential wood frame construction in North America uses OSB for sheathing of walls (56%), floors (65%) and roofs (72%) in ever increasing amounts (24.7 billion board feet in 1993, increased by approx. 100% since 1994, (APA 2003)). Given the widespread use, OSB is often unjustly blamed for many of the problems. Since Canadian OSB production in 2003 accounted for 37% of the total global production ($2.4 billion USD), OSB is a vital value added product of the Canadian forest industry. A fundamental understanding of the moisture-related properties of OSB and how they relate to manufacturing is needed in order to protect and expand the industry.

The two main areas where an understanding of the moisture related properties are essential are in the design of buildings and in the manufacture of the building materials. The key moisture-related hygroscopic material properties are permeance and sorption. Water vapour permeance is defined as "the timed rate of water vapour transmission through unit area of flat material or construction induced by unit vapour pressure difference between two specified surfaces, under specified temperature and humidity conditions" (ASTM E96-95). In other words, permeance is a measure of the water vapour flux through a given thickness of material, due to the mechanism of water vapour diffusion. Permeability is the arithmetic product of permeance and thickness. Sorption, the combined effect of absorption and adsorption, is a property which relates the amount of moisture which a material will store, to the specific relative humidity and temperature of the environment. This study aims to determine what are the permeance and sorption properties of various compositions of OSB, and specifically how these are related to the manufacturing parameters in the OSB mill. Although sorption properties were measured, they will not be dealt with in this paper.

2 METHODS

2.1 Test Panel Manufacture

The trial panels were specifically manufactured for this project at a commercial OSB mill under controlled conditions. All of the relevant mill conditions were carefully controlled and documented while the parameters to be investigated were varied one by one. Research experience has repeatedly demonstrated that panels made on a small scale laboratory press behave quite differently from commercial panels for a number of reasons, such as differences in internal gas pressure, resin distribution, resin addition level and the way in which the mat is formed. The mill-made panels have the same properties and hence they will behave as the real commercial panels.

The panels were all made during a one-day mill trial. The panel thickness and grade chosen for the project was 7/16" (11 mm) PRS (performance rated sheathing). This is the most common panel made in industry since it can be used for the largest range of applications, from wall sheathing to roofs.

The trial panels were made on a 9 foot (2.74 m) by 24 foot (7.31 m) , twelve opening multi-daylight press, with a 153 second pressing time at approximately 205^{o}C. The surface resin was a liquid phenol formaldehyde made by Borden, and the core resin was a methylend diphenyl di-isocyanate supplied by Huntsman Polyurethanes Inc. The wax was a Borden emulsefied wax. The species mix at the time of the trial was 60% aspen (Populus tremuloides), 30% lodgepole pine (Pinus contorta), and 10% birch (Betula papyrifera). The core and surface strands were dried to approximately 2% and 3% moisture content respectively before the addition of resin and wax.

The variables selected for this study (Figure 1) were: Panel density; resin content; and surface treatment. Preliminary research indicated these were likely to have the largest impact on permeance and sorption properties, and these were variables which are controllable at the mill. The standard density of a 7/16" PRS grade panel varies greatly in the industry, so the average of 39.0 lbs/ft^3 (626 kg/m^3) was chosen as the mid point or control density for the study. Without varying any other factors, and at a standard resin and wax addition rate, three runs varying only target density were made at 34.5 lbs/ft^3 (554 kg/m^3), 39.0 lbs/ft^3 (626 kg/m^3) and 42.9 lbs/ft^3 (689 kg/m^3). Next, density was returned to the control level at 39.0 lbs/ft^3 (626 kg/m^3), and the resin addition in the surface and core layers was raised to the upper limit. The final variable to be adjusted was the surface treatment, or, in other words, whether or not the panel was sanded after being pressed. This was done again at the standard density and resin and wax settings. At least three press-loads of each trial step were made and set aside for the study.

Figure 1: OSB variables selected for study.

Panel Types / Groups	Target Density (lbs/ft^3)	Surface Resin (% solids)	Core Resin (%)	Surface Wax (% liquid)	Core Wax (% liquid)	Target Thick. (in.)	Press Time (sec.)
Group 1: 34.5 lbs/ft^3	**34.5**	3.0	2.0	1.8	0.6	0.430	153.0
Group 2: 39 lbs/ft^3	**39.0**	3.0	2.0	1.8	0.6	0.430	153.0
Group 3: 42.9 lbs/ft^3	**42.9**	3.0	2.0	1.8	0.6	0.430	153.0
Group 4: Resin	34.5	**4.25**	**4.0**	1.8	0.6	0.430	153.0
Group 5: Top Surface	34.5	3.0	2.0	1.8	0.6	0.430	153.0
Group 6: Core	34.5	3.0	2.0	1.8	0.6	0.430	153.0
Group 7: Bottom Surface	34.5	3.0	2.0	1.8	0.6	0.430	153.0
Group 8: Sanded Top Sfc	34.5	3.0	2.0	1.8	0.6	0.430	153.0
Group 9: Sanded Bottm Sfc	34.5	3.0	2.0	1.8	0.6	0.430	153.0
Group 10: 1-cycle soak	34.5	3.0	2.0	1.8	0.6	0.430	153.0
Group 11: 3-cycle soak	34.5	3.0	2.0	1.8	0.6	0.430	153.0
Group 12: 8-cycle soak	34.5	3.0	2.0	1.8	0.6	0.430	153.0

2.2 Testing at Mill

During the mill trial, the panels were subject to mill quality assurance procedures, both internal and those set by the APA (The Engineered Wood Association). Panels were randomly selected during each phase of the trial, and were graded and structurally tested in accordance with APA and CSA/ASTM D-1037, CSA 0437 and CSA 0325 standards to assure that they met all the requirements to be grade stamped and sold as commercial PRS OSB panels. This aspect was important in assuring that the ranges in which the production variables were set, were within the limits of producing commercially viable panels. After the required storage time in the warehouse for hot-stacking, they were packaged and shipped to Toronto for this study.

2.3 Specimen Preparation

The mill panels from each group were sorted by density, and panels which came closest to matching the target densities of the study (34.5 lbs/ft3, 39.0 lbs/ft3, and 42.9 lbs/ft3) were selected. Later, at the laboratory, the three panel groups of one, three and eight cycle soaking and drying were prepared, again at standard density, resin and wax additions. The panels were first cut into permeance specimen size discs (92 mm diameter), and then submersed in cold water for 24 hours, followed by drying in a laboratory oven at 102°C +/-2°C for 24 hours. The soaking and drying were repeated for the required number of cycles. This moisture cycling method is only to give a hypothetical, worst case scenario, and not intended to simulate real life conditions.

The final variable studied was the individual component layers of OSB. The individual layer specimens (top surface, core and botton surface) were prepared by carefully planing off the other layers, leaving only the layer to be studied.

2.4 Laboratory Testing

A slightly modified form of the commonly used cup test, ASTM E 96-95 (Standard Test Methods for Water Vapor Transmission of Materials) was used to measure the permeance of the OSB specimens (Figure 3). In the standard cup test, the material to be evaluated is sealed as a lid onto a dish or cup, containing either a desiccant (Dry Cup test) or liquid water (Wet Cup test). Next, the whole test assembly is placed into a controlled atmosphere chamber at 50% RH. Periodic weighing of the cup and lid assembly determines the rate of mass gain or loss, and in turn the permeance of the material, but at only two RH gradients, and each spanning a 50% RH range.

The modification to the ASTM cup test method applied in this study, was the use of saturated salt solutions for the control of relative humidity, allowing specimens to be tested through five relative humidity gradient steps, ranging from 2% RH up to 85% RH. It is a property of most salts, that when in the form of a saturated solution within a given temperature range, they create an environment of constant relative humidity above them, unique to that salt. The reason for this test method modification is that OSB, being made of wood, is a hygroscopic material, and as such, it's permeance varries greatly with relative humidity. Therefore, evaluating the permeance of hygroscopic materials across only two relative humidity gradients in accordance to the ASTM Cup Test method is of limited use for the accurate characterization of the material, and for the end goal of using the data for hygrothermal modeling and building design.

In this study, the moisture flux was always maintained in the direction of from the chamber into the cup. Thus, the chamber was always maintained at a higher relative humidity than the cup. Each step of relative humidity is listed in fig. 2, with the first salt listed as the one in the chamber, and the second listed as the one in the cup, and the resultant relative humidities of each below. The average specimen relative humidity is the arithmetic mean of the cup and chamber relative humidities, and is that which would be measured at the midpoint of the specimen. A desiccant consisting of molecular sieves was used in the cup in the first step to attain a relative humidity as close to zero as possible. The same specimens were used throughout the experiment at every relative humidity step.

Fig.2: Salt solutions used and the relative humidities of each.

Salts in Chamber: Salt in Cup:	**$CaCL_2$ - Desiccant**	**$MgNO_3$ - $CaCL_2$**	**$NaNO_2$ – $MgNO_3$**	**NaCl - $NaNO_2$**	**KNO_3 – NaCl**
Chamber RH:	28.00%	50.00%	60.00%	70.00%	85.00%
Cup RH:	2.00%	29.00%	53.00%	64.40%	75.00%
Avg Specimen RH (%)	**15.00%**	**39.50%**	**56.50%**	**67.20%**	**80.00%**

The constant humidity and temperature chamber consisted of a plexiglass box, with dishes of saturated salt solutions for relative humidity control and two circulating fans (Figure 4). The chamber was in turn placed in a guard room within a temperature controlled room set at 21°C. The temperature inside the chamber was controlled by means of small sheilded lightbulbs, activated by a control program, which maintained the temperature of the chamber at 25°C (+/- 0.5°C). RH was monitored with a Vaisala RH meter and temperature probe, and verified periodically by means of a sling psychrometer.

Fig. 3: Test cup and specimen.

Fig. 4: Temp and RH chamber.

3 RESULTS AND DISCUSSION

Permeance specimens exhibited a very steady rate of mass gain over the entire test period for each relative humidity step. Each series shown is the average of the results from five specimens (Figure 5).

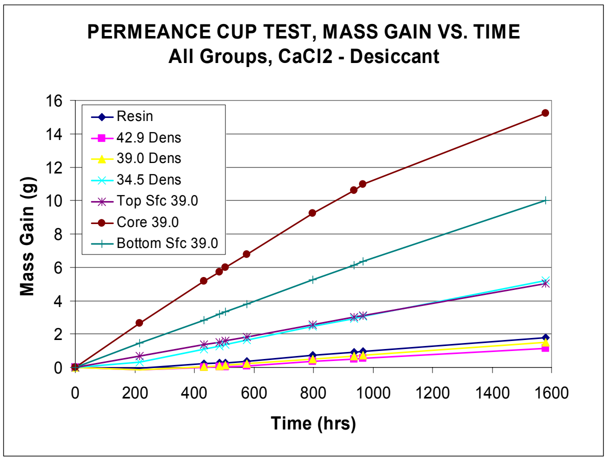

Fig. 5: Mass Gain of Specimens over 66 days, at first RH range (2% - 28%).

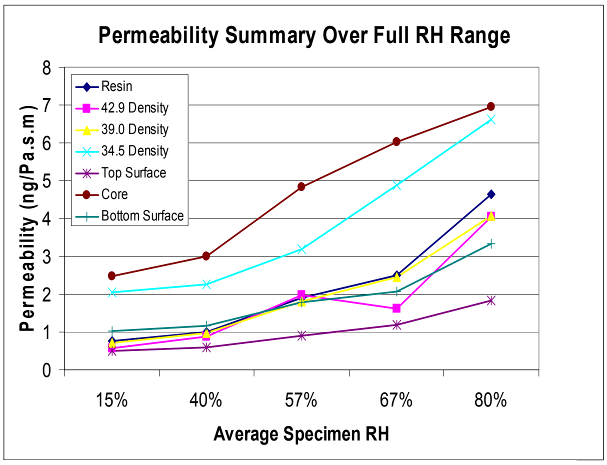

Fig. 6: Permeabilities of Various Groups over Full RH Range.

Each five specimen average plot will yield one permeance or permeability value at that given relative humidity and relative humidity gradient. For the purposes of comparison, the mass data has been converted to permeability and a series of plots over the entire relative humidity range as shown in figure 6.

As would be expected with a hygroscopic material, the permeability in each group increases with relative humidity. This behaviour of hygroscopic materials can be explained by the various moisture transport mechanisms involved (J. Arfvidsson, 1989), but will not be dealt with in this paper. As the relative humidity increases, permeability increased by approxmately three-times in the core layer group, and up to a seven times in the 42.9 lbs/ft^3 (689 kg/m^3) “higher” density group. These results underscore the need to know the permeance of hygroscopic materials such as OSB over the entire relative humidity range.

Another, perhaps more important finding is the difference in the permeabilities among the various series. Where only density was varied, it was found that the lowest density group, 34.5 lbs/ft^3 (554 kg/m^3), shows the highest permeability. The highest density group (42.9 lbs/ft^3, 689 kg/m^3) had the lowest permeability and the mid density group, (39.0 lbs/ft^3, 626 kg/m^3) was in between. As intuitively expected, the relationship between density and permeability is inverse. The higher the density of a given material, the lower the permeability. This is due to a reduction in the number of voids or free paths not blocked by material through which water vapour can diffuse. The relationship can more easily be seen by plotting the densities of individual specimens against permeability at each relative humidity range (figure 7). The relationship between permeability and density can best be described by a logarithmic function for each RH step, as shown in Figure 7.

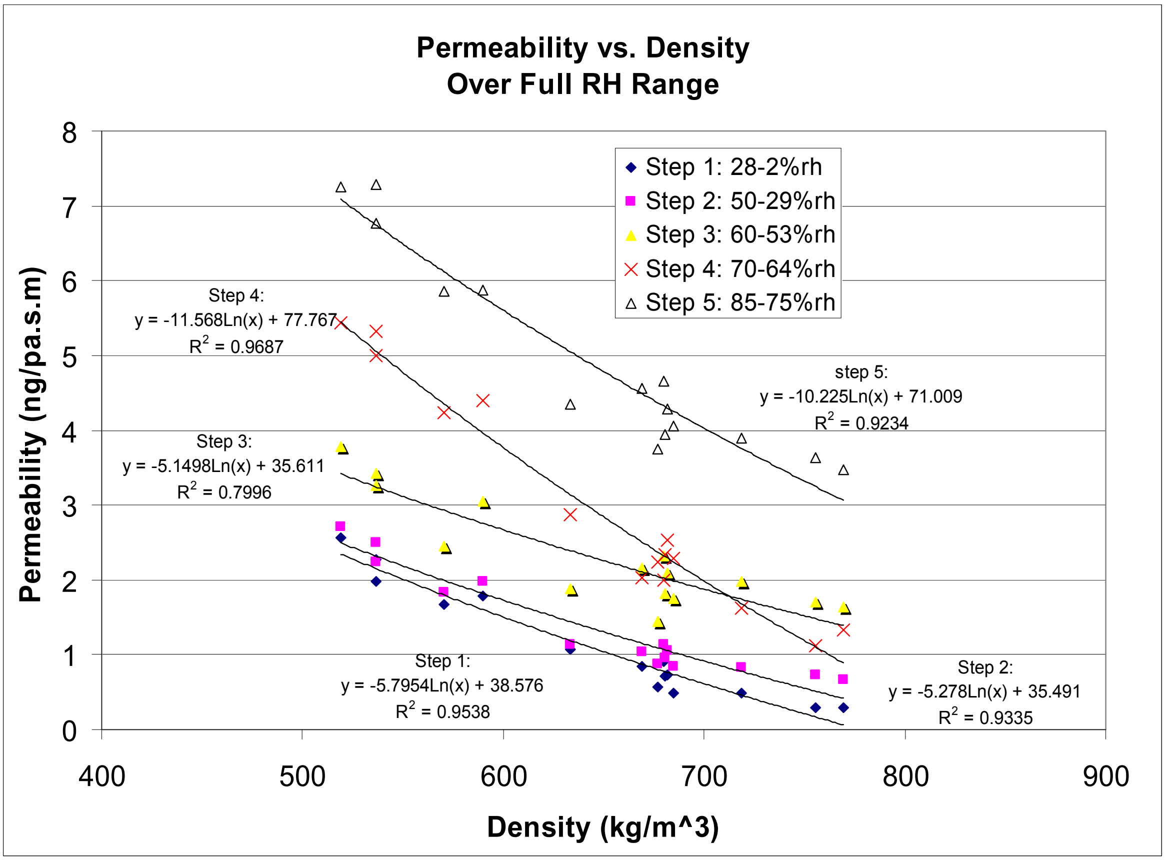

Fig.7: Permeability vs. Density for Various Relative Humidity Ranges.

An examination of Figure 6, again reveals that each component layer has a very different permeability. The experimental results clearly show that the core layer has the highest permeability and the top surface layer has the lowest permeability of all the series, throughout the relative humidity range. Thus, OSB is not a uniform homogenious material; OSB should be seen as being composed of three distinct individual component layers. Each layer has very different properties that result from the manufacturing process. First, the resin and wax addidion levels are not the same for all layers. Surface layers typically have 50% more resin and 200% more wax than the core layer. Second, there is a large variation in density throughout the thickness of OSB. Surface layers are significantly more dense than the core. This is because of the heat transfer process during pressing, resulting in the outer surfaces plasticising and densifying to a higher degree than the core at the center of the panel. Third, there is also a difference in the very outer surfaces as a result of how the panel is pressed. The very top surface has a glassy smooth finish since it is in direct contact with the hot metal platen of the press. The bottom surface, with the exception of panels made on a continuous press, will have a rough, textured surface from having been pressed on a caul screen used with a multi-daylight opening type press. The other possible variations in moisture performance properties may be due to the migration of resin, wax, gases and moisture during pressing.

The final series in Figure 5 has been called "resin". The resin-rich samples have been made by raising the resin addition rate for both the surface and core layers to the upper end of the commercially feasible limit. These high-resin specimens showed little difference in permeability from the 39.0 lbs/ft^3 (626 kg/m^3) density control group, indicating that the resin addition level has little influence on permeability of OSB as compared to density.

The impact of cyclic wetting and drying, as well as of sanding on permeability are illustrated in Figure 8. These tests were carried out at the middle relative humidity ranve of 49% - 29% rh. This series of tests was conducted using standard density (39.0 lbs/ft^3, 626 kg/m^3) and resin addition levels (3.0% surface, 2.0% core), and can be compared against the control (standard density and resin content, non-cycled 7/16"PRS). Cyclic wetting and drying clearly has a large effect on permeability. Following one cycle of wetting and drying, permeability more than doubled. Subsequent cycles had a diminished effect.

Sanding on the otherhand, had a far lesser effect on permeability. However, recalling the variation in permeability between the various component layers illustrated in figure 6, one would expect that a removal of the top or bottom surface material should change the overall permeability. The degree of sanding or the amount of surface material removed may not have been sufficient in this preliminary investigation to have shown a significant impact, and will have to be further studied.

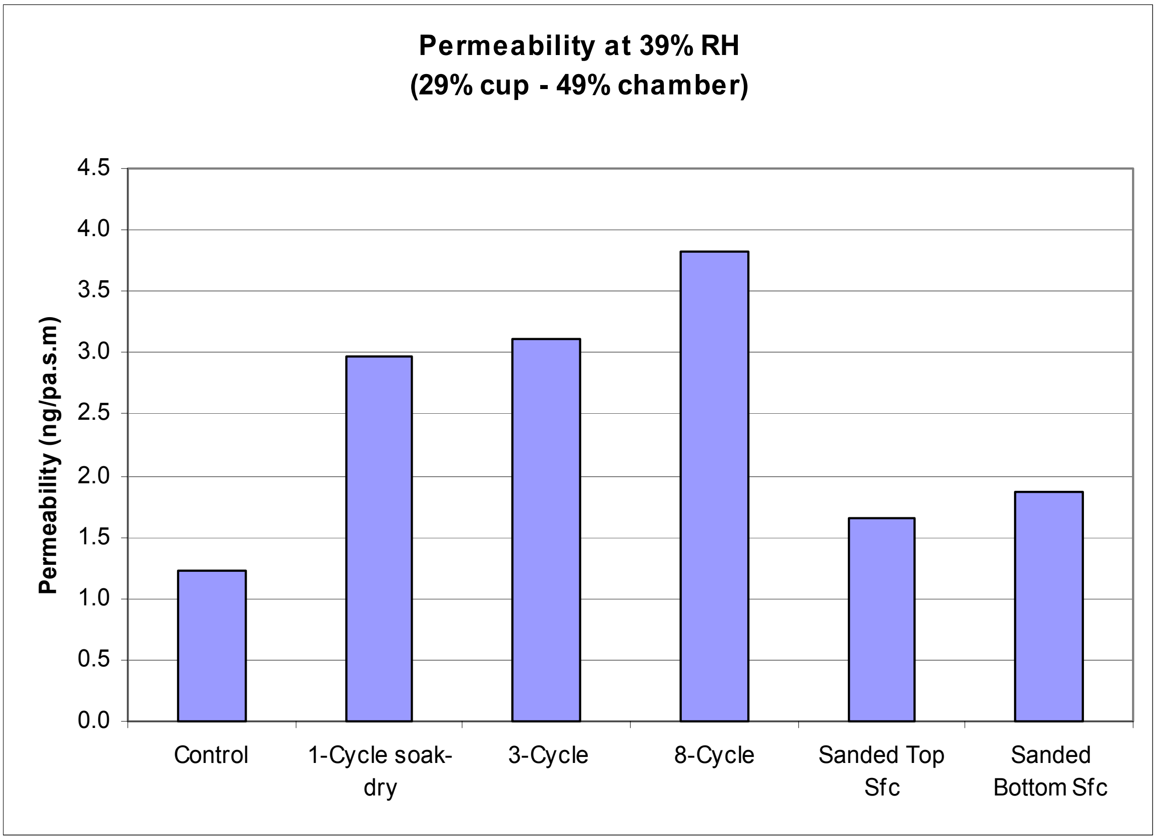

Figure 8: Effect of cyclic soaking and drying, and sanding on permeability.

4 CONCLUSION

The permeability of OSB can vary several fold as a result of variation in the manufacturing parameters at the mill. Further, it varies significantly when subjected to cycles of wetting and drying. This strongly suggests that in addition to virgin material moisture properties, those which restul from wetting and drying should be considered in design and analysis. A simple factor such as whether or not the OSB has been sanded or not, may alter the permeability slightly, depending on the degree of sanding. When wall, roof or floor systems are designed with OSB, these variations in permeability may have an impact on the overall moisture performance of the wall system, and ultimately on whether or not mould, mildew or rot will develop under adverse service conditions. Ultimately, both the designer and the OSB manufacturer can take advantage of these variations in permeability in order to optimize building systems for occupant health, efficiency and durability

5 ACKNOWLEDGMENTS

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6 REFERENCES

APA- The Engineered Wood Association. 2003. *Performance Standards and Policies for Structural Use Panels.* APA PRP-108. Tacoma, WA: APA- The Engineered Wood Association.

Arfvidsson, J. 1989. *Computer model for two-dimensional moisture transport. Manual for JAM-2.* Department of Building Technology, Lund University, Lund.

Straube, J.F. 1998. *Moisture Control and Enclosure Wall Systems.* Ph.D. Thesis, Civil Engineering Department, University of Waterloo.

A comparative study of reinforced concrete beams strengthened with glass, carbon and Aramid fibers

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ABSTRACT

The repair and structural strengthening techniques using non-metallic materials, such as Fibre Reinforced Polymers (FRP) are focusing the attention of researchers all over the world. The use of these innovative materials is stimulated by their very good mechanical performance and by the fact that they show some particular properties, like a high corrosion resistance, that enables them to overcome or minimize some of the deficiencies of the techniques used nowadays. The Carbon Fibre Reinforced Polymer (CFRP) is currently the most used system. However, since the cost of the carbon fibre is still quite high, other alternative fibres are being investigated. Among the large diversity of fibres found in the market, the ones that stand out are the glass and aromatic polyamides (Aramid®), which have a considerable resistance and a more competitive cost. Given the fact that each fibre can be used to create a composite that has some specific advantages, it may be interesting to develop the technology to use each one of them, while researching the specific features of each composite, collecting data to help ensure the quality and efficiency of the reinforcement operation.
The main purpose of this work is to develop a comparative study focused on the flexural behaviour of beams strengthened with glass, carbon and Aramid fiber composites. The experimental program is based on four point bending tests performed in 7 x 14 cm beams, strengthened with one layer of the fibre composite. The results indicate that both the glass and the Aramid fibres can be used to create functional and efficient reinforcement composites. All strengthened beams presented considerably higher ultimate loads. The main conclusion is that it is possible to create competitive repair and reinforcement alternatives using different fibres to structure an epoxy matrix.

KEYWORDS

High performance fibre composites, glass fibre, Aramid fibre, carbon fibre, reinforcement and strengthening techniques.

1 INTRODUCTION

Researchers all over the world are dealing with the study of the miryad of new materials developed during the last few decades. In the construction industry, some of the main developments that have occurred were related to new chemical addititives and fibre reinforcement polymers (FRP). FRP are composite materials created by embedding high performance fibres in an epoxy or polyester matrix. These lightweigth and high strength materials offer new and interesting alternatives for the construction of new structures or for the rehabilitation of corroded or degraded concrete elements.
FRP composites currently used in the construction industry are normally being structured with carbon fibre, because this material has a high modulus of elasticity and very high tensile strength. However, the relatively high cost of these composites is inhibiting their dissemination, especially in third world countries which have to import all the components of the system.
Therefore, the use of CFRP (carbon fibre reinforced polymer) is limited to very critical situations, where a high resistance is necessary and the access for performing the rehabilitation operation is not very good, or when the environmental conditions are very aggressive.
Searching for a more economic alternative, the research team at LEME (Laboratory of Testings anf Structural Models) started a research program aimed at comparing the performance of CFRP with composites structured with aramid and glass fibres. The experimental program described in this paper was designed so that the performance of CFRP, AFRP (aramid fiber reinforced polymers) and GFRP (glass fiber reinforced polymers) reinforcement composites could be compared. To this end, six reinforced concrete beams (7 by 14 by 130-cm long.) were cast of which one (1) was left unstrengthened, wheras the other 5 were strengthened in flexure using carbon, Aramid or glass fiber materials. The beams were subjected to 4-point bending tests, with the evolution of middle point deflection and of the specific strain on the composites being monitored.

2 STRENGTHENING WITH CARBON, ARAMID AND GLASS FIBER REINFORCED POLYMERS

Fiber reinforced polymers (FRP) have some features that make their use advantageous when compared to other repair and reinforcement techniques that use steel or concrete. Among these features are the high tensile strength and modulus of elasticity that enables them to achieve great structural performance. Meier [1995] adds that these materials have outstanding fatigue performance and a reasonably high stiffness.
The low specific weight and the short setting and curing times make the transport, handling and application of the system quite easy, even in restricted spaces. Moreover, the low weight to resistance ratio allows the use of fine sheets to carry the load. Therefore, the final thickness of the composite is very small and does not change the dimensions of the structural member or affect its connection with other structural elements.
Furthermore, when an epoxy matrix is employed, the composite gains a chemical resistance to many common aggresive agents, such as chlorides or acid solutions, not requiring any special protection even in aggressive environments.
Due to all of these characteristics, CFRP composites have proved to have a very good performance as reinforcement elements, as demonstrated by Meier [2002] and Beber [2003]. There is still the need for further research to understand how these composites will age and to check if their performance will be maintained over time. Nonetheless, they are already being used in some challenging situations.
One of the main obstacles to the larger diffusion of this technology, however, is the relatively high cost of its components. A commercial CFRP system can cost close to U$ 60,00 per square meter.
With this in mind, studies have begun to look at the use of alternative, less expensive, fibers, in order to increase the competitiveness of the system. Among the diversity of fibers available in the market, there are two that have considerable strength and have already been used by the LEME team as addition to concrete mixes, with very good results [Bernardi, 2003]. Therefore, the present study was focused on the investigation of the performance of reinforcement composites created by adding aromatic polyamides (popularized under the commercial name Kevlar© by Du Pont) and glass fibers to epoxy matrices.

It is necessary to carry out comparative studies before using these new fibers, since the specific characteristics of each fiber can influence the behavior of the resulting composite, as shown by the following examples:

a) The thermal expansion coefficient of carbon FRPs are nearly zero, while glass FRPs have thermal expansion coefficient similar to concrete;
b) Glass and aramid FRPs are electrical isolators while carbon FRP is an electrical conductor;
c) Glass and aramid fibers show better impact performance when compared to carbon fibers, which are brittle and more resistant to small elongations;
d) Carbon FRPs show better performances when creep and fatigue failures are analyzed. Aramid fibers have an intermediate behavior while glass fibers are more susceptible to these kinds of damages;
e) The density of carbon FRPs is located between the densities of aramid and glass FRPs;
f) The tensile and compressive strengths of carbon fibers are higher than the respective strengths of the other two fibers.

Meier [1995] compares some qualitative aspects of carbon, aramid and glass FRPs in Table 1.

Table 1. Qualitative comparison between CFRP, AFRP and GFRP (Meier, 1995)

Criterion	FRP made of		
	Carbon	Aramid	E-Glass
Tensile strength	very good	very good	very good
Compressive strength	very good	inadequate	good
Modulus of elasticity	very good	good	adequate
Long term behavior	very good	good	adequate
Fatigue behavior	excellent	good	adequate
Bulk density	good	excellent	adequate
Alkaline resistance	very good	good	inadequate
Price	adequate	adequate	very good

Each fiber can be employed to create a different composite, which have specific advantages for some kind of use. That means that it is interesting to develop technologies to use each one of them and also that it is necessary to research the specific features of each of the composites formed.

It is important to highlight that the applicability and effectiveness of the strengthening techniques using FRP composites depend largely on the material and on the type of the concrete member to be strengthened. For example, the strengthening material is generally expected to have a similar or higher stiffness compared to the concrete base.

From a structural point of view, an important concern is related to the prevention of brittle de-bonding failures. Such failures, unless adequately considered in the design process, may significantly decrease the effectiveness of the strengthening. In recent years, many researchers have focused on this important issue through both experimental and theoretical investigations [Buyukozturk *et al.* 2004].The mechanism of premature failures caused by the introduction of shear stresses in the composite due to differential movements in the borders of a crack has also been observed and is being examined by the team at LEME.

In addition to this, perhaps one of the most important unresolved questions is related to the fire resistance. Epoxy matrices start to degrade at 80°C, burning almost totally when the heat is higher than 300-400°C. Protective measures must then be used to ensure that eventual fires will not damage or compromise the strengthened structures.

Given these lingering doubts, Nanni [2003] believes that further research and validation is still necessary to increase confidence in FRP technology for concrete construction.

3 MATERIALS AND METHODS

As mentioned before, the experimental program was based on bending tests performed on six (6) reinforced concrete beams. One of these beams did not receive any strengthening, being used as a control beam. The other beams received flexural strengthening of carbon, aramid or glass fiber.

3.1 Test Beams

The test beams had a cross section of 7 by 14-cm, with a span of 130cm. The lower longitudinal reinforcement consisted of 2-8-mm diameter steel bars (CA-50). The shear reinforcement consisted of 4.2-mm steel stirrups, each spaced at 4-cm. The reinforcement cover was 1-cm.

3.2 Strengthening Systems

The carbon fiber system used as reference was a commercial system manufactured by Mitsubishi. The other systems were created in the laboratory using aramid fiber sheets provided by Du Pont and glass fiber sheets by Texiglass. Each of these systems is shown in Figure 1.

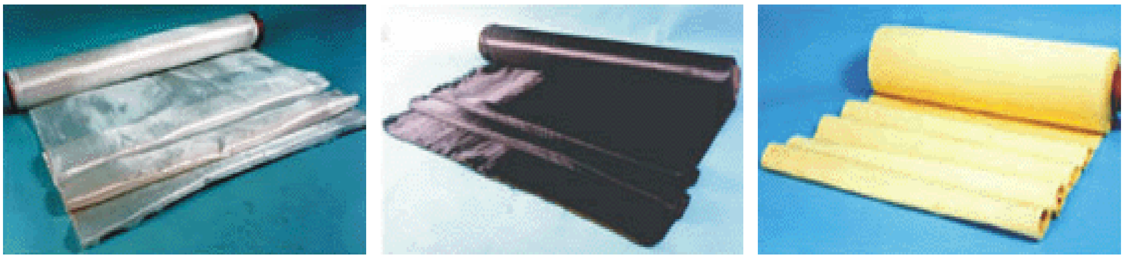

Figure 1. Glass, carbon and aramid fiber sheets used in the strengthening

For the creation of the matrix of the AFRP, a two component epoxy formulation was selected. To ensure that the aramid fiber sheet, which was quite dense, would be properly involvedby the matrix, the formulation chosen had a very low viscosity. Since the glass fiber sheet was more open, with the individual strands being more spaced, a more viscous formulation had to be chosen for the creation of the matrix of the GFRP. A polyamide epoxy resin formulation, with primer, manufactured for use with carbon fiber sheets was therefore selected. Table 2 shows the properties of the fibers and epoxy systems employed, according to the manufacturers.

Table 2. Properties of the materials used in the strengthening

	Glass Fiber TRB 600 - 140	**Carbon Fiber Replark 20**	**Aramid Fiber AK - 40**
Flexural Strength (MPa)	3400	3800	3620
Thickness (mm)	0.74	0.176	0.196
Young's Modulus (MPa)	72400	240000	130000
Weight (kg/m²)	0.60	0.2	0.45
Ultimate Elongation (%)	3.5	1.58	2.3
	Epoxy Resin System 1	**Epoxy Primer**	**Epoxy Resin System 2**
Components	Araldite LY 1564 Aradur 955	Mbrace Primer - A Mbrace Primer - B	Mbrace Saturant - A Mbrace Saturant - B
Flexural Strength (MPa)	106	24	124
Ultimate Elongation (%)	6.5 – 7.5	3	2.5
Young's Modulus (MPa)	2600-2800	717	3034

3.3 Concrete Production

The beams were cast using a 30 MPa concrete. The concrete mix was determined according to the IPT Method (Helene and Terzian, 1992). To achieve the required resistance, the chosen mix proportions were 1:2.12:2.88 (type V Portland cement: sand river fine aggregate: basalt coarse aggregate), with a water-cement ratio of 0.45. The workability was set with a slump value of 80 ± 10-mm.

3.4 Strengthening Application

The application of the strengthening composite systems was made according to the manufacturer's instructions, which included the following steps:

a) Surface preparation.
b) Primer application (in case of the carbon and glass strengthening composite systems).
c) Application of the first layer of saturation resin.
d) Application of the fiber sheet.
e) Application of the second layer of saturation resin.

2.5 Instrumentation

To obtain maximum deflections at mid-span, a digital deflectometer, with a 0.001mm precision, was placed on the lower face of each beam, ,. Strain gages were used to monitor the specific strains in the strengthening. The data from the strain gages and load cells, used to control the loading, was collected by a dedicated computer controlled data acquisition system

2.6 Test Procedure

The load application was made using a 4-point bending configuration with the loads being applied symmetrically on each third of the span. To permit ease of reading the deflectometer, loading steps chosen at every 5 kN .
During the test, the evolution of cracks was monitored and noted on the beams. Special attention was paid to any signs of premature failure.
Figure 2 shows the scheme of the test procedure.

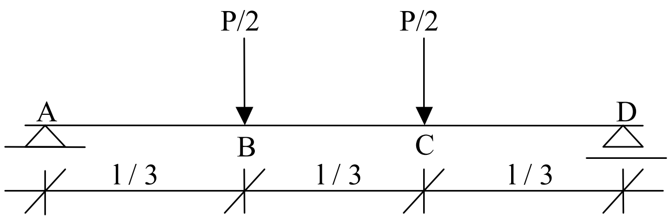

Figure 2. Scheme of the test procedure

4 RESULTS AND DISCUSSION

Unfortunately, a problem with the application of the carbon fiber system in one of the beams was noticed and that prototype was discarded without being subjected to the test

4.1 Mid-span deflection

Figure 3 shows the plots of load versus deflection for all beams tested.
The results indicate a slight increase in the stiffness of all strengthened beams, when compared with the control beam, especially after the 20 kN thresholds.
All strengthened beams presented similar behavior, indicating that the replacement of carbon fiber by aramid and glass fibers is feasible. In fact, up to a load of 50 kN, the lower deflections were observed

in one of the glass fiber a strengthened beam, which was an interesting observation, since the glass fibers had the lowest Young's modulus.

The maximum deflection was observed in the aramid 1 strengthened beam, which reached a deflection higher than 18 mm before failure. This beam also showed the second lowest deflections up to 50 kN, and the lowest deflection in 50 kN. Nonetheless, the other AFRP was the one with the highest deflections, indicating that the quality of the application can be decisive in the performance.

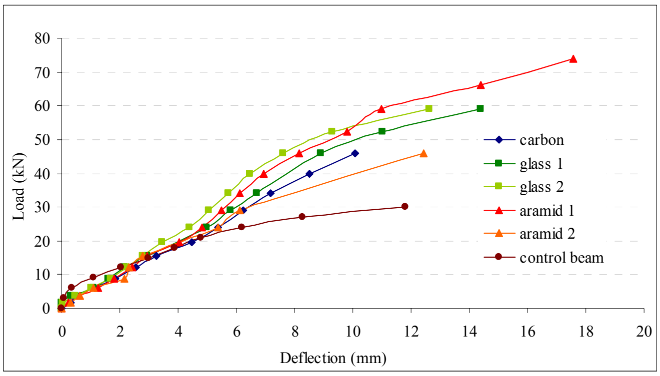

Figure 3. Load x deflection plot for strengthened beams

4.2 Mid-span strain

Figure 4 presents the evolution of the strains in the FRP composites at mid-span. It is important to highlight that the data goes up just until the failure of the strain gages, which happened normally when the elongation exceeded 1.3%.

As can be noticed in the figure, the values of the maximum strains recorded were lower than the ultimate elongation values given by the manufacturers (see Table 2), which varied from 1.6 to 3.5%. Projecting the expected strain values for the ultimate load achieved (see Table 3), the strain values would probably be quite close to the reference values given for each fiber. The problem is that the strain evolution is somehow limited by the occurrence of compressive failures in the concrete, observed in four of the beams (both AFRP and both GFRP). The deflections before the failures increase considerably, leading the strain in the composites to evolve towards their limit.

The maximum strain value recorded was obtained by one of the glass fiber composites, which reached 1.43%. However, this beam was clearly less rigid than the others, indicating that the original concrete beam might have had some defects or is less resistant.

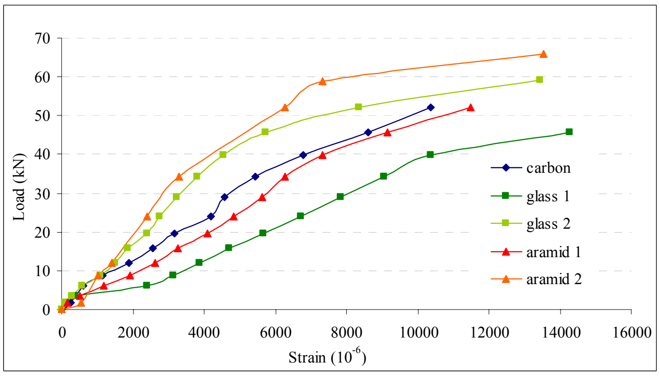

Figure 4. Load x strain plot for strengthened beams

4.3 Ultimate Load

The ultimate loads registered in the tests can be observed in Figure 5 and Table 3. Information on the failure mode of each beam is also indicated in the Table. As can be seen, the beams strengthened with

AFRP composites showed a very good performance, reaching ultimate loads of 79 and 90 kN, at the point of compressive failure in the concrete.
Failure of the reinforcement was just recorded in the case of the CFRP beam, when the load reached 65 kN. The mode of failure and aspect of the fracture surface indicated the possibility of a shear failure caused by differential displacements in crack tips. However, the strain on the composite was probably very near the maximum value admitted by its manufacturer.

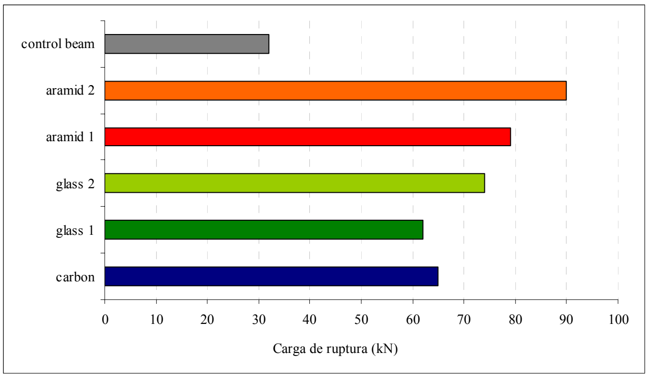

Figure5. Ultimate load for strengthened beams

Table 3. Ultimate loads and failure modes of the strengthened beams.

Beam	Ultimate Load (kN)	Failure Mode
Aramid 1	79	Concrete compressive failure
Aramid 2	90	Concrete compressive failure
Carbon	65	FRP premature failure
Control Beam	32	Yielding of the steel
Glass 1	62	Concrete compressive failure
Glass 2	74	Concrete compressive failure followed by FRP failure

As was previously stated, the crack-induced shear failure mechanism, which can be seen in greater detail in Figure 6, is being studied and modeled by the LEME team with the aim of proposing application techniques and design guidelines that avoid this type of occurrence and allow the beams to reach their full flexural capacity.
According to Wong and Vecchio [2003], the failure mode of reinforced concrete members is governed by the local bond stress–slip relationship at the interface. However, one must take into account the relative displacement between FRP and concrete.

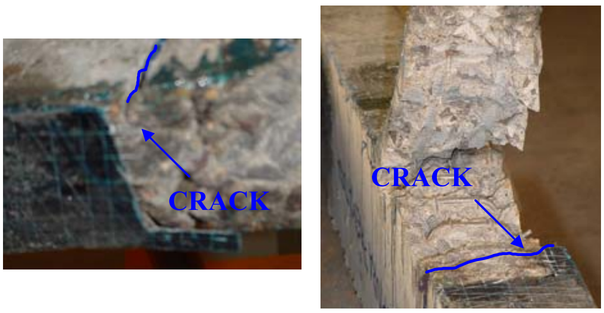

Figure 6. Premature failure in the CFRP strengthened beam

In general terms, the performance of the GFRP beams was quite similar to the CFRP beam, with the ultimate failure load values being equivalent. The AFRP beams, otherwise, has a superior performance in terms of ultimate load values. The load majoring coefficients (increase in ultimate bending load) for the AFRP beams were 146% and?? 181%, 103% for the CFRP and 93% and?? 131% for the GFRP.

4.4 Crack patterns

The influence of the external strengthening composites on the development of the cracks was found to be significant. Comparing the crack patterns of the strengthened and control beams, it can be clearly seen that the cracking is more diffuse and evolution of the cracks is retarded in the strengthened beams.
The first crack was noticed on the control beam when the load reached 12kN. On the CFRP and GFRP strengthened beams the first cracks appear when the load was close to 35 kN. For the AFRP, cracking was recorded just after 50 kN.

4.3 Costs

The cost analysis presented in Table 4 shows the relative costs of each fiber type. Assuming a generic cost for the epoxy formulations (circa U$ 24.00/kg in Brazil during october/2004), it can be concluded that GFRP composites are much less costly than other composites, with AFRP composites costing 60% the value of CFRP composites.

Table 4. Cost comparison among fibers (values for Brazil during october/2004)

Fiber Type	Unitary cost ($U\$/m^2$)
Carbon	42.00
Aramid	25.00
Glass	2.00

5 CONCLUSIONS

The results obtained clearly show that composite strengthening systems work well with any of three fibers tested. Since the glass fiber is much less costly that the other types of polymer-based fibre systems, glass fibre reinforcement becomes very competitive in terms of cost versus performance. Nonetheless, there are some studies that indicate that glass fibers might be more prone to sudden failures under sustained long-term loads. Further research is necessary to ascertain if the GFRP can be used with confidence in the long-term. Nonetheless, it appears that when the necessary load increase is not very high, the use of these less costly strengthening systems offer a competitive advantage over other FRP systems such as CFRP.
The results also show that the mechanism of premature shear failure might be a limiting factor, especially when dealing with CFRP. AFRP have presented the best performance and seen to be a well-balanced solution, with very good resistance and durability, and a moderate cost.
All reinforcement composites increased the stiffness and reduced the central displacements. The ultimate loads of the strengthened beams were more than double that of the control beam, with the concrete compression failure becoming a limiting factor. It is important to emphasize that there are still some unresolved issues regarding the use of externally bonded FRP composites, such as long-term durability studies, analysis of the performance under dynamic loads; development of standards for testing and quality assurance and investigation of the causes of premature failures, especially those associated to crack displacement.

6 ACKNOWLEDGMENTS

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7 REFERENCES

TT3-142, A Comparative Study of reinforced concrete beams strengthened with glass, carbon and aramid fibers, M.R.Garcez , L.C.Meneghetti, L.C.P. Silva Filho, J.L. Campagnolo.

Beber, A. J. 2003, 'Comportamento estrutural de vigas de concreto armado reforçadas com compósitos de fibra de carbono', *Doctoral Thesis*, Porto Alegre – RS – Brasil.

Bernardi, S. T.; Filho, L. C. P. S. 2003, 'Potencial de utilização da fibra de aramida no reforço de estruturas de concreto armado', In: VII Congresso IberoAmericano de la Construcción, Mérida. Anais... [CD-Rom]

Buyukozturk, O.; Gunes, O.; Karaca, E. 2004, 'Progress on understanding debonding problems in reinforced concrete and steel members strengthened using FRP composites', *Construction and Building Materials*, 24, 9 - 19.

Helene, P. E. & Terzian, P. 1992, *Manual de dosagem e controle do concreto*, PINI, São Paulo.

Meier, U. 2002 'The use of composites in public works', In: ARQUIMACOM, São Paulo. Anais... [CD-Rom].

Meier, U. 1995, 'Strengthening of structures using carbon fibre/epoxy composites', *Construction and Building Materials*, 9, 341-351.

Nanni, A. 2003, 'North American design guidelines for concrete reinforced and strengthening using FRP: principles, aplications and unresolved issues', *Construction and Building Materials, 17.* 439 – 446.

Wong, R. S. Y. & Vecchio, F. J. 2003, 'Towards Modeling of Reinforced Concrete Members with Externally Bonded Fiber-Reinforced Polymer Composites', *ACI Structural Journal,* 100-s6, 47 – 55.

A Rediscovery for Fenestration Installation; Correcting the Mistakes of the Past Century

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ABSTRACT

An investigation of the evolution of installation methodologies for fenestration products over the past century leads to new conclusions regarding the value of previous installation methodology. In fact, many of the technology developments over this time period have resulted in a decline in the effectiveness of the fenestration products to divert moisture intrusion into buildings. Fenestration systems of the past (before 1910) were uniquely designed to capture and manage the inevitable moisture leaks that occur, either through or around the fenestration product, and skilled craftsman were employed to ensure this effectiveness. The Industrial Revolution, however, brought on the development of complete pre-assembled fenestration products that were then installed into a building opening, without consideration for the specific details of the installation. As a result of this design change, and combined with lower skilled workmanship, moisture intrustion problems involving fenestration products have become chronic. This study uncovers key design and fabrication changes in fenestration technology over this time period that has resulted in this situation. We then present a revolutionary new installation methodology that takes advantage of the water management principles of the past, combined with the fabrication efficiencies of today.

KEYWORDS

Fenestration, Flashing, Interface, MIFS, Receptor, Water Management, Windows

1 INTRODUCTION

Prior to the twentieth century, fenestrations were created by a two-part mechanism, in which the builder would include the framework of fenestration as part of the building construction process. The fenestration was not an item that was installed separately, but in fact, carefully blended into the "skin", or the envelope, of the structure. To complete the fenestration process, the builder simply installed a sash that was typically produced in a mill. Sashes were stand-alone items, sometimes glazed prior to delivery. Once in receipt of the sash, the builder would simply install them into the framework that was already part of the building and already included sloping sills and complete integration to the building exterior envelope. The integration of the framework included a variety of products to ensure continuity with the siding.

As part of the industrial revolution in the early 20th Centruy, sash manufacturers expanded by creating frames that were pre-assembled and included the hardware and glazing. The sash then were pre-assembled and pre-installed into the framing, thus delivering a complete unit to the builder. This product gives birth to the modern window or fenestration and has been manufactured consistently for the past century. When the completed window fenestration became accepted, the method of integration of the product to the wall was unclear. A variety of techniques have been used that attempt to integrate the entire fenestration into an ever-growing and more complex variety of wall designs. This has led to a number of installation defects and building failures, due to the complexity of using a standard installation method that effectively integrates the fenestration product with the building envelope to prevent moisture intrusion. For many reasons, making the fenestration as a complete unit that includes frame, sash, and hardware, as a permanent part of the envelope is highly problematic.

This presentation will spotlight a completely new method of installing fenestration products, the principle of which goes back more than one hundred years, but encapsulates and enhances current methods of integration with the wall envelope. The new method features a permanent frame to the building, which allows simple and non-invasive replacement of the fenestration product at the end of its useful life. This frame is constructed of a highly durable polymeric material that fully encloses the fenestration product in a continuous, moisture tight encasement that will be much more robust than the methods of integration today. The audience will see a demonstration of the integration of the frame into the wall envelope, and then see the installation of self-contained sash fixtures, literally plugged into the frame, and easily removable.

There is a plethora of benefits to this new method. For example, this method will provide enhanced moisture management, and thermal / accoustical performance. Installation benefits will also be realized, such as a simplified, more standardized method that enables more flexible sequencing of the fenestration installation. Thus, damage to the fenestration product is greatly reduced by intsalling in a sequence that reduces exposure to parts of the construction phase, such as installation of the siding, that can cause premature damage to fenestration product. In addition, the cost of installing fenestration may be reduced, as the installation method will cater to a lesser skill level. Initially, these frames are more suited to standardized fenestration sizes and configurations, making the pre-fabricated or large volume builder the optimal user for this system.

2 BACKGROUND – Evolution of Window Design

Figure 1. Example of typical details available circa 1900 through published books usually written by architects. This method always had the sill running through and beyond and under the balance box and pulley rails, by at least 150 mm, approx. 6 inches, thus diverting corner leakage to the outside, well beyond the plane of the sash.

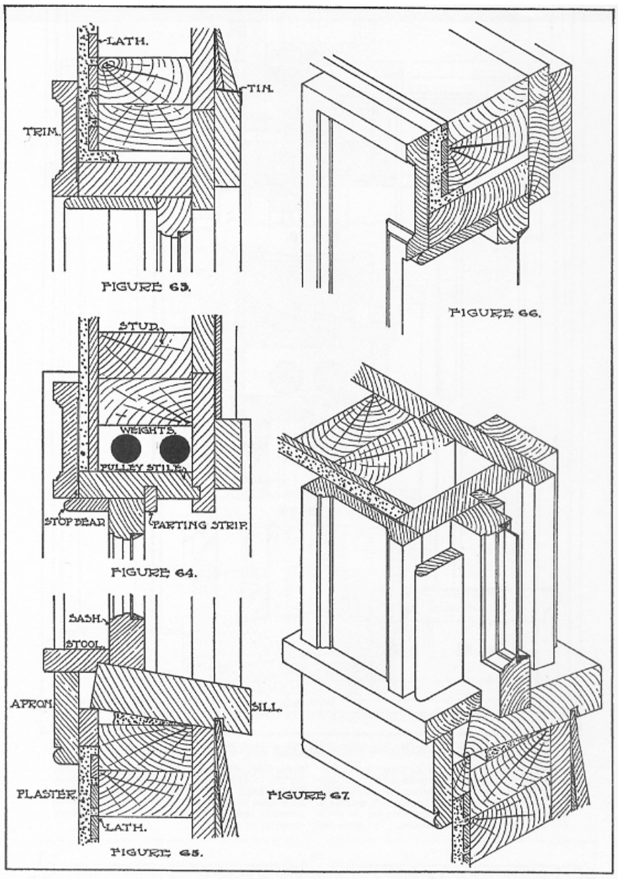


PLATE 51—CHEAP DOUBLE HUNG WINDOW

Arrangement and construction for ordinary inexpensive work, using skeleton frame without ground casings. Fig. 63, section through window head. Fig. 64, section through jamb. Fig. 65, section through sill. Fig. 66, isometric view of window head. Fig. 67, isometric view of jamb and sill. Note tin flashing above window and rabbeted sill to keep out water.

As part of the industrial revolution at the beginning of the twentieth century, evolution dictated that the fenestration products we know today - a single unit consisting of the frame and the sash - was born. As a result of this, the construction methods changed, becoming more industrialized and considered improved. Prior to the turn of the twentieth century, fenestrations were created by a two-part mechanism: Part 1 - the builder, in the construction process, would purchase from a mill those elements that would become the frame and that would be integrated into the building's wall. The frame was not an item that was installed separately and afterward, but in fact, was carefully blended into the "skin", or the envelope of the structure. Those mill items included sills, stools, aprons, jambs or pulley jambs, balance box, parting beads, head stock, blinds stock, and finish trim. Part 2 - to complete the fenestration process, the builder simply installed sash that was typically produced in a mill. Sash were stand-alone items, sometimes glazed prior to delivery.

The builder, once in receipt of the sash, would simply install them into the framework that was already part of the building and already included sloping sills and complete integration to the building exterior envelope. The integration of the framework included mortar grounds, lead or tin flashing materials, and a variety of products to ensure continuity with the siding. Hardware was developed, that would allow the sash to either be balanced for vertical operation or pivoted on any one of four sides to create a variety of operability and ventilation. See Figure 1.

As part of the industrial revolution, fenestration manufacturers replaced sash manufacturers and mills that typically supplied the building trim package, thus a completely assembled fenestration product with glass and hardware was delivered to the job site. These early products gave birth to the modern window or fenestration, which has been manufactured consistently for the past fifty years. The manufacturers addeded features such as integral and non-integral installation fins and a variety of hardware, which made the fenestration easy to operate. As the completed window fenestration became accepted widely in construction, the method of integration of the product to the wall became considerabley more difficult, as an interface needed to be installed, which in essence connected the modern fenestration with the wall. A variety of techniques have been used over the twentieth century, which ultimately ends up with our current methodology, which attempts to integrate the entire fenestration into an ever-growing and more complex variety of wall designs. To this day, we have been unsuccessful in any number of schemes that interface fenestration with the building envelope, as evidenced by the growing number of construction defect claims related to intrusion of moisture through windows and their interfaces.

This paper spotlights the substantial difficulties that resulted when the frame and sash became one unit. Making fenestration as a complete unit that includes frame, sash, and hardware, as a permanent part of the envelope, created unique complications, as well. Fenestration, like many components in a wall, has a useful and predictable life, which is substantially shorter than that of the building it is intended for. Thus, integrating the entire fenestration unit in such a manner as to cause destructive intrusion made it difficult and expensive to remove the product for replacement.. Standardizing a fenestration frame without knowing the installation environment or location of the fenestration in the building was another problem created from this development. Construction sequencing that requires several trades, which typically do not communicate with each other, to work on the rough opening, results in failure to effectively integrate the fenestration with the other wall components; for example, the carpentry, sheathing, membranes, flashings, sealant, and exterior cladding are typically installed by separate contractors. As a consequence, proper sequencing is often compromised.

Fenestrations as we know them today, have limited durability. The consensus is that durability of fenestration would be equal to other items integral to the building, such as water heaters or roofs, which typically have a ten-to-twenty-year life cycle. It is known that the petrochemical portions of fenestration, such as weather-stripping, gasketing, and sealants have limited life cycles, which, when failed, greatly degrade the performance of the fenestration and shorten its useful life. Because of this, one has to wonder: why should it not be as easy to replace a fenestration product as it is to replace other less durable components of the wall, such as light fixtures? Why shouldn't a fenestration product be removed and replaced as simply as or more simply than a water heater, which has approximately the same life expectancy? Shouldn't it be expected to be able to remove a fenestration without causing destruction to a wall and its watertight integrity?

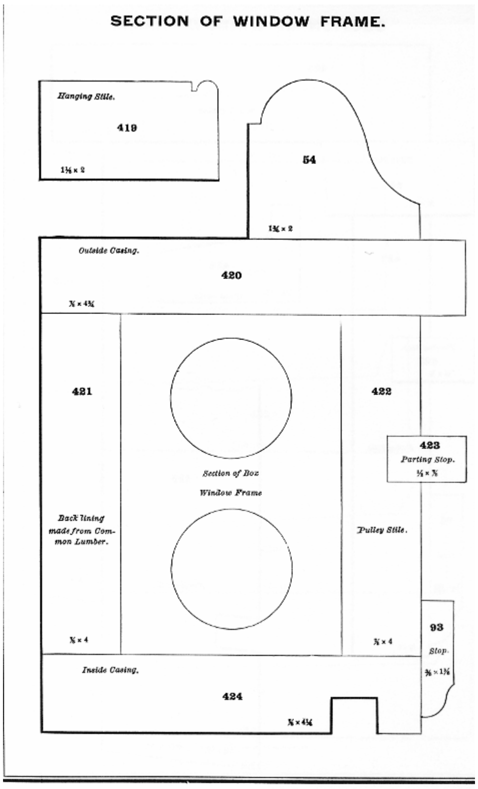
This paper will spotlight a completely new method of installing fenestration, the principle of which goes back more than one hundred years, but encapsulates and enhances current methods of integration with the wall envelope. This method allows a permanent frame to the building, while enabling simple and non-invasive replacement of the fenestration product at the end of its useful life. The integration of the frame into the wall envelope will be demonstrated, followed by the installation of self-contained sash fixtures, literally plugged into the frame, and easily removable.

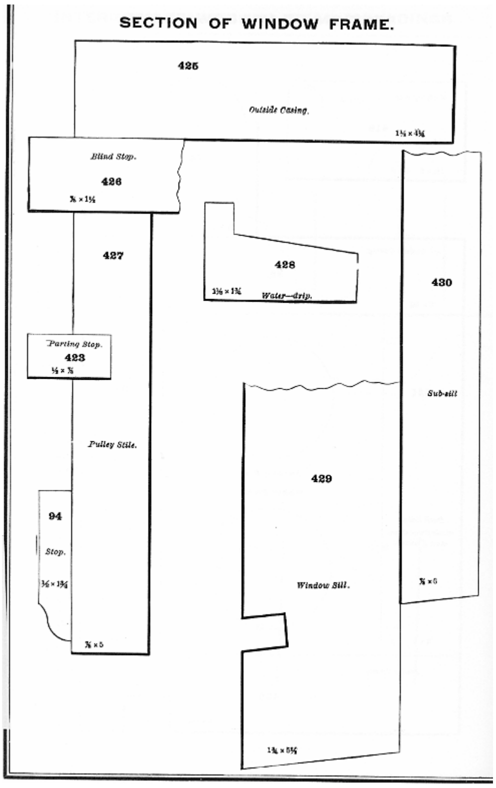
3 EVOLUTION OF FENESTRATION INSTALLATION METHODS AND STANDARDS

At the turn of the last century, books on architectural details were plentiful, accurate, and provided clear assembly details for millwork members. All of the members of a fenestration frame were clearly identifiable, readily avialable at all mills, and easily understood by the carpenter. See figures 2 and 3. Through post-World War II, installation techniques were taught father-to-son, journeyman-to-apprentice, with the wide use of the guild method.

Post-World War II brought about an enormous building boom and the introduction of new materials, which had not been traditionally used in buildings, or new materials that had been previously unavailable. Some examples of new building materials are aluminum, polymer materials, and a complete subset of plastics and chemistries that make up today's building materials. Even the fenestration itself goes from wood to metals to plastics to composites. Newly manufactured fenestration that is comparable to that of one hundred years ago do not exist today, with the exception of custom historic replicas.

Figures 2 and 3. Typical shapes readily available from mills, circa 1900.





Skilled labor and craftsmanship has slowly deteriorated and become largely unavailable, as the post-World War II building boom erupted, the demand for skilled craftsmen was outstripped by the building demand. By 1970, guilds and unions lost membership and a further decline in qualified craftsmen resulted. To overcome this dilemma, we start to see the development of some manufacturers' instructions, most of which are developed on the commercial side of the building industry. Other than the few manufacturers that offered installation instructions prior to 1990, there was no consensus standard for installation of fenestration, nor were there training courses or vocational education in this area. In 1992 the California Association of Window Manufacturers started developing the first consensus-built installation instructions for fenestration installation. That document was CAWM 400-95, Standard Practice for the Installation of Windows with Integral Mounting Flange in Wooden Frame Construction.

In 1995, and using the CAWM 400-95 as a template, ASTM started work on a consensus standard that is now known as ASTM E 2112-01, Standard Practice for Installation of Exterior Windows, Doors and Skylights. In 2002, ASTM starts work on ASTM E 2112, Revision 1, which is currently being balloted. In 2002, AAMA published AAMA 2400, which is loosely based on CAWM 400. Lastly, the Canadian Standards Association (CSA) developed and continues to develop CSA A 440.4, Window and Door Installation. Table 1 describes the advantages and disadvantages of each of these systems.

Also included in Table 1 is the new methodolgy proposed in this document, named the Modular Insert Fenestration System, or MIFS. This system captures many of the design & water management advantages of the pre-industrial revolution design, but in a modernized form that takes advantage of mass production efficiencies. The MIFS concept is described in more detail below.

Table 1 – Overviw of Fenestration Installation Methods & Standards, with key features, advantages and disadvanatages.

Standard Methods	*Description*	*Advantages*	*Disadvantages*
CAWM 400	Addressed one type of fenestration with integral fin, in one type of wall, wood framing, and 4 possible interfaces.	Was simple and emphasized integration of interface with fenestration.	Limited to one type of window. Barrier mehtod: Presupposed there was no leakage from corners of fenestration.
AAMA 2400	Same as CAWM 400, except it generalized the interface methods into two types.	Was simple and emphasized integration of interface with fenestration	Limited to one type of window. Barrier method: Presupposed there was no leakage from corners of fenestration.
ASTM E 2112-01	Windows, Doors, and Skylights, mostly Residential construction. Includes a variety of window frame types and walls.	User can integrate with both barrier and drainage type walls. Gives precise information on sealants, anchoring, and related aspects of installation, recognizes incompatibilities of dissimar materials.	Barrier System: Does not recognize leakage at window corner or around / through wall interface, integration to the wall only with finned windows, only integration of fenestration with wall system.
ASTM E 2112-Revision 1	Same as above	Includes Drainage Method: Assumes that incidental water enters the wall cavity at window joinery or interface, adds pan flashing details with variety of material combinations.	High level of skill is required, costs more than other methods. Window leakage is drained to the Water-Resistive Barrier inside the wall cavity.
CSA A440.4	Window and door installation, based on rainscreen method of wall design	Gives techniques for mulled windows, requires pan flashings of sorts.	Can allow leakage beyond the window sill and is over-reliant on sealant.
MIFS System	Requires a receptor for all fenestration including windows, doors, and other through-wall penetrations.	Simple standardized method, flexible construction sequen-cing, delivers drained water and window corner leakage to the exterior cladding, allows for non-destructive removal of fenestration.	Limited flexibility of size and shape with initial offerings.

4 THE MODULAR INSERT FENESTRATION SYSTEM (MIFS)

The Modular Insert Fenestration System consists of two parts: 1) a ‘receptacle’, which is a rigid or semi-rigid frame that is sealed to a rough opening of a building structure, such that it becomes a permanent, moisture-proof part of the building envelope, and 2) a ‘fenestration product’, such as a window, door, or any other object to be inserted into a building structure, which is designed to connect to the MIFS receptacle in a manner that is structurally sound, yet provides for easy installation and removal of the fenestration product.

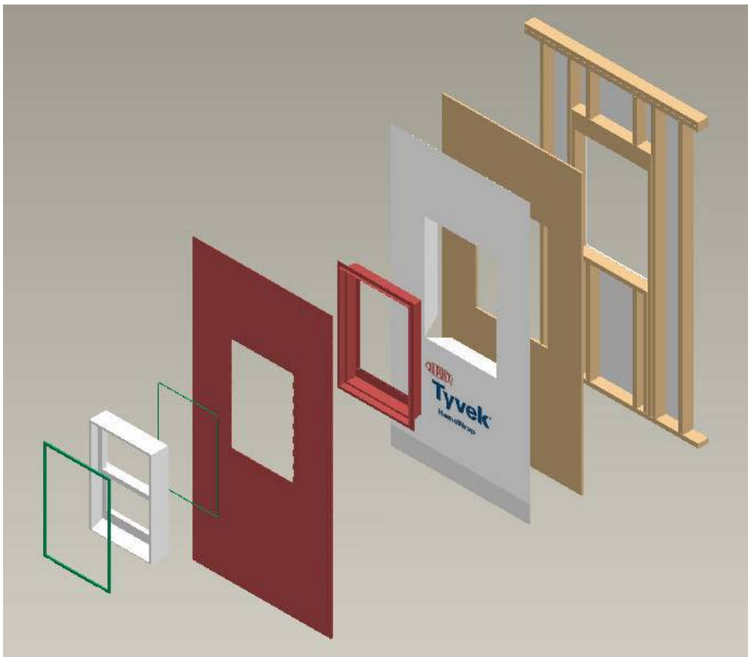
Thus, the fundamental MIFS concept is for the receptacle to form a continuous enclosure around the fenestration product, completely isolating it from the building envelope. The gap between the receptacle and the fenestration product will form a pressure equalized cavity, sealed at the interior joint between the fenestration product and the interior frame of the receptacle, providing a drainage path directed out the exterior sill of the receptacle for any moisture intrusion that occurs at or around the fenestration product. A schematic of the MIFS system with the insert receptacle is shown on Fig. 4, where the insert receptacle (shown in red) is installed and made a permanent part of the building opening. The fenestration product can then be installed at any time during the construction sequence, either before (as shown on Fig 4) or after the siding.

Key advanteges that are anticipated for the MIFS concept are highlighted as follows:

- **Provides a Robust Moisture Seal** – the seal is at the interior of the receptacle, which is protected from environmental exposure. The resulting fully encapsulated, pressure equalized chamber will direct any moisture intrusion on or around the fenestration product to the exterior. In addition, we expect that thermal, acoustical, and hygrothermal transfer performance is enhanced because of the isolation of the fenestration from the wall.
- **Greatly Improved Installation Method** – fenestration products can be inserted into the receptacle through screws or clamping mechanisms in a greatly simplified, more standardized method than exists today.
- **Flexible Installation Sequencing** – the receptacle will be installed, plumb, level and square in the wall opening first. The fenestration product can then be installed at any time during the construction process. Construction critical paths are enhanced, allowing perfect on-time sequencing. This will protect the fenestration product, which is very expensive and fragile, from damage during harsh conditions during the construction phase, such as exposure to siding, bricks, electrical cords, tools, etc. . It is possible to plug in the fenestration sash module after the building has been completely finished, painted, cleaned; thus eliminating damage to the fenestration product by those processes.
- **Material Durability** – the MIFS receptacle will be produced from a material that will be durable for many years as a permanent part of the building envelope. It will provide structural integrity through environmental exposure and be of sufficient toughness to withstand hammer impact and other harsh exposure (falling bricks) during the construction phase
- **Numerous Cost Advantages** – The simplified, standardized installation practice will enable better results with a lower skilled workforce, which is typically what is available. Installation time will decrease, as well as loss due to damage to the fenestration product; and the cost of incorrect installation, both during the construction phase and in the life cycle of the building, will be reduced. This is a critical and key point, as the skill levels of the craftsmen of yesteryear have not transferred to the present, and we can no longer rely on skilled personnel for the future.
- **Enhanced Ability to Replace Fenestration Products** – Fenestration products will be easily removed without the need to disrupt the siding building, as is the case with flanged windows today. Thus, a homeowner can take advantage of new advances in glazing technology or 'seasonal' glazing choices.
- **Full Integration into the Building Envelope** – the receptacle will contain robust flanges that will be fully integrated and sealed (through self-adhered flashing or adhesive means) onto the building envelope. Because this material will be more durable than current flanges and the receptacle will be installed and squared into the building rough opening before the fenestration product (which is heavy and difficult to install correctly), the installation of the receptacle will be more robust.

This system is currently in the prototype phase and end use testing to prove out and demonstrate the advantages listed above will be done through independent test labs in the next several months. A concern may lie in the ability for the MIFS frames to conform to all the sizes and shapes available to fenestration products. Thus, the initial offerings will likely be more suited for large volume builder or pre-fabricated construction, which utilize a more limited range of fenestration configurations. As this system becomes more widely accepted, a broader offering of size and shape will become more feasible.

Figure 4: Schematic of MIFS insert receptacle system. The receptacle is permanently installed into the rough opening and window can be inserted at any time during the construction sequence (i.e., either before or after the siding is installed).



5 SUMMARY AND CONCLUSIONS

Over the past century, the design and installation of fenestration products have evolved in such a way that production efficiencies have greatly increased, which of course is by necessity due to the massive growth in building construction. However, these same production efficiencies have resulted in a loss in the fundamental design principles practiced in pre-industrial revolution techniques, in that the mass-produced fenestrations of today are poorly designed to manage the inevitable moisture intrusion in and around the fenestration. As a result, building failures at the window-wall interface are common. A new methodology, named the Modular Insert Fenestration System, that combines the design principles of the past with the mass-production efficiencies of today, has been developed. This methodolgy has the potential to usher in a new, more robust, era for fenestration installation and performance in the 21^{st} century.

6 REFERENCES

The Muliner Catalog of 1893; The Muliner Box & Planing Co.; Re-published as Turn-of-the-Century Doors, Windows and Decorative Millwork, Dover Publications, Inc., Mineola, NY, 1995

Radford's Portfolio of Details of Building Construction, 1911 - Re-published as Old House Measured and Scaled Detailed Drawings for Builders and Carpenters, William A. Radford, Dover Publications, Inc., Mineola, NY 1983
CAWM 400, 1995 – Standard Practice for the Installation of Windows with Integral Mounting Flange in Wooden Frame Construction - California Association of Window Manufacturers
AAMA 2400-02 Standard Practice for Installation of Windows with a Mounting Flange in Stud Frame Construction – American Architectural Manufacturers Association, Schaumburg, IL
ASTM, 2001, E 2112 Standard Practice for Installation of Exterior Windows, Doors, and Skylights, ASTM International, W. Conshohocken, PA and Revision 1 (unpublished at this writing)
Canadian Standards Association, CSA A440.4 Window and Door Installation - CSA, Mississauga, Ontario, Canada

Generic limit state design of structures

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ABSTRACT

Generic limit state design is aiming at fullfilling the generic requirements and criteria of the life time quality. Lifetime quality means the capability of a facility or structure to fullfill the requirements of owners, users and society over the design life. On generic level the requirements include social (human), economic, ecological and cultural requirements, which in design are transformed into technical and economic level. The extended limit states including the following three classes:

1. Mechanical limit states under static and dynamic loads
2. Durability limit states under physical, chemical and biological loads
3. Usability limit states under obsolescence loads (changes of use or requirements)

Traditionally the design is focused on mechanical design, because of the structural safety reasons and high consequences of these types of failure to human beings. However, degradation and obsolescence together are most important loads causing needs for refurbishment or demolition, and therefore have big economic consequences. Extensive analysis of reasons of heavy refurbishments or demolitions of buildings and bridges have shown, that the obsolescence under changing use, demands of users and general requirements of the society are the most dominant reasons and still more common than the degradation. Therefore it is important to develop rational and systematic methodologies and methods for controlling all these three types of limit states in lifetime planning, design and management process of buildings and civil infrastructures. This report is focused on durability limit state design and obsolescence limit state design.

There is an analogy between the terms and parameters of mechanical design against static and dynamic loads, and the durability design against degradation, but most of the variables and methods are different. The durability design can be carried out with similar statistical or deterministic safety factor methods as mechanical design.

Usability design against obsolescence can be based on statistical risk analysis and other methods of systems engineering, like Risk Analysis, Quality Function Deployment (QFD) or Multi Attribute Decision Analysis (MADA). The obsolescence analysis and optimisation is always made with comparison of planning, design or product alternatives, comparing their response towards changing requirements and supposed changes of user demands.

KEYWORDS

design, durability, limit state design, safety factors, risk analysis

1 INTRODUCTION

The current objectives towards sustainable society and construction sector can be interpreted into generic requirements for construction as presented in table 1.

Table 1. Generic classified requirements of structures and buildings [1,3].

1. Human requirements
• functionality in use
• safety
• health
• comfort
	2. Economic requirements
	• investment economy
	• construction economy
	• lifetime economy in:
	  ○ operation
	  ○ maintenance
	  ○ repair
	  ○ rehabilitation
	  ○ renewal
	  ○ demolition
	  ○ recovery and reuse
	  ○ recycling of materials
	  ○ disposal

3. Cultural requirements • building traditions • life style • business culture • aesthetics • architectural styles and trends • imago	***4. Ecological requirements*** • raw materials economy • energy economy • environmental burdens economy • waste economy • biodiversity

When looking at the statistics of demolitions of buildings and structures we can notice the following reasons for refurbishment or demolition:

- Degradation is the the main reason for refurbishment of buildings in 17 % (Aikivuori, 1994)[15] and in 26 % (steel) to 27 % (concrete) of demolition of bridges (Iizuka, 1988) [16]. In individual cases degradation can be a dictating reason for refurbishment or demolition of the structures, which are working in highly degrading environment.
- Obsolescence is the cause of refurbishment of buildings in 26 % (Aikivuori, 1994) [15] and the reason of demolition of bridges in 74% of demolition cases (Iizuka, 1988)[16].
- In the case of modules or component level renewals of facilities the share of obsolescence is still higher.
- This means that the obsolescence is the dominating reason for refurbishments and demolitions of facilities and their structures.

A conclusion of this, and a challenge for structural engineering is, that we have to include the degradation and obsolescence criteria into the design, as well as into the MR&R (Maintenance, Repair and Rehabilitation) planning of structures. In this use we need new methodology, models and methods for analysis, optimisation and decision making.

2 DESIGN LIFE

Integrated life time design is aiming at fullfilling the generic and specific requirements of lifetime quality of a facility or structure. The **lifetime quality** means the capability of a facility to fullfill the requirements of owners, users and society over the design life. The generic classification of these requirementas are presented in table1.
Design life is a specified time period, which is used in calculations. The classification of design life is presented in table 2.1 of the European standard EN1990: 2002 [2].

3 EXTENDED LIMIT STATES OF LIFETIME DESIGN

The classes of extended limit states for integrated lifetime design are as follows:

- Mechanical (static and dynamic serviceability and safety)
- Durability (degradations)
- Usability (obsolescence)

The serviceability limit states and ultimate limit states of concrete structures in relation to this classification are presented in table 1 [1]. Models of generic specifications of these limit states are presented in tables 2 and 3. Analogical features of terms and variables between mechnical, durability and usability designs are presented in table 4.

Table2. Summary of performance and functionality limit states (Sarja, 2003) [1].

A. Performance limit states	
Serviceability limit states	Ultimate limit states
1. Surface cracking	1. Failure under static, dynamic or fatigue loading
2. Surface scaling	
3. Deflection	
4. Carbonatisation or chemicals (e. g. chlorides) penetration until reinforcement	
5. Corrosion of reinforcement	
B. Functionality limit states	
1. Weakened functionality	1. Total loss of functionality
2. Weakened economy of operation	2. Total loss of economy of operation
3. Weakened economy of MR&R	3. Total loss of economy of MR&R
4. Minor health problems in use	4. Severe health problems in use
5. Aesthetic change of surface (abrasion, colour changes)	5. Total loss of aesthetic acceptability
6. Cultural ineligibility	6.Total loss of cultural acceptance
7. Weakened ecology	7. Severe ecological problems or hazard

Table 3. Generic mechanical, durability and usability limit states of concrete structures [1].

Classes of the limit states	***Limit states***		
	Mechanical limit states under static and dynamic loads	Durability limit states under physical, chemical and biological degrading loads	Usability limit states under loads causing obsolescence through changes of use and requirements
1. Serviceability limit states	1. Deflection limit state 2. Cracking limit state	3. Surface faults causing aesthetic harm (colour faults, pollution, splitting, minor spalling) 4. Surface faults causing reduced service life (cracking, major spalling, major splitting) 5. Carbonation or chemicals penetration into the concrete cover (grade 1: one third of the cover , grade 2: half of the cover, grade3: entire cover)	6. Reduced usability and functionality, but still usable • The safety level does not allow the requested increased loads • Reduced healthy, but still usable • Reduced comfort, but still usable
2. Ultimate	1. Insufficient	2. Insufficient safety due to	3. Serious obsolescence causing

limit states	safety against failure under loading	indirect effects of degradation: • heavy spalling • heavy cracking causing insufficient anchorage of reinforcement • corrosion of the reinforcement causing insufficient safety.	total loss of usability through: • loss of functionality in use (use of building, traffic transmittance of a road or bridge etc.) • safety of use • health • comfort • economy in use • MR&R costs • ecology • cultural acceptance

Table 4. Comparison of the terms and variables of limit state method*s* in mechanical, durability and usability design and control.

Mechanical limit state design	Durability limit state design	Obsolescence limit state design
1. Strength class 2. Target strength 3. Characteristic strength (5 % fractile) 4. Design strength 5. Partial safety factors of materials strength 6. Static or dynamic loading onto structure 7. Partial safety factors of static loads 8. Service limit state (SLS) and ultimate limit state (ULS)	1. Service life class 2. Target service life 3. Characteristic service life (5% fractile) 4. Design life 5. Partial safety factors of service life 6. Environmental degradating loads onto structure 7. Partial safety factors of environmental loads 8. Serviceability and ultimate limit states, related to the basic requirements: Human requirements, lifetime economy, cultural aspects and lifetime ecology	1. Service life class 2. Target service life 3. Characteristic service life (5%fractile) 4. Design life 5. (Partial safety factors of service life) 6. Obsolescence loading onto structure 7. Partial safety factors of obsolescence loading 8. Serviceability and ultimate limit states related to obsolescence in relation to the basic requirements: Human requirements, lifetime economy, cultural aspects and lifetime ecology

4. EXTENDED RELIABILITY OF STRUCTURES

The statistical reliability requirements are defined in the European standard EN 1990: 2002 [2]. The reliability requirements are expressed also in terms of statistical reliability index β.

$$P_f = \Phi(-\beta) \tag{1}$$

where Φ is the cumulative distribution function of the standardised Normal distribution. The requirementsfor the reliability index are shown in Table B2 of the standard EN 1990 for the design of new structures, as well as for the safety of existing structures.

5. DURABILITY LIMIT STATE DESIGN WITH SAFETY FACTOR METHOD

In practice it is reasonable to apply the lifetime safety factor method in the design procedure for durability, which was first time presented in the report of RILEM TC 130 CSL [8,11]. The lifetime safety factor method is analogous with the static limit state design. The durability design with lifetime safety factor method is always combined with static or dynamic design and aims to control the serviceability and service life of a new or existing structure, while static and dynamic design controls the loading capacity. The design service life is determined by formula (*Sarja and Vesikari 1996 [8,11], modified:Sarja 2001 [12] and Sarja 2002[3,4]*):

$$t_{Ld} = t_{Lk} / \gamma_{tk} >= t_g \tag{2}$$

where t_{Ld} is the design service life,
t_{Lk} the characteristic service life
γ_{tk} the lifetime safety factor, and
t_g the target service life.

The lifetime safety factor depends on the maximum allowable failure probability. The lifetime safety factor also depends on the form of service life distribution. Figure 1. illustrates the meaning of lifetime safety factor when the design is done according to the performance principle. The function *R*(*t*) – *S* is called the safety margin.

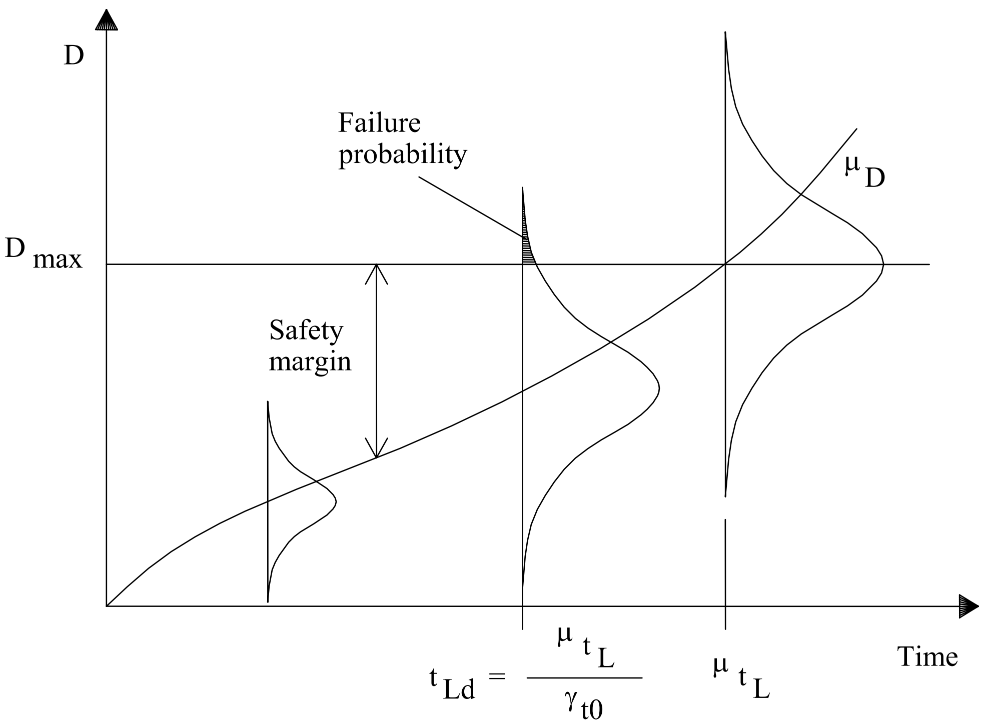

Figure 1. The meaning of lifetime safety factor in a degradation process.

An example of calculated lifetime safety factors using sepecific values of standard deviations of parameters is presented in table 5.

Table 5. Central lifetime safety factors γ_0 and characteristic lifetime safety factors γk in the cases, when $V_D = 0{,}3$ and $V_D = 0{,}4$. The reliability index values of EN1990: 2002(Table B2) are applied.

Reliability Class/ Consequence Class	Safety index β		Lifetime safety factor							
			1 year reference period				50 years reference period			
			Central safety factor γ_0		Characteristic safety factor γk		Central safety factor γ_0		Characteristic safety factor γk	
Ultimate limit states										
	1 year reference period	50 years reference period	$V_D = 0{,}3$	$V_D = 0{,}4$	$V_D = 0{,}3$	$V_D = 0{,}4$	$V_D = 0{,}3$	$V_D = 0{,}4$	$V_D = 0{,}3$	$V_D = 0{,}4$
RC3/CC3: High consequence for loss of human life, *or* economic, social or environmental consequences very great	5,2	4,3	2,56	3,08	2,07	2,42	2,29	2,72	1,80	2,06
RC2/CC2: Medium consequence for loss of human life, *or* economic, social or environmental consequences considerable	4,7	3,8	2,41	2,88	1,92	2,22	2,14	2,52	1,65	1,86
RC1/CC1: Low consequence for loss of human life, *or* economic, social or environmental consequences small or negligible	4,2	3,3	2,26	2,68	1,77	2,02	1,99	2,32	1,50	1,66
Serviceability limit states										
RC3/CC3	No general recommendations. Will be evaluated in each case separately									
RC2/CC2	2,9	1,5	1,87	2,16	1,38	1,50	1,45	1,60	1	1
RC1/CC1	1,5	1,5	1,45	1,60	1	1	1,45	1,60	1	1

The degradation limit state design procedure is as follows (Sarja&Vesikari 1996 [8], Sarja 2002 [3]:

1. mechanical (static, dynamic, fatique) limit state design with traditional limit state design, applying a relevant norm or standard (e. g. EN 1990: 2002: Basis of structural design)
2. specifying the target service life and design service life
3. analysing environmental loads onto structures
4. identifying durability factors and degradation mechanisms
5. selecting a degradation model for each degradation mechanism
6. calculating durability parameters using available calculation models
7. possible updating the calculations of the ordinary mechanical design
8. transferring the durability parameters into the final structural design documentation

6 RELIABILITY UNDER OBSOLECENCE

6.1 Principles

Obsolescence means the inability of a facility or a part of it to satisfy changing functional (human), economic, cultural or ecological requirements. Obsolescence can affect to the entire building or civil infrastructural facility, or just some of its modules or components (Sarja 2002 [3], Sarja 1998 [7]).
The obsolescence behaviour of a facility is very different in comparison to mechanical and durability behaviour. The principal difference is, that the obsolescence limit states are related to changes in requirements and user demands, while mechanical and durability limit states are related to original requirements, while the changes leading to limit states happen in the properties of the facility or structure itself. The obsolescence control is aiming to guarantee the ability of the buildings and civil infrastructures to maintain their ability to meet all current and changing requirements with minor changes of the facilities.
Because the obsolescence process of a facility depends on the development of local conditions, as well as on the general development of society during the service life (or residual service life) of a facility, there is lot of uncertainty involved in obsolescence analyses and control. Like in any uncertainty-filled problem, also in obsolescence situation the case must be structured down to smaller parts, which can be consistently handled. It must be noted that the systematic obsolescence avoidance and control thought should be present in all life cycles of the facility planning and programming: design; construction; operations, maintenance, refurbishment, renewal and reuse. The obsolescence analysis should be performed before the onset of obsolescence, as a part of the facility owning and management strategy (Sarja et al 2004 [14]).
The obsolencence control is included in investment strategies and planning, lifetime design and lifetime management strategies and planning, which all aim at minimising the need of early and unexpected refurbishment, renewal or demolition. Examples of the obsolescence are as follows:

- Functional obsolescence is due to changes of users or changes of functions or other demands of the user of the facility. Examples of these are currently frequent change of users, increased demands of healthy, convenience and accessability etc.
- Technological obsolescence can be a cause when new products providing better performance and easier maintenance in operation become available with new materials, new technology, new equipments etc..
- Economic obsolescence means that operation and maintenance costs are too high in comparison to new facility, systems or products.
- Cultural obsolescence is related to the changes of demands and criteria of culture, living and working, aesthetic and architectural styles and trends, and imago of the owners, users and society. An example of this are the buildings of mass production time in Europe, especially in eastern Europe, which are not more accepted because of a pure architectural and aesthetic imago.
- Ecological obsolescence means the inablity of a facility to fullfill the incraeasing ecological and environmental requirements of the society, regarding to energy consumption, pollution, raw materials consumption, waste production or loss of biodiversity or geodiversity.

6.2 Methods for obsolescence limit states control

The usability analysis and optimisation against obsolescence is always made with comparison of alternatives, comparing their response towards changing requirements and supposed new demands. For each alternative of strategic or operative process in investment briefing, design of a facility or MR&R strategy or plan, the following obsolescence procedure will be made:

1. identifying the relevant obsolescence factors (changes)
2. analysing relevant obsolescence limit states
3. selecting evaluation methods for the relevant potential obsolescence cases
4. evaluating the characteristic service life against the actual modes of the obsolescence
5. evaluating the required lifetime safety factors for each mode of obsolescence
6. listing the modes of the obsolescence, and the corresponding values of the design service life

7. moving the results into the general planning, design or MR&R planning procedure

The following methods can be applied in obsolescence analysis, control and optimisation:

- Quality Function Deployment method (QFD) (Sarja 2004, Lifecon Deliverables D2.3 and D5.1) [14]
- Multiple Attribute Decision Aid (Sarja et al. 2004, Lifecon Deliverable D2.3) [14]
- Risk Analysis (RA) (Rissanen 2004, Lifecon Deliverable D2.3) [14])
- Life Cycle Costing method (LCC) [Miller et al 2004: Lifecon Deliverable D5.3) [14]
- Simulations

Detailed descriptions of the applications of these methods are presented in (Lifecon, 2004) [14].

7. REFERENCES

[1] Sarja, Asko 2003, Lifetime performance modelling of structures with limit state principles. in: *Proceedings of 2nd International SymposiumILCDES2003, Lifetime Engineering of Buildings and Civil Infrastructures*, Kuopio, Finland, December1-3, 2003. Association of Finnish Civil Engineers pp. 59-66.

[2] EN 1990: 2002: Eurocode - Basis of structural design. CEN: European Committee for

Standardisation. Ref. No. EN 1990:2002 E. 87 pp.

[3] Sarja, Asko 2002, *Integrated Life Cycle Design of Structures.* 142 pp. Spon Press, London. ISBN 0-415-25235-0.

[4] Sarja, Asko 2002, Reliability based life cycle design and maintenance planning. Workshop on Reliability Based Code Calibration, Swiss Federal Institute of Technology, *ETH* Zurich, Switzerland , March 21-22,. http://www.jcss.ethz.c

[5] ISO/DIS 15686-1 1998, Buildings-Service life planning-Part 1 General Principles. Draft.

[6] Sarja, Asko 2004, Generalised lifetime limit state design of structures. *Proceedings of the 2 nd International Conference, Lifetime-Oriented Design Concepts, ICDLOC*, pp. 51-60. Ruhr-Universität Bochum, Germany, 2004. ISBN 3-00-013257-0

[7] Sarja, Asko, Sarja, Asko (ed) 1998, Open and industrialised building. London. E & FN Spon. 228 pp.

[8] Sarja, Asko & Vesikari, Erkki (Editors) 1996, *Durability design of concrete structures. RILEM Report of TC 130-CSL. RILEM Report Series 14. E&FN Spon, Chapman & Hall,*. 165 pp.

[9] *EN 206-1 Concrete-Part , 2000: Specification, performance, production and conformity. CEN European Committee for Standardisation. REf. No EN 206-1:2000 E. 72 pp.*

[10] *JCSS Model Code. Joint Committee of Structural Safety.* http://www.jcss.ethz.ch ,

[11] Sarja, Asko, 1999, Environmental Design Methods in Materials and Structural Engineering. RILEM Journal: Materials and structures, Vol. 32, , pp 699-707

[12] Iselin, D.G., Lemer, A.C., (Eds) 1993., The Fourth Dimension In Building: Strategies For Minimizing Obsolescence. National Research Council, Building Research Board. National Academy Press, Washington, D.C. 59 pp. + Appendix 42 pp.

[13] Sarja, Asko 2000, Development towards practical instructions of life cycle design in Finland. *RILEM Proceedings PRO 14, Proceedings of the RILEM/CIB/ISO International Symposium: Integrated Life-Cycle Design of Materials and Structures, ILCDES.* pp. 1-5

[14] Sarja, Asko (Co-ordinator) et al. 2004. Life Cycle Managemnt of Concrete Infrastuctures, Lifecon, Deliverables D1-D15. http://www.vtt.fi/rte/strat/projects/lifecon/

[15] Aikivuori, Anne 1994, *Classsification of demand for refurbishment projects.Acta Universitatis Ouluensis, Series C, Technica 77.*University of Oulu, Department of Civil Engineering. ISBN 951-42-3737-4.71 pp.+7 Appendices.

[16] Iizuka, Hiroshi 1988, A statistical study of life time of bridges. Structural Eng./Earthquake Eng. Vol. 5 No1, pp. 51-60, April 1988. Japan Society of Civil Engineers.

Durability Increase of Special Concrete by Application of Waste Raw Materials

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ABSTRACT

A great deal of research is currently focused on the durability of building structures and concrete degradation caused by the effect of corrosive liquids or the gaseous phase. Test results have shown that concrete resistance to aggressive media can be increased by the addition of suitable types of waste materials or admixtures having pozzuolana properties.
The paper discusses the potential for increases in durability of special concrete types, including Self Compacting Concrete, High Performance Concrete, Reactive Powder Concrete by addition of suitable waste materials (power plant fly ash) or admixtures (meta-kaolin). The aim of the work is to verify, compare and analyze the effect of addition of power plant fly ash and meta-kaolin respectively to concrete subjected to immersion in aggressive media. Concrete were prepared with and without the utilization of waste raw materials or admixtures. The concrete was stored in different aggressive liquid media (SO_4^{2-}, Cl′, oil) or gases (simulated concentrations of CO_2, SO_2 with different relative humidities, and ambient exterior atmospheric conditions) and reference samples were stored in water. The storage periods were 3 and 6 months. After storage, specimens were tested by physico-mechanical and physico-chemical methods, and their appearance and state after exposure to individual aggressive effects was recorded. Results from short-term test of 6 months exposure in aggressive media show that enhanced durability can be achieved with concretes incorporating power plant fly ash or meta-kaolin. No significant changes of physico-mechanical and physico-chemical properties were found in any samples of concrete modified by power plant fly ash or meta-kaolin after 6 months of storage in different aggressive media. Definite conclusions regarding the enhanced performance of these modified concretes cannot be made given that an exposure period of 6 months is too short for generalization of results. However, these initial results can help orient further research and further work continues with additional tests results from exposure periods of 48 months to be reported at a later date.

KEYWORDS

Waste raw materials, Reactive Powder Concrete, Self Compacting Concrete, High Performance Concrete.

1 INTRODUCTION

Over the past decades it is apparent that High Strength Concrete (HSC) is increasingly being used and on a continually larger scale given the improved levels of performance. Of basic interest is the characteristic high compressive strength of this concrete that may range from at least 65 $N.mm^{-2}$ to upwards of 100 $N.mm^{-2}$. Research in this area is on-going and research results indicate that significantly higher compressive strengths can bee achieved of up to, for example, 200 $N.mm^{-2}$ and more. Such type of concretes have by some been categorised as “Ultra High Strength Concrete” (UHSC) and one representative type of UHSC developed in France is RPC (Reactive Powder Concrete). In essence, UHSC is characterised by having extremely high physico-mechanical parameters. Increasingly the current trend is to utilize functional industrial wastes, such as power plant fly ash, in the formulation of concrete mixes. However, the long-term performance of concretes prepared with the addition of different waste raw materials is not readily understood, nor for example, what level of enhanced resistance to deterioration such concretes may offer when exposed to different environmental conditions, in particular, aggressive media and corrosive environments in which concretes are often exposed.

2 METHODS AND TECHNIQUES

A study was conducted to verify possible increases in durability (or resistance to deterioration) of new types of concrete such as Reactive Powder Concretes (RPC), Self Compacting Concretes (SCC) and High Performance Concretes (HPC) by the addition of waste materials (e.g. power plant fly ash), or admixtures having pozzuolana properties (e.g. micro-silica). The primary focus of this study was determining the resistance to deterioration when such types of concrete are subjected to an aggressive chemical environment.
The study was undertaken in two phases, the first of which was determining the base composition of the different concrete types and the amounts of either fly ash or admixture to be incorporated in the mix in relation to standard concrete, in which no admixtures nor fly ash were added. Various trials and test batches helped establish the final composition of the concrete mixes and were used to adjust mix proportions to achieve the desired functional properties of the different blended concretes.
The second phase of work was divided into two parts, the first of which focused on the use of fly ash as an additive to enhance the durability of either RPC or SCC; the second part consisted of incorporating meta kaolin to High Performance Concrete (HPC) as a means to improve durability in this type of concrete. In both parts of the study, concrete, incorporating additives in the concrete mix, were compared to a standard concrete mix in terms of their physico-mechanical properties when subjected to immersion in aggressive media for a period of 3 to 6 months. In this manner, the effect of the additives on the physico-mechanical and physico-chemical properties of the different concrete types could be determined depending on the aggressive medium in which the test pieces were stored.
The determined values characterizing the properties and the state of the Comparison in terms of the physico-mechanical and physico-chemical properties of the different concrete types are made between test specimens stored in different aggressive media to those stored in water under standard conditions (state conditions).
Likewise, comparisons could be made between the properties of concrete incorporating selected quantities of waste materials or admixtures and a standard concrete mix that does not include waste materials or admixtures. This permits determining the potential for enhanced long-term performance (durability) of these novel concrete mixes.
Test specimens for High Performance Concrete (HPC) and Self Compacting Concrete (SCC) were cubes (100-mm) cast in steel moulds. Specimens were demoulded after 24 hours and were cured for 28 days in water following Standard conditions (T = 22°C and RH = 100%). Specimens prepared for the Reactive Powder Concrete (RPC) were cast in the form of prismatic beams having dimensions of 40 by 40 by 160 mm. The test specimens were demoulded 5 to 6 hours after being cast and were cured for a period of 72 ± 2 hours in controlled conditions of 100 % relative humidity and a temperature of 90±3°C. After this period, the RPC test specimens were placed in water as other test pieces.

The composition of different types of concrete, including RPC, HPC and SCC, are found in Tables 1, 2 and 3 respectively.

Concrete components	*RPC I*	*RPC II*
	[kg.m^{-3}]	*[kg.m^{-3}]*
Cement-SVC III/A 32,5 R	712	712
Quartz sand - PR33	1200	1140
Fly ash	-	60
Micro-silica	228	228
Water	178	178
Plasticizing admixture – Chrysofluid Premia 150	21	21

Table 1: Composition of Reactive Powder Concrete (RPC)

Concrete components	*SCC I*	*SCC II*
	[kg.m^{-3}]	*[kg.m^{-3}]*
Cement - CEM I 42,5 R	450	360
Aggregates – 0/4 mm	930	930
4/8 mm	200	200
8/16 mm	510	510
Fly ash	-	90
Plasticizing admixture –Chrysufluid Optima 200	7,2	5,76
Water	203	213

Table 2: Composition of Self Compacting Concrete (SCC)

Concrete components	*HPC I*	*HPC II*
	[kg.m^{-3}]	*[kg.m^{-3}]*
Cement - CEM I 42,5 R	387	430
Aggregates – 0/4 mm	817	817
4/8 mm	209	209
8/16 mm	874	874
Meta-kaolin	-	43
Plasticizing admixture -Woerment FM 794	7,4	7,4
Water	160	160

Table 3: Composition of High Performance Concrete (HPC)

3 PROGRESS OF EXPERIMENTAL WORK

3.1 Possibility of increase in durability by addition of industrial waste (power plant fly ash) to Reactive Powder Concrete (RPC) and Self Compacting Concrete (SCC)

The aim of this part of the paper has been to verify the durability of Reactive Powder Concrete (RPC) and of Self Compacting Concrete (SCC) by application of power plant fly ashes. This concrete was produced both with the addition of definite quantity of admixture and without this admixture. The test pieces produced from this concrete were exposed to selective aggressive media and the reference samples were stored in water medium following Standard demands. The test pieces were stored in testing media for the period of 3 to 6 month and after this period the test pieces were checked by physico-mechanical tests and by physico-chemical analysis i.e. there were determined the physico-mechanical and physico-chemical parameters of samples, their appearance and state after the effect of individual aggressive medium for the given period. The obtained values were compared on the one hand following the individual aggressive media and on the other hand of course with values obtained with reference test samples that were stored in Standard atmosphere. More important are naturally the properties and state comparison of the same concrete type produced with and without use of waste raw materials. The possibility of durability increase by application of waste raw materials can be evaluated just by comparison of identical concrete types produced without the use of admixture (meta-kaolin) and with the use of this admixture.

3.2 Possibility of increase in Durability by addition of puzzolona-based admixtures (meta-kaolin) to High Performance Concrete (HPC)

The aim of this part of the paper has been to verify the possibility of increases in durability of High Performance Concrete (HPC) by the addition of meta-kaolin to the concrete mix. As is provided in Table 3, concrete mixes were produced with (HPC II) and without (HPC I) the addition of admixture. The test specimens prepared from both these concrete mixes were exposed to chemically aggressive media and the reference samples were stored in water following standard requirements. The test specimens remained exposed to the aggressive environment for periods of 3 and 6 months after which they were subjected to mechanical tests and physico–chemical analysis. Comparisons of results were made, as provided in Figures 6 and 7.

The selection of different types of aggressive media was based on ensuring the broadest range of substances to which concrete is typically exposed in an industrial environment.
The individual types of selected aggressive media, including their characteristic properties, are provided in Table 4.

Characteristic of medium		
Substance	*Concentration*	*Relative humidity*
Gaseous - CO_2	98%	75%
Gaseous - SO_2	98%	75%
Sulphates - Na_2SO_4	36 000 $mg \cdot l^{-1}$	--
Chlorides – NH_4Cl	2 000 $mg \cdot l^{-1}$	--
Engine oil	100%	--
Effect of atmospheric action	Outside storage	

Table 4: Specification of corrosive media

Throughout the storage period, the concentration of the different aggressive substances used in the tests was controlled and maintained at constant value.

4 RESULTS

4.1 Reactive Powder Concrete (RPC)

The following graphs provide a summary of results from mechanical tests on concrete mixes RPC I and RPC II after 6 months (180 days) of storage under different aggressive conditions. Comparative values between the two types of RPC concrete mixes of compressive and tensile strength and dynamic elastic modulus are given in Figures 1, 2 and 3 respectively. Reference samples refer to those that were stored in standard conditions and not subjected to any special environmental conditions.

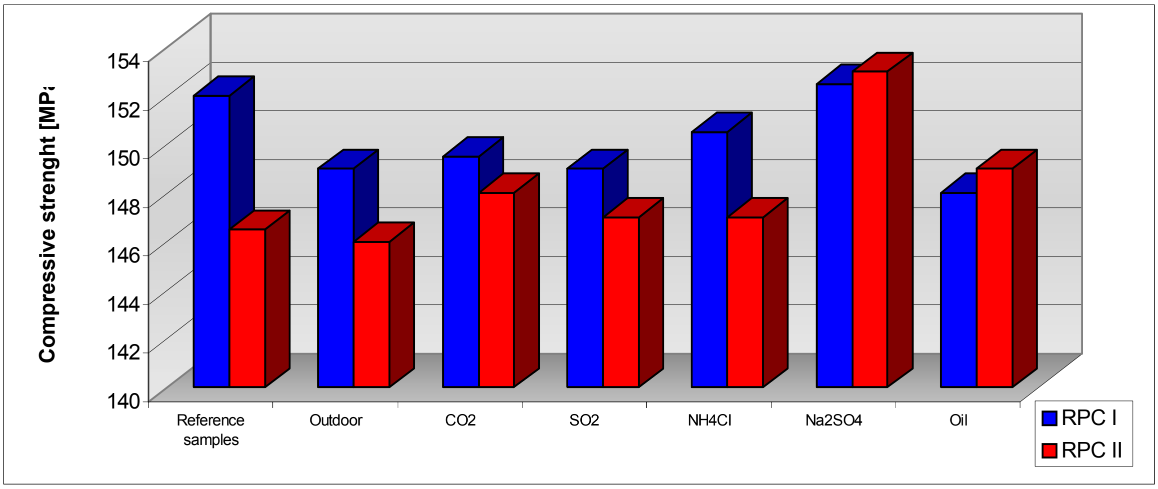

Fig. 1: Comparison of compression strength of RPC I and RPC II after 180 days storage in different aggressive media

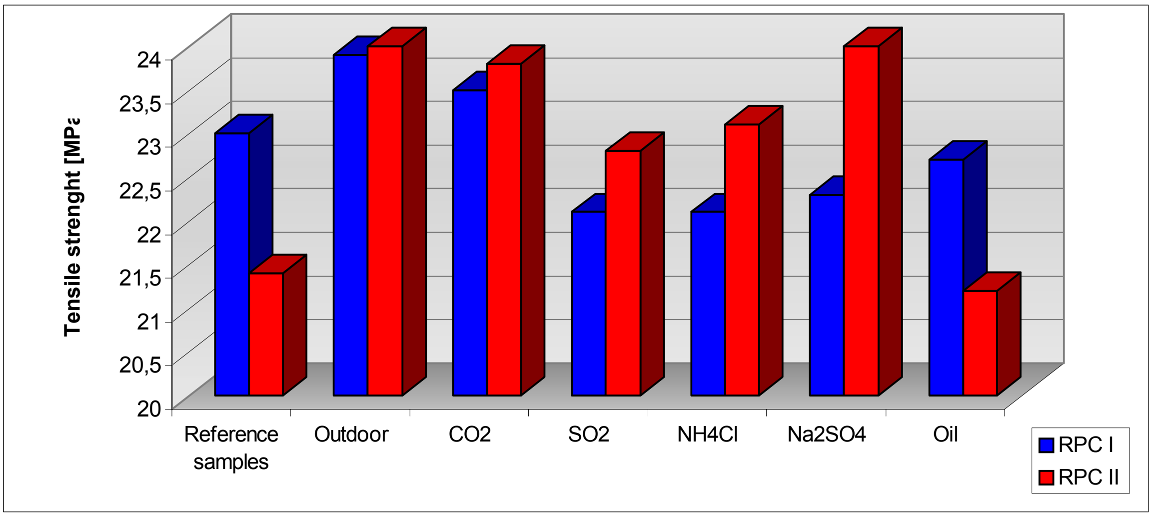

Fig. 2: Comparison of bending strength of RPC I and RPC II after 180 days storage in different aggressive media

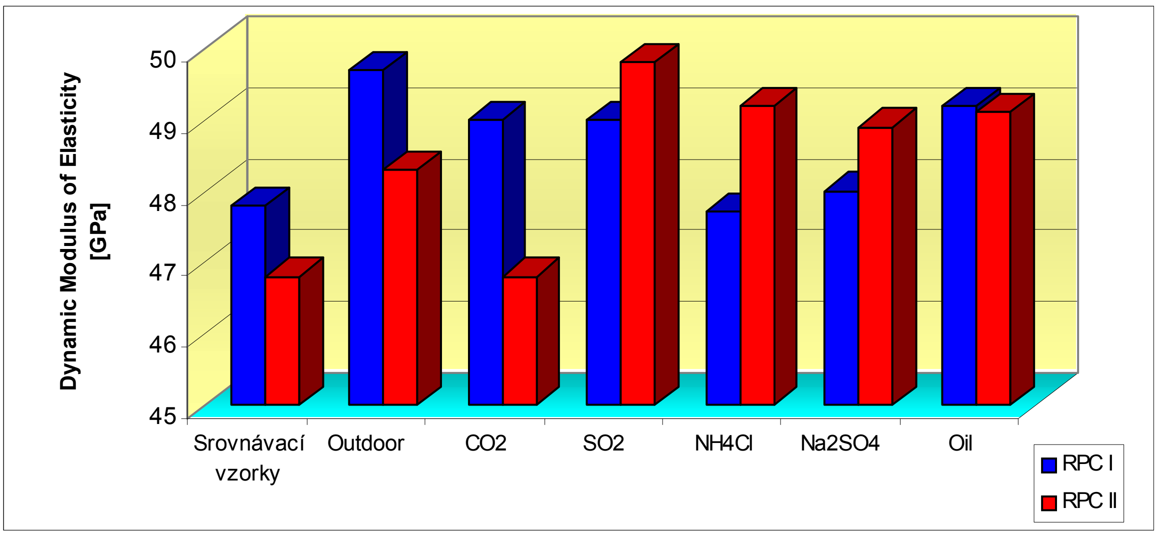

Fig. 3: Comparison of dynamic elasticity modulus of RPC I and RPC II after 180 days storage in different aggressive media

4.2 Self Compacting Concrete (SCC)

The summary of analysed SCC I and SCC II properties after 6 month of storing under aggressive conditions you will find in following graphs.

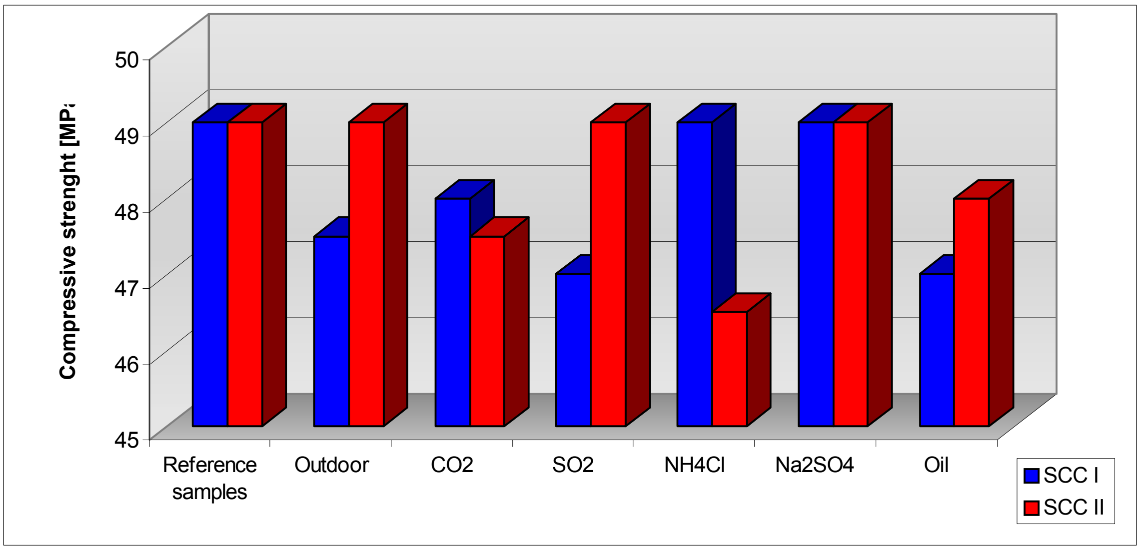

Fig. 4: Comparison of compression strength of SCC I and SCC II after 180 days storage in different aggressive media

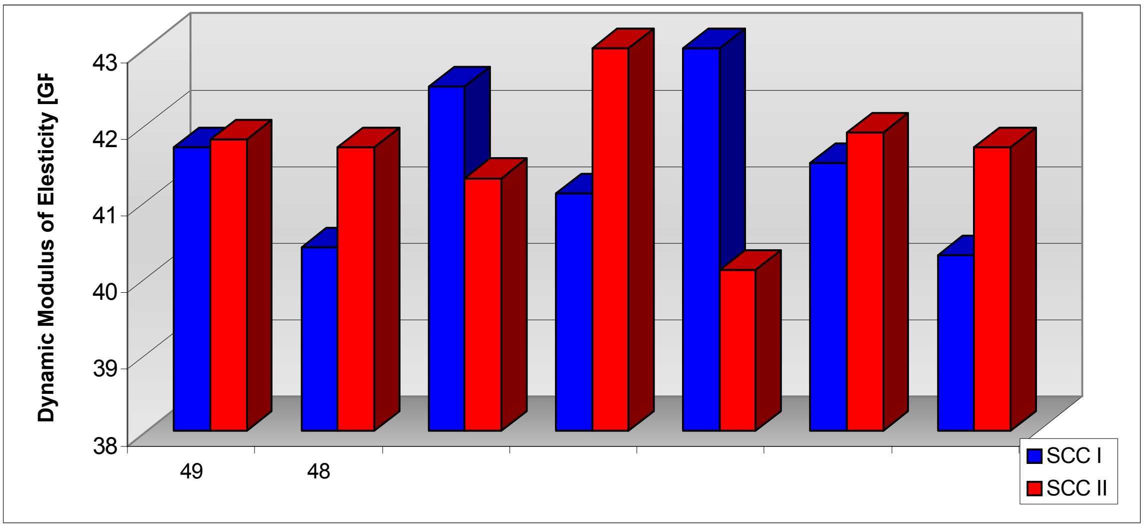

Fig. 5: Comparison of dynamic elasticity modulus of SCC I and SCC II after 180 days storage in different aggressive media

4.3 High Performance Concrete (HPC)

The summary of analysed HPC I and HPC II properties after 6 month of storing under aggressive conditions you will find in following graphs.

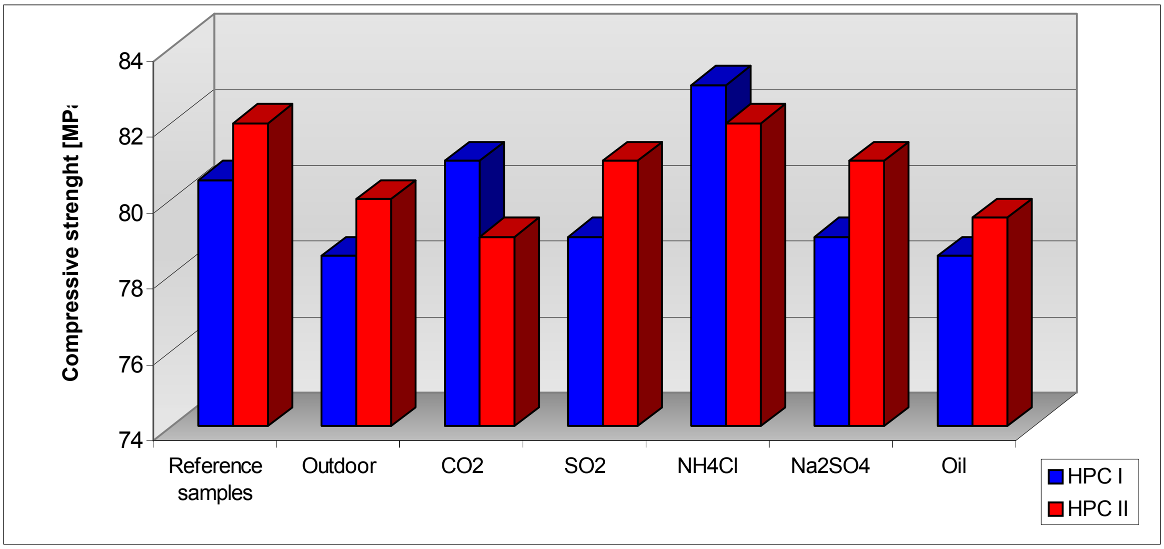

Fig. 6: Comparison of compression strength of HPC I and HPC II after 180 days storage in different aggressive media

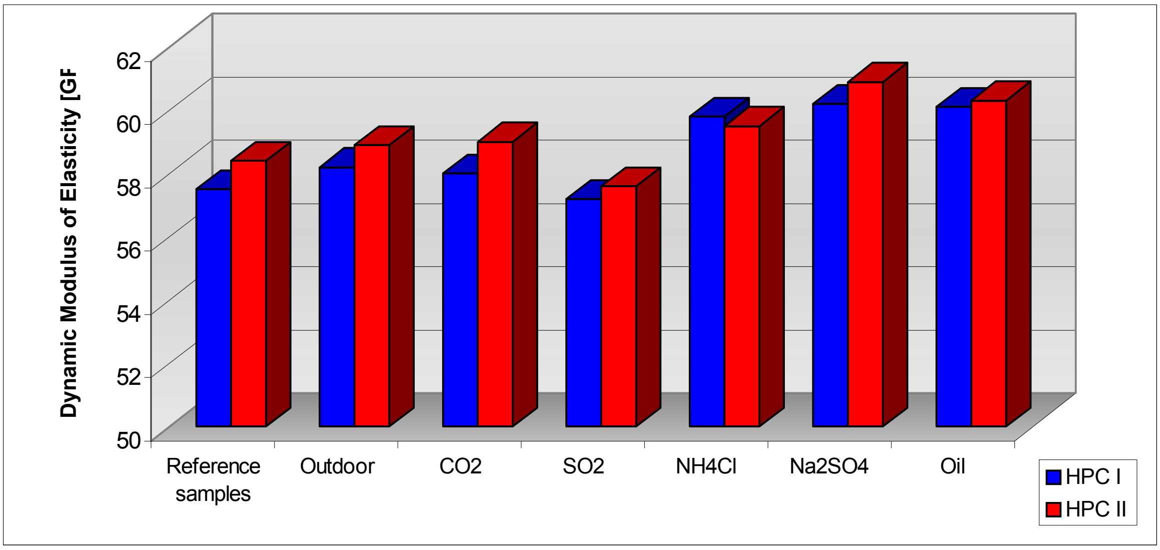

Fig. 7: Comparison of dynamic elasticity modulus of HPC I and HPC II after 180 days storage in different aggressive media

5 CONCLUSIONS

Concrete modified by the addition of waste raw material (power plant fly ash) and by admixtures having properties similar to pozzuolana attained comparable physico-mechanical properties as those of standard PC concrete. Specifically, the following was noted:

- Reactive Powder Concrete (RPC) modified by fly ash (5 % of filler mass) had moderate increases in flexural and compressive strength ;values of dynamic modulus of elasticity were approximately the same.
- Self Compacting Concrete (SCC) modified by power plant fly ash (20% of cement mass) had lower values of compressive strength than concrete without modification . This was caused

by the higher water/cement ratio necessary to achieve the same consistency of analysed concrete and mainly due to the effect of a slower rate of reaction from fly ash pozzuolana.
- Replacement of 10 % of cement mass by meta-kaolin in HPC has no significant effect on the quality of HPC; the values of compressive strength and dynamic elasticity modulus are nearly identical.

The effect of waste raw materials (admixtures) on the durability of concrete was evaluated in phase II of study. No significant changes in concrete properties (e.g. compressive and flexural strength, dynamic modulus of elasticity) have taken place after 6 months of storage in selected aggressive chemical media. As well, the appearance and mass of test specimens did not change. This was supported by a study of the phase composition using X-ray analysis reported elsewhere.

In regard to the partial substitution of binder (filler) for power plant fly ash or meta-kaolin (HCP) in the concretes evaluated in this study (i.e. RPC, SCC), it can be stated that the substitution has not caused more significant change in concrete properties as compared to standard concrete not even during an exposure period of 6 months in aggressive media. However, it is generally known that the effect of aggressive chemicals on concrete usually bring about changes after longer periods of exposure and therefore these effects will be subsequently reviewed after 48 months of exposure to aggressive chemical media.

6 ACKNOWLEDGMENTS

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7 REFERENCES

Matoušek, M., Drochytka, R.: *Atmospheric Corrosion of Concrete*, IKAS Praha 1998 (in Czech)

Drochytka, R.: *Atmospheric Corrosion of Concrete and Cellular Concrete*. Sborník vědecké konference VUT FAST Brno, 1984, sv. B-97, pp. 209-213 (Proceedings of Konference at Brno University of Technology (in Czech)

Biocatalytic Processes on Concrete: Bacterial Cleaning and Repair

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ABSTRACT

Biological techniques for cleaning and repair of concrete and stone surfaces can be an ecological alternative for traditional conservation techniques.

Concrete specimens weathered for over a decade in the Belgian moderate climate, showing a black organic outer layer, mainly consisting of lichens, were cleaned with a new biological technique. The weathered samples, made with Portland cement or with blastfurnace slag cement, were treated with *Thiobacillus* bacteria and an appropriate nutrient, by submersion or sprinkling. The *in situ* production of acid metabolites resulted in a cleaning effect, which was documented by the use of colorimetry and microscopy. Differences in the effectiveness were seen between concrete samples of a different cement type. The sprinkling treatment was quite effective for regaining the original appearance of the concrete surface. A side effect was the formation of a gypsum layer on some of the specimens.

For remediation of decayed stone and concrete, biomineralisation by *Bacillus sphaericus* was investigated. The best microbial strains were selected based on criteria such as the amount of the microbiologically produced calcium carbonate, adhesion of the created layer to the stone surface and change in capillary water absorption. Next to application of pure bacteria cultures, mortar samples of different porosity were also treated with ureolytic sludge. The most pronounced reduction in water absorption was reached for the most porous mortar samples. When urea, nutrient broth and an external calcium source were provided, the amount of water absorbed by the mortar samples after 200 hours was decreased by a factor 5 compared to untreated samples. This treatment also caused a significant reduction in total porosity. SEM and XRD analyses showed that a dense layer of calcite and vaterite crystals was deposited on the mortar surface.

KEYWORDS

Biological cleaning, biomineralisation, calcium carbonate precipitation, concrete, bacteria

1 INTRODUCTION

Concrete weathering is a complex process and may include physical, chemical and biological factors. Weathering can induce an increased porosity and structural degradation of the surface layer and can result in an unattractive appearance. A proper cleaning procedure should not only be regarded as an aesthetical operation, but can also increase the service life of building materials. The cleaning of building façades is a delicate operation that can bring about irreparable damage if not carefully performed using appropriate techniques. Aesthetical damage traditionally is repaired by a combination of physical and chemical cleaning. In forensic practice, damage cases such as excessive abrasion, staining, deposition of soluble salts, and biological growths, resulting from the inaccurate use of cleaning procedures, are frequently encountered [e.g. Maxwell 1992; Young & Urquhart 1992; Warscheid & Braams 2000]. With the use of traditional chemical products, there is a risk of pollution, especially when the formulation of the cleaning product is unknown, or when products and rinsing water are discharged directly in the sewer. Moreover, cleaning personnel working with organic solvents may suffer from irritation of the eyes and of upper respiratory tract [Anundi *et al*. 2000].

As an alternative for chemical and physical cleaning techniques, with their described disadvantages, new biological methods have been proposed. Hempel [1978] was one of the first to address the possibility of biological cleaning. He noted the effectiveness of a clay poultice containing urea and glycerol and proposed that microorganisms were at least partially responsible. Kouzeli [1992] has reported favourably on the technique in comparison with pastes based on EDTA or ammonium bicarbonate. Several authors focused on the application of *Desulfovibrio* in the reconversion of gypsum crusts into calcite [Heselmeyer *et al*. 1991; Gauri *et al*. 1992]. For the removal of sulphates, nitrates and organic matter from artistic stone works, carefully selected microbial cultures have been used [Ranalli *et al*. 2000]. A similar methodology has been applied for the elimination of insoluble calcium oxalate patinas from monuments [Tiano *et al*. 1996]. Enzymes, such as lipase, have been successfully used to remove aged acrylic resin coatings in paintings [Bellucci *et al*. 1999]. Besides the mentioned cleaning techniques, several research groups over the world are performing tests on a microbially induced protective calcium carbonate layer. Different species are being used for this purpose: *Bacillus cereus* by the French group, *Bacillus pasteurii* by the Americans, *Myxococcus xanthus* by the Spanish group, and *Bacillus sphaericus* by our Belgian research team [Tiano *et al*. 1999; Bang *et al*. 2001; Castanier *et al*. 1999; Rodriguez-Navarro *et al*. 2003; Hammes *et al*. 2003]. However, since test conditions and parameters measured are not the same for different investigations, it is difficult to compare the capacities of the different species.

In the present article, a new cleaning procedure will be proposed, using bacteria of the genus *Thiobacillus* with an appropriate nutrient, in order to clean fouled concrete surfaces. Furthermore, concrete repair through biomineralisation by *Bacillus sphaericus* will be discussed.

2 MICROBIOLOGICAL CLEANING OF CONCRETE SURFACES

2.1 Principle

A new biological cleaning technique for concrete, using a biological sulphur solution called Thio-S is under development in our laboratories. Thio-S is a mixture of sulphur oxidising bacteria of the genus *Thiobacillus* with appropriate microbial nutrients. *Thiobacilli* are able to produce energy out of the oxidation of elementary sulphur and reduced inorganic sulphur bonds. *Thiobacilli* are acidophilic, or acid tolerant bacteria and are able to fix CO_2. The end product of the oxidation executed in their metabolism is sulphuric acid [Vincke *et al*. 1999]. The species *T. thiooxidans* can survive values below pH 1. The application of Thio-S on fouled concrete, results in acid production *in situ*, and in a local cleaning action. Since the organisms involved are very sensitive to desiccation and since further colonisation of building stones depends on the presence of reduced sulphur compounds, the application of Thio-S on façades can be adequately controlled.

2.2 Materials and methods

2.2.1 Concrete specimens

Two concrete cubes with sides of 200 mm, were weathered for over 10 years in a Belgian outside climate. One of the cubes was composed of blast furnace slag cement (BFS), the other was composed of ordinary Portland cement (OPC). The cubes were fouled with lichens and atmospheric pollution, forming a black patina. Using a reaction with potassium hydroxide and microscopic investigation, the fouling on the OPC cubes was characterized as *Lecanora albescens*, a white lichen common on mortars and calcareous stone; and *Candelariella aurella*, a dark grey crust with orange fruit bodies (apothecies). On the BFS cubes, only *Candelariella aurella* was found, but in slightly less dense crusts. Small concrete cubes with sides of approximately 4 cm, were sawed out of these cubes with a diamond cut saw. These small specimens had at least one fouled plane.

2.2.2 Biological sulphur solution

An active consortium of bacteria was selected by inoculation of biofilm material scraped off a corroded sewage pipe in mineral medium M35 (De Graef et al., 2003) and by repeated transfer of 10 ml of an acidified culture in the early stationary phase as inoculum to a new culture of 1 L at 28 °C. This procedure was repeated 4 times in triplicate until the fastest acidification rate was reached. The nutrient consisted of 10g/l powdered sulphur (S), 0.1 g/l NH_4Cl, 3.0 g/l KH_2PO_4, 0.1 g/l $MgCl_2.6H_2O$ and 0.14 g/l $CaCl_2.2H_2O$. The dissolved oxygen amount of the medium was set at a minimum of 5 mg/l. In previous experiments, the Thio-S consortium had been applied to concrete samples as an acidic cell suspension, when the micro-organisms had reached late stationary phase, at an ambient temperature of 20 °C. The pH at application amounted to 1.0 – 1.2. In the current experiment, the aim was to quantify the biological effect, i.e. the production of metabolites *in situ*. The cell mass of a culture in the early exponential growth phase was harvested by centrifugation (10 min at 5000 x g) and put into fresh medium, with an initial pH of 7. During the test, which was run at 28 °C, the acidification was monitored.

2.2.3 Test procedure

In a previous experiment, 5 cubes from each set of concrete samples were immersed in an acidified Thio-S solution of pH 1.0-1.2. Another 5 cubes were immersed in water in such a way that only the fouled surface surmounted the water level with about 1 mm. Through capillarity, this surface remained continuously moist. On this surface, Thio-S was sprinkled with a brush four times a day. Simultaneous to this treatment, 3 cubes of each set were completely immersed in water as a control, and 3 cubes of each set were immersed in a sulphuric acid solution of the same initial pH as the Thio-S solution. These treatments all had a duration of three days per cycle. Three cycles were performed, in an atmosphere of 20 °C and 60 % relative humidity (RH). The pH of the Thio-S with the immersed concrete cubes rose during the test because of the high alkalinity of the concrete. The average concentration of sulphate after a cleaning cycle of three days in Thio-S fluid was 9.9 g SO_4^{2-}/l. After each treatment cycle, the cubes were dried for 4 days in an atmosphere of 35 °C and a RH of 40%. These experiments indicated that the proposed technique was 30 to 100% more effective on concrete with ordinary Portland cement than on blast furnace slag cement samples. The sprinkling treatment was about 50% as effective as the submersion treatment (which had in turn a similar or higher effectiveness than submersion in sulphuric acid solution), but still had a good cleaning potential and had only an effect on the outer material surface. In the current test set-up it was the aim to apply a solution of neutral pH, being safer to work with for the cleaning personnel. Biomass at the early exponential growth phase was harvested, suspended in fresh medium of neutral pH, and applied at 28 °C, through immersion or through sprinkling as described above. The test cycle was stopped after nine days, since the exponential growth phase had ended at that time (end of active acidification). After this, a second cycle was carried out, where the centrifuged biomass in neutral medium was put directly on the samples used for the sprinkling treatment, and where only the medium was sprinkled intermittently as described above.

2.2.4 Measurements to quantify the effect of cleaning

A X-rite SP60 colorimeter with a circular measurement area of 8 mm diameter was used to obtain spectral reflectance graphs of the fouled surfaces. The relative reflectance of light with wavelengths ranging from 400 nm to 700 nm was measured per 10 nm. The specular component of the reflected light was excluded. At the beginning of the test cycles, and after the drying period of 72 hours at 35 °C following each treatment cycle, three reflectance measurements were taken, evenly distributed over the fouled surface of each concrete cube. The mean reflectance curve of these three measurements was obtained, as an indication of the degree of fouling. In the case of concrete, the measured colours are all grey values, which results in a more or less horizontal reflectance graph. As an effect of the cleaning, the appearance of the concrete becomes lighter grey. Therefore the measured effect of the cleaning should be a higher curve, more or less parallel to the one of the fouled concrete and approaching the reflectance curve of clean concrete. An approximation of the values of clean concrete was made by measuring the saw planes in between the aggregates. The spectra can also be represented by tristimulus values, based on the spectral sensitivity of the human eye (CIE definition). In this case L*a*b* values under Standard Illuminant D65 (daylight standard) were chosen for the 10° standard observer (CIE 1964 supplementary colorimetric observer). The L* values range from 0 to +100 and respectively represent black and white. The negative and positive a* values represent green and red, respectively. The negative and positive b* values represent blue and yellow, respectively. These values can be plotted in a Cartesian co-ordinate system, called the L*a*b* colour space, with the a* and b* axes in the horizontal plane, and the L* axis perpendicular to that. In this colour space a colour difference can be expressed as the distance between the points of two colours $\Delta E^*_{ab} = \sqrt{(\Delta L^*)^2 + (\Delta a^*)^2 + (\Delta b^*)^2}$. In the case of concrete, the colour difference ΔE^*_{ab}, is a distance more or less parallel to the L* axis. A greater colour difference between fouled and cleaned concrete means a lighter grey value, with more reflectance, and indirectly indicates the effectiveness of the cleaning of the concrete surface.

Viability staining was performed using commercial live/dead stain (L-13152, Molecular Probes, Leiden, The Netherlands). This stain allows fluorescence microscopy to distinguish between organisms with intact cell membranes (stained green and scored alive) and organisms with damaged cell membranes (stained red and scored dead). 25 µl stain was put directly on 1 cm^2 of mortar surface and was incubated for 10 min in the dark and examined by standard epifluorescence microscopy. The microscope was equipped with a Peltier cooled single chip digital colour CCD camera and connected to a PC to obtain digital images. For each treatment, two preparations were examined under fluorescent microscopy.

2.3 Results

In Fig. 1, the average colour difference ΔE^*_{ab} between fouled and cleaned concrete is shown for two Thio-S treatment cycles with acidification *in situ*. Ordinary Portland cement samples and blast furnace slag cement samples are portrayed with their respective reference values, i.e. the mean colour difference between the fouled surface and the colour of the saw planes measured in between the aggregates. After one cycle of nine days, an effect was noticeable on the specimens of the sprinkling treatment. No significant effect was apparent on the specimens that underwent the immersion treatment. This could be due to limited diffusion of CO_2 and O_2 to the submerged samples. Presumably, the sprinkling treatment provided a good environment for the organisms to settle and to perform their cleaning action, provided that the humidity was sufficient. This cleaning cycle had a duration of 9 days, because the acidification was monitored during the test (Table 1), and the test was stopped at the end of the exponential growth phase (end of active acidification). After this, a second cycle was carried out, where the centrifuged biomass was put directly on the samples used for the sprinkling treatment, and where only the medium was sprinkled on intermittently as described above. For the immersed treatment, which was performed in the same way as the first cycle, this cycle was more effective than the first cycle, although the effect remained small. On the other hand, for the sprinkling treatment, the reference value, i.e. the mean colour difference between the fouled surface and the colour of the saw planes measured in between the aggregates, was quite well approached after this second cycle, for OPC as well as for BFS specimens. Table 1 shows that acidification was limited in the second cycle of the submersion treatment. The results suggest however that acidification did take place in the case of the second sprinkling cycle.

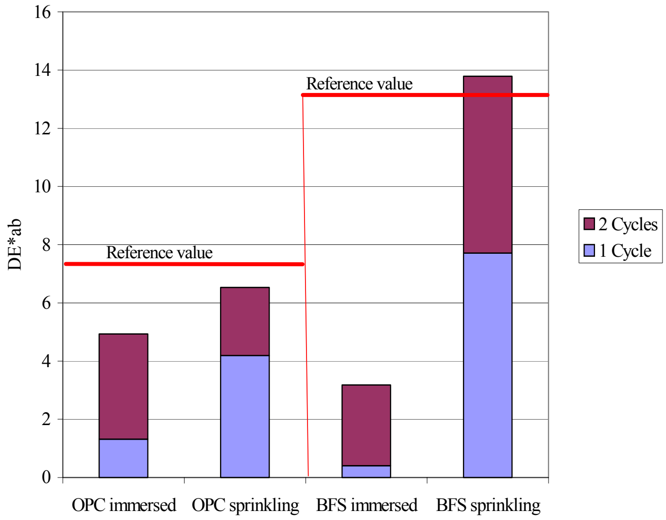

Figure 1. Effect of two Thio-S treatments on fouled OPC cubes and BFS cubes with acidification *in situ* (average of measurements on 3 samples per cycle). The average standard deviation amounted to ΔE^*_{ab} = 0.20 for the first cycle with immersed treatment and 1.85 for the first cycle with sprinkling treatment, and 0.24 and 1.05 respectively for the second cycle.

	Initial	Day 1	Day 2	Day 3	Day 4	Day 5	Day 7	Day 9
Cycle 1	7.85	7.70	6.89	6.30	3.51	2.81	2.90	3.29
Cycle 2	5.20	6.09	5.30	4.75	5.01	4.76	4.37	4.30

Table 1. pH evolution of the Thio-S used in the immersed treatment for measuring the *in situ* production of metabolites

Clusters of live cells could be detected on the mortar cubes treated with the Thio-S culture, which were not seen on untreated specimens. These cells were present as groups of attached individual cells and also as organised in biofilm structures. It was proposed that these cells represented active *Thiobacillus sp.* cells from the Thio-S culture. The results were similar for Portland and blast furnace slag cement samples. These results suggest that the micro-organisms can use the concrete as a substratum and that they can locally produce sulphuric acid and as such exert a cleaning effect.

3 MICROBIOLOGICAL REPAIR OF CONCRETE SURFACES

3.1 Principle

The general term biomineralisation refers to biologically induced mineralization in which an organism creates a local micro-environment, with conditions that allow optimal extracellular chemical precipitation of mineral phases [Hamilton 2003]. *Bacillus sphaericus* is able to precipitate $CaCO_3$ on its cell constituents and in the environment by degradation of urea into ammonia and carbon dioxide [Hammes et al. 2003]. The bacterial degradation of urea apparently increases the pH at the cell surface and this promotes the microbial deposition of carbon dioxide as calcium carbonate [Warren et al. 2001]. Through this process the bacterial cell is coated with a layer of calcium carbonate of increasing thickness, resulting in death of the microorganism. However, in the meantime a loose carrier material such as sand can be bound together, or a protecting layer can be deposited on damaged concrete or stone surfaces. Biomineralisation technology has been succesfully applied on limestone monuments [Castanier *et al*. 1999; Tiano *et al*. 1999] and results show that the characteristics of this biological coating even improve with time. In our research groups, first the criteria for the selection of calcium precipitating *Bacillus* strains were established. *Bacillus sphaericus* strains capable of the remediation of Euville limestone, by precipitating a dense and coherent calcium carbonate layer and concomitantly inducing a reduction of capillary water absorption, were characterised by a high urease activity, abundant EPS-production, a good biofilm production and a very negative ζ-potential [Dick *et al.* 2004]. In the current research mortar samples of different porosity were treated with ureolytic sludge (which is cheaper than the use of pure bacteria cultures and allows fast biomass production) and the effect of $CaCO_3$ deposition on water absorption and porosity was determined.

3.2. Materials and methods

Standardized mortar prisms of 40 x 40 x 160 mm were prepared with ordinary Portland cement (OPC) or blast furnace slag cement (BFS). Prisms were not only made with a water-to-cement ratio (w/c) of 0.5, but also with w/c ratios of 0.6 and 0.7, to obtain a more porous microstructure (simulating a degraded material). Cubes with sides of 40 mm were sawn out of these prisms. Ureolytic sludge was obtained through cultivation of active sludge, obtained from an aerobic sewage water treatment plant, in a semi continuous active sludge (SCAS) reactor. The SCAS reactors were filled with 1 litre activated sludge. After sedimentation in Imhoff cones, 300 ml of supernatant was replaced by the same volume of tap water, containing 1 g/l nutrient broth, 5 g/l urea and 10 g/l SLM 1228. One g/l of the SLM 1228 represents a chemical oxygen demand (COD) of 1135 mg/l, a phosphorus concentration of 50 mg/l and a Kjeldahl N concentration of 44 g/l. The reactors were continuously stirred at 150 rpm and 28 °C. Every second day, part of the reactor content was replaced, using the procedure described above. This procedure offered ureolytic bacteria a selective advantage and therefore stimulated their growth in the sludge microbial community. An adaptation period of 7 days was respected, to start the experiment with a system in equilibrium. The dry matter content and the concentration of volatile organic compounds were monitored continuously and amounted to 20.68 ± 1.14 g/l and 13.8 ± 1.01 g/l respectively. On three surfaces of the mortar cubes a paste of centrifuged ureolytic sludge (10 minutes at 8075 x g) of 0.5 – 1 mm thickness was applied. After 10 minutes settling, allowing the paste to attach itself more or less to the mortar surface, the mortar cubes were immersed in solutions of varying composition in order to investigate the effects of the provided nutrient and of an external calcium source (Table 2).

Treatment	1	2	3	4	5
Cement type	OPC	OPC	OPC	OPC	BSC
Number of samples					
w/c = 0.5	3	2	1	1	3
w/c = 0.6	3	2	1	1	3
w/c = 0.7	3	2	1	1	3
Biomass	yes	yes	yes	no	yes
Nutrient medium (g/l)					
$CaCl_2.2H_2O$	90	90	-	-	90
Urea	25	25	25	-	25
Nutrient broth	-	26	-	-	26

Table 2. Overview of the different test series according to cement type, presence of biomass and composition of the nutrient medium

The concentration of the different medium components per treated surface area was the same in all experiments (e.g. 0.105 g $CaCl_2.2H_2O$ per cm^2 of mortar surface). The cubes were removed from the solution after deposition of a crystalline layer on the surface, which was generally after 2 to 3 days.

To determine the increase in water penetration resistance obtained by depositing the $CaCO_3$ layer, a modified version of the sorptivity test, based on the Belgian standard NBN B 05-201, was carried out.

The mortar specimens were coated at the four edges adjacent to the treated side, to ensure unidirectional absorption through the treaded side. The coating existed of two layers of polysiloxane and one layer of silicon paint. After coating, the test cubes were dried at 70 °C in a ventilated kiln, establishing a mass equilibrium of less than 0.1% between two measurements at 24 hour intervals, to ensure low uniform moisture content in the matrix. The specimens were then exposed, to 10 +/- 1 mm of water, with the treated side facing downwards. This takes place in an atmosphere of 20 °C and relative humidity of 60%. At regular time intervals, the specimens are removed from the water and weighed, after drying the surface with a wet towel. Immediately after the measurement the test specimens are submerged again. After the last measurement (200 h of capillary water absorption) the specimens were dried at 70 °C and the mass (m_1) determined. Then, a vacuum saturation was performed in order to establish a full saturation. After application of a vacuum (2.7 kPa for 2.5 hours), water was injected in such a way that the samples were completely submerged within 1 h. This submerged state was maintained for 24 h and the surface-dry weight of the samples was measured. The capillary water absorption $E_{c,t}$ during the first part of the experiment is expressed as $E_{c,t} = \frac{m_t - m_1}{m_1} \times 100$ (%), while the water absorption under vacuum E_v is expressed as

$$E_V = \frac{m_v - m_1}{m_1} \times 100$$ (%) with m_1 the initial mass of the test cube after drying in the oven at 70 °C; m_t the mass at time t after the start of the water absorption test and m_v the mass after water absorption under vacuum. The results of the capillary absorption measurement can then be expressed as the relative impregnation rate (S_t) on a certain moment t

$$S_t = \frac{E_{c,t}}{E_v} \times 100 \ (\%)$$

The sorptivity of the cubes is calculated as the slopes of the functions representing the volume of absorbed water per surface area, versus the square root of time. The total porosity is estimated by calculating the volume of absorbed water after vacuum saturation.

The morphology and mineralogical composition of the deposited $CaCO_3$ crystals were investigated with scanning electron microscopy and X-ray diffraction.

3.3 Results

For the treated cubes the lowest water/cement ratio resulted in the fastest increase of relative impregnation rate S_t at the beginning of the experiment (slope of the S_t versus time curve) and also the highest final value at 200 hours (e.g. for treatment 2: $S_{t,final}$ = 20% for w/c = 0.7 and $S_{t,final}$ = 40% for w/c = 0.5), while the untreated cubes showed an opposite trend (for treatment 4: $S_{t,final}$ = 90% for w/c = 0.7 and $S_{t,final}$ = 80% for w/c = 0.5). All treatments resulted in a reduction of the slope of the S_t versus time curves and final S_t values in comparison with untreated cubes. The largest effect was seen for the cubes with w/c = 0.7 which underwent treatment 2. The treated cubes with blastfurnace slag cement (treatment 5) showed higher slopes of the S_t versus time curves and higher final S_t values (50-65%) when compared to the Portland cement cubes which underwent the same treatment. The slope of the curve provides information on the initial rate of water absorption, while the final impregnation rate allows one to judge the effectiveness of the treatment after prolonged exposure to water. For the cubes with lower w/c, which are normally less porous, the water absorption under vacuum E_v will be lower, and therefore the S_t value can be higher. This effect could be noticed for the treated cubes.
The sorptivity curves of the untreated samples typically have a bilinear shape with a fast increase of sorptivity up to 5-6 hours, after which the curve levels off (Fig. 2). For the treated samples a linear change with a highly reduced slope, can be noticed within the measurement interval (Fig. 2). The final sorptivity values were somewhat higher for treatment 3 (0.15 – 0.19 cm^3/cm^2) than for treatments 1 and 2 (0.09 – 0.14 cm^3/cm^2), which indicates the effect of an externally supplied calcium source. The differences between cubes with different w/c ratio were not significant. The most pronounced reduction in water absorption compared to untreated samples was reached for the most porous mortar

(w/c = 0.7) and when urea, nutrient broth and an external calcium source were provided (treatment 2): the amount of water absorbed by the mortar samples after 200 hours was then decreased by a factor 5.

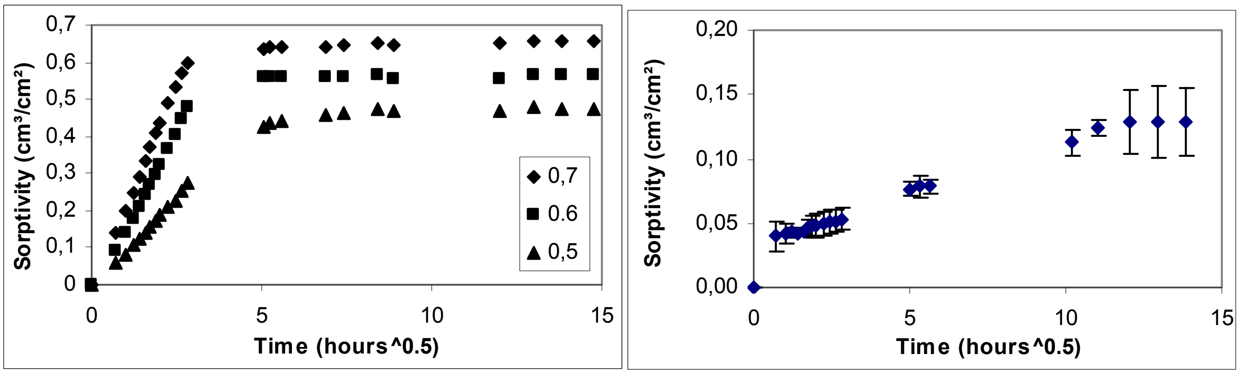

Figure 2. Sorptivity of the untreated Portland cement mortar samples with different water/cement ratios (left); sorptivity of Portland cement mortar samples with water/cement ratio 0.7 treated with biomass, urea, $CaCl_2.2H_2O$ and nutrient broth (right)

All treatments also caused a reduction in total porosity from 14-18% for untreated Portland cement cubes to 6-9% for treated cubes. SEM and XRD analyses showed that a dense layer of calcite and vaterite crystals was deposited on the mortar surface. For Portland cement mortar samples, no effect of the treatment on the carbonation depth was observed.

4 ACKNOWLEDGEMENTS

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5 REFERENCES

Anundi H., Langworth S., Johanson G., Lind M.L., Akesson B., Friis L., Itkes N., Soderman E., Jonsson B.A.G. & Edling C. 2000, 'Air and biological monitoring of solvent exposure during graffiti removal', *International archives of occupational and environmental health*, **73** [8], 561-569.

Bang S.S., Galinat J.A. & Ramakrishnan V. 2001, 'Calcite precipitation induced by polyurethane-immobilised Bacillus pasteurii', *Enzyme and Microbial Technology* **28**, pp. 404-409.

Bellucci R., Cremonesi P. & Pignagnoli G. 1999, 'A preliminary Note on the use of enzymes in conservation: the removal of aged acrylic resin coatings with lipase', *Studies in Conservation*, **44**, 278-281.

Castanier S., Le Métayer-Levrel G., Perthuisot J.P. 1999, 'Ca-carbonates precipitation and limestone genesis – the microbiogeologist point of view', *Sedimentary Geology* **126**, pp. 9-23.

De Graef B., De Windt W., De Belie N., Verstraete W. 2003, 'Bacterial cleaning of concrete surfaces', Proceedings of the International RILEM Conference on Microbial Impact on Building Materials', RILEM PRO 34, RILEM TC 183-MIB, Lisbon, 8-9 September 2003, pp. 27-35.

Dick J., De Windt W., De Graef B. De Belie N., Verstraete W. 2004, 'Bio-deposition of a calcium carbonate layer on degraded limestone by species of the *Bacillus sphaericus* group'. Submitted.

Gauri K.L., Parks L., Jaynes J. & Atlas R. 1992, 'Removal of sulphated-crust from marble using sulphate reducing bacteria', in *Stone Cleaning and the Nature, Soiling and Decay Mechanisms of Stone*, ed. R. G. M. Webster, Donhead, London, pp. 160-65.

Hamilton, W.A. 2003, 'Microbially influenced corrosion as a model system for the study of metal microbe interactions: A unifying electron transfer hypothesis', *Biofouling*, **19**[1], 65-76.

Hammes, F., Boon, N., de Villiers, J., Verstraete, W., Siciliano, S.D. 2003, 'Strain-specific ureolytic microbial carbonate precipitation', *Applied and Environmental Microbiology*, **69**[8]: 4901-4909.

TT3-233, Biocatalytic processes on concrete: bacterial cleaning and repair
N. De Belie, B. De Graef, W. De Muynck, J. Dick, W. De Windt, W. Verstraete

Hempel K. 1978, 'The biological pack', Rilem International Symposium on Deterioration and Protection of Stone Monuments, Paris, 5-9 June 1978, sec. 7.6.

Heselmeyer K., Fischer U., Krumbein K.E. & Warscheid T. 1991, 'Application of *Desulfovibrio vulgaris* for the bioconversion of rock gypsum crusts into calcite', *BIOforum ½*,

Kouzeli K. 1992, 'Black crust removal methods in use: Their effects on Pentelic marble surfaces', 7th International Congress on Deterioration and Conservation of Stone, Lisbon, pp. 1147-1156.

Maxwell I. 1992, 'Stonecleaning - for better or worse? An overview', in *Stone Cleaning and the Nature, Soiling and Decay Mechanisms of Stone*, ed. R. G. M. Webster, Donhead, London, pp. 3-49.

Ranalli G., Matteini M., Tosini I., Zanardini E. & Sorlini C. 2000, 'Bioremediation of Cultural Heritage: Removal of Sulphates, Nitrates and Organic Substances, of Microbes and Art - The role of Microbial Communities in the Degradation and Protection of Cultural Heritage', eds Ciferri O., Tiano P., Mastromei G., Kluwer Academic- Plenum Publisher, pp. 231-245.

Rodriguez-Navarro C., Rodriguez-Gallego M., Ben Chekroun K., Gonzalez-Muñoz M.T. 2003, 'Conservation of ornamental stone by Myxococcus xanthus-induced carbonate mineralisation', *Applied and Environmental Microbiology*, pp. 2182-2193.

Tiano P., Biagiotti L., Mastromei G. 1999, 'Bacterial bio-mediated calcite precipitation for monumental stones conservation: methods of evaluation', *Journal of Microbiological Methods* **36**, 139-145.

Tiano P., Tosini I., Rizzi M. & Tsakona N. 1996, 'Calcium oxalate decomposing microorganisms: a biological approach to the oxalate patinas elimination', II Inter. Symp. The oxalate films in the conservation of works of arts, Milan, 25-27 March 1996, Editeam (539), pp. 489-493.

Vincke E., Verstichel S., Monteny J., Verstraete W. 1999, 'A new test procedure for biogenic sulphuric acid corrosion of concrete', *Biodegradation* **10**[6], pp. 421-428.

Warren, L.A., Maurice, P.A., Ferris, F.G. 2001, 'Microbially mediated calcium carbonate precipitation: implications for interpreting calcite precipitation and for solid-phase capture of inorganic contaminants', *Geomicrobial Journal*, **18**, 93-115.

Warscheid T. & Braams J. 2000, 'Biodeterioration of stone: a review', *International Biodeterioration & Biodegradation*, **46**, 343-368,

Young M. E. & Urquhart D., 1992, 'Abrasive cleaning of sandstone buildings and monuments: An experimental investigation', in *Stone Cleaning and the Nature, Soiling and Decay Mechanisms of Stone*, ed. R. G. M. Webster, Donhead, London, pp. 128-40.

Innovative prefabrication system in Housing Block Construction

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TT3-244

ABSTRACT

Prefabrication is substantially used in housing block construction in Hong Kong to expedite the construction cycle, providing a green construction environment and achieving higher quality construction. Experience on the use of volumetric modules like prefabricate toilet and staircase indicated there are benefits in quality control. However, drawbacks are also identified. The paper overviewed the drawbacks on existing construction method with a view to develop innovative precast system. Conceptual study on alternative precast system indicated the use of lightweight concrete can reduce the weight of precast component, allowing design flexibility to overcome difficult site conditions and significantly shorten the construction time.

KEYWORDS:

lightweight concrete, prefabrication.

INTRODUCTION

Hong Kong is one of the developed cities with the highest density ratio of high rise residential building. The demand for high rise building is great to accommodate the increasing population for limited space. Since 90's prefabrication was used in the housing block construction in Hong Kong in order to expedite the speed of construction, quality of work and for improving environment condition of construction sites. For a standardized housing block, the quantity of precast units is approximately 17% of the total concrete volume. Most of the precast components were used as non-structural element.

Prefabrication provides a solution to expedite the construction process and significantly improve the quality of work. It allows the precast component to manufacture in the precast yard on ground level instead of the on-site concreting work at high level. Moreover, multiple tasks can be scheduled by detail planning so that construction duration can significantly be reduced.

This study is a part of the comprehensive investigation on the development of innovative present system for housing block construction. It overviews the prefabrication method currently adopted in Hong Kong standard housing block construction. Questionnaire survey was carried out to gather the view points on the adoption of prefabrication system. 14 stakeholders including engineering consultants, designers, precast manufacturers and contractor were interviewed. Findings of the questionnaire form the bases of conceptual development of innovative precast system. Opinion on the followings areas was gathered;

- Existing prefabrication industry,
- Strength on current practice,
- Limitations,
- Areas of improvement,
- Ideas towards innovative prefabrication system.

ADVANTAGES OF PREFABRICATION SYSTEM

<u>Standardization construction</u>

Public housing constituted a key portion of construction work in Hong Kong and it has adopted standardized block design. The use of standardize building elements allows the precast components can be mass production. The findings of this study confirmed are advantages of adopting prefabrication in housing block construction. The precast components can be produced under factory condition with re-useable steel mould and achieving a higher degree of precision than the cast in-situ construction. The precastors can set up the production line and allocate a designated crew to monitor the manufacturing process. Table 1 summarized the details of precast modules used in standardized housing block.

Table 1: Details of precast components used in housing block

Type of precast	Length (mm)	Height (mm)	Thickness (mm)	Weight in Tonnes [normal concrete]
Precast Staircase	1650	2650	300	3.15
Precast Semi-slab	4655	2125	65	1.54
Precast Façade (Single storey)	4615	2700	200	3.20

Flexibility

Comparing with the traditional cast in-situ construction method, finding of this study confirmed that the conventional construction method is more labor intensive and relatively low in efficiency than the prefabrication method. The construction activities heavily rely on the site condition (e.g. weather).

Quality & Safety

The factory production of the prefabricated unit eliminated the formwork requirement and allows concreting work to be carried out on ground level; enabling a closer supervision for achieving higher quality products. In case defected work is encountered, it will cause serious delay of the construction progress and the costs of rectification works can be substantial in order to ensure the stability of the entire structure. For the prefabrication construction method, the defective precast components can be identified at the precast yard and the defected work can be rectified before delivered to site.

The use of prefabrication external wall does not require massive scaffolding. It required fewer work process to be performed at the external wall, thus eliminating the potential risk of working at height which is the key factors to foster accidents.

Expedite in construction progress and cost effectiveness

The distinctive characteristic of prefabrication enables multiple construction process to be preceded within the same timeframe. For example, whilst the foundation works proceeding on site, the precastor can start manufacturing precast components and provide steadily supply to the project. This significant reduce the construction time and ease the inflexibility inherited from the traditional cast in-situ construction method. Hong Kong Housing Authority successfully lowering the construction cost after utilizing the prefabrication construction method. The process of typical floor construction can be cycled and benefits from the economic of scale under the standard block design. The consolidated standardized construction cycle contribute to significant reduction in idle time and elimination of possible interruption occurred in the critical path which induced by the varying site condition. High degree of control throughout the project leads to significant decrease in resources, timely program planning, effective budget control and monitoring about the progress of the program.

Environmental Concern

Substantial use of prefabrication leads to less cast in-situ activities and reduce the ‘wet work’ performed on site. This contributes to less environmental impact (e.g, noise, dust, wastage) being induced to its surrounding area.

Prefabrication offers lower level of wastage in comparison with cast in-situ method. Since the precast components were factory produced, the site environment can be enhanced with significant reduction in material wastage. Stringent control on waste management reduces the adverse effects onto to air, ground and water.

DOUBLE FAÇADE SYSTEM

TT3-244. Innovative prefabrication system in housing Block Construction, Tommy Yu Lo, Jon C., H. Lee

Precast façade is one of the worth doing precast component used in housing block construction. Five distinguish type of precast facades (A,B,C,D,E as shown in Figure 1) were adopted in a typical floor of housing block with total number of 10 pieces of precast façade installed for each wing. As illustrated in Table 2, the maximum dimension for precast façade is 4.6m width x 2.7m height and with weight 3.2 tonne. It is the external envelop of a building. It is designated as non-structural component whilst its main purpose is to transfer the horizontal wind load to the cast in-situ core walls which taking the load bearing function to support the entire structural. With the adaptation of precast façade, the construction program of the typical floor of 4 wings standard block can be completed within a 6 days construction cycle.

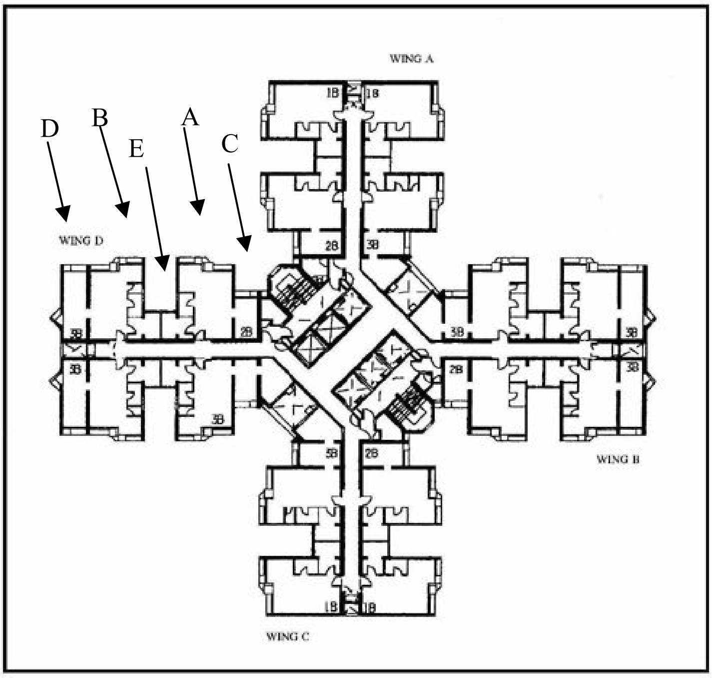

Figure 1: Layout Plan of Harmony Housing Block Type 1

Table 2: 5 types of façade component adopted in a typical housing block

Façade Type	Height (mm)	Length (mm)	Thickness (mm)	Weight using normal weight concrete (tonne)	Weight if constructed with lightweight concrete (tonne)
A	2700	4615	200	3.2	2.3
B	2700	4615	200	3.2	2.3
C	2700	2300	200	1.7	1.2
D	2700	2300	200	1.7	1.2
E	2700	2300	200	1.9	1.4

Merit of double precast façade over single storey precst facade

Conceptual study on alternative precast facade indicated that the use of lightweight double storey façade could reduce the total number of precast components utilized onto the building and contributes to the following advantages;

- Significant reduction in connection joint & effectively eliminate the potential areas of water leakage problems - even though the current design of façade had incorporate a 'boot' shape at base to protect the connection joint and increase the pressure different between internal and external environment. The window frame also fixed to mould of facade before concrete casting, there is a potential of water leakage for the precast unit. The use of double storey façade will significantly reduce the total number of connection joints.
- Combined 2 single storey façade components into one double storey precast façade can reduce of unit weight and the number of hoisting. As illustrated in Figure 2 that the conversion of 2 single precast façade into a double storey façade can effectively reduce the number of boots and therefore reduce the total weight for hoisting. Table 3 compared the weight different between single storey façade and the double storey façade under investigation.
- It is seen that the weight can be further reduced by 30% if lightweight concrete is used, leaving more flexibility in dimension changes to the architecture.
- It's of utmost importance that the double façade approach is made based on the assumption that the existing crane system does not change.
- The fewer crane usage allows for rescheduling of the existing work program and reallocation of resource. The shortening the construction time and expedite in progress implies cost saving with higher productivity and effectiveness.
- Less cast in-situ wet trades and fewer connection joints construction.

<u>Areas of consideration for proposed double storey precast façade</u>

There are several drawbacks for the double façade precast construction, they are:

- the complexity of the new precast system required reschedule of work program for each construction cycle
- a new works sequence based on 2 storey construction cycle is required
- Storage and handling may pose physical constraints of the precast components on site
- The change on the pattern of 'boot' on every floor to alternative floors may raise the aesthetic issue. Additional architectural feature may be required to preserve the consistent outlook for the structure.
- The practical problem of erecting, hoisting and placing the double storey façade onto the designated location at high level may exist. Provision of temporary bracing and supporting strut is required to stabilize the double height façade panel.
- There are space problem for the storage of precast unit during construction.
- The joint for the double façade will be different to the existing single storey façade construction method.

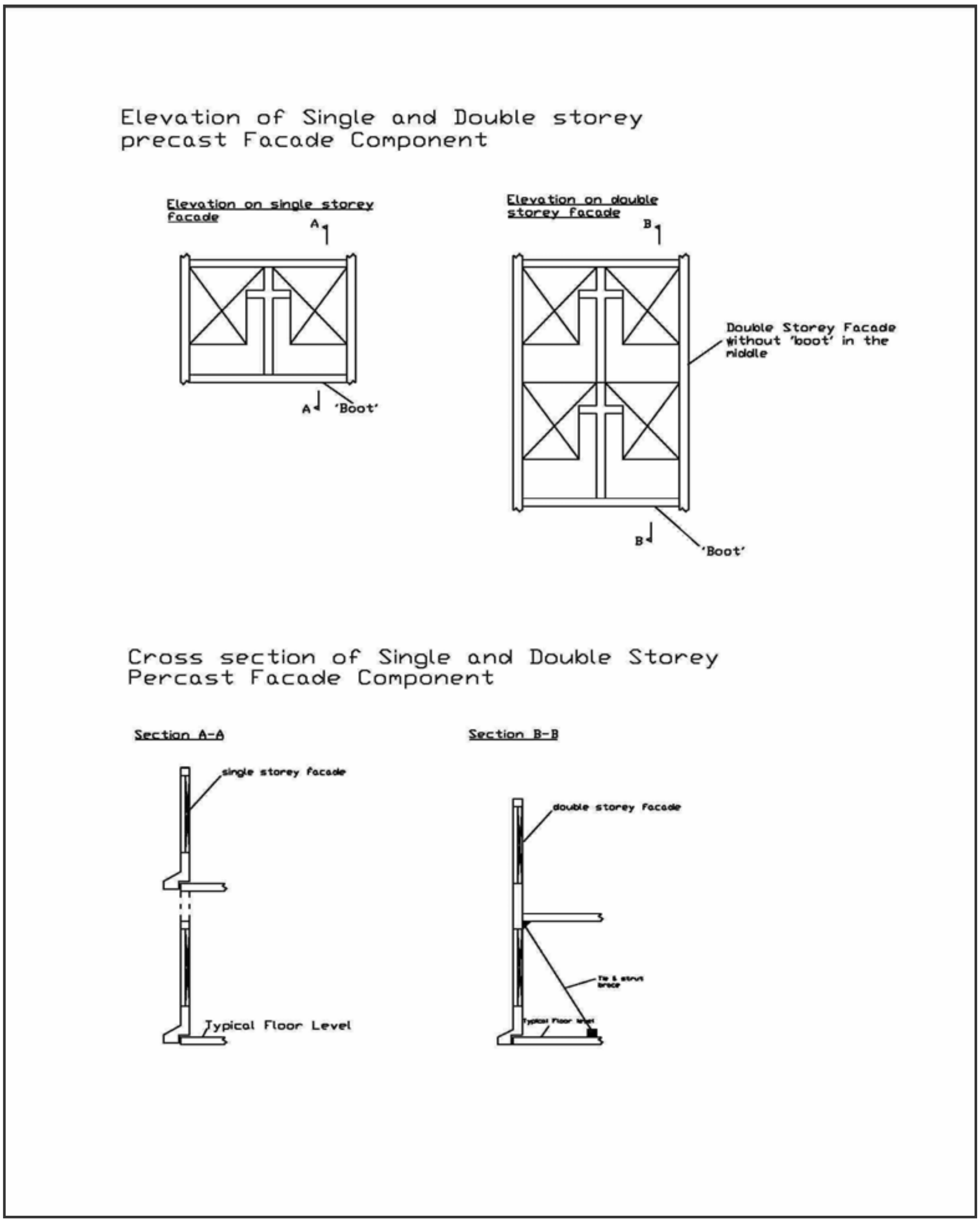

Figure 2: Details of Single Storey façade and Proposed Double Storey Façade

Table 3: Weight comparison between single storey façade & the double storey façade under investigation

	Length	Height	Thickness	Concrete Volume	Unit weight in of Normal Concrete (Tonnes)	Unit weight in Lightweight (Tonnes)
Single storey Façade	4615	2700	200	2.49	3	2.3
Double storey Façade	4615	5400	200	4.98	6	4.4

CONCLUSIONS

It is concluded that the most outstanding merit of prefabrication construction method aligned with expedite of progress and shortening of construction time and multi-tasking. In addition, prefabricaiton construction method offer environmental friendly and green construction environment by significantly reduce the conventional 'wet work'/ cast in-situ works on site. The proposed double storey façade permits the combination of 2 precast components into one which significantly reduce the totally number of precast components to handle for the entire structure; significant reduce the number of joints and minimizes the risk of water leakage.

Acknowledgements

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Reference

1. A M Haas (1983) *Precast Concrete – Design and Applications*. UK: Applied Science Publishers.
2. Robert A Hartland (1975) *Design of Precast Concrete*. London: Surrey University Press in association with International Textbook Company Limited
3. M Levitt (1982) *Precast Concrete -- Materials, Manufacture, Properties and Usage.* UK: Applied Science Publishers.
4. J G Richardson (1973) *Precast Concrete Prouction.* London: Cement and Concrete Assoication.
5. David A Sheppard, William R Phillips (1989) *Plant-Cast Precast & Prestressed Concrete-- A Desgin Guide.* USA: McGraw-Hill Publishing Company.

Durability of UHPFRC specimens kept in various aggressive environments

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TT3-246

ABSTRACT

Ultra High Performance Fibre Reinforced Concretes are the kind of materials that can lead to the design of structures in which the fibres take the function of reinforcing bars. This means that the fibers should be able to carry loads even when structural cracks occur. In this context, an essential point for the use of UHPFRC in structural applications is to check their ability to retain the initial mechanical characteristics when the cracked element is subjected to aggressive environment.

In this paper, the results of an experimental study on the evolution of mechanical characteristics of pre-cracked specimens conserved in various conditions are presented. Two types of UHPFRC are evaluated, one reinforced with steel fibres and the other one reinforced with organic fibres. The aggressive environments are hot water, sodium chloride solution and wet-dry cycles. It is shown that the mechanical characteristics of cracked specimens are not affected in such environment.

KEYWORDS

UHPFRC, concrete, steel fibre, mechanical characteristic, durability

1 INTRODUCTION

Ultra High Performance Fibre Reinforced Concretes are the kind of materials that can lead to the design of structures in which the fibres take the function of reinforcing bars. Being inspired by the codes for the prestress concrete and the reinforced concrete, a design code was recently developed in France [AFGC 2002]. This method, based on Serviceability Limit State (SLS) and Ultimate Limit State (ULS), allows to design structures with the classical cracking criteria (non-critical cracking, critical cracking and very critical cracking). In this context, an essential point for the use of UHPFRC in structural applications is to check their ability to retain the initial mechanical characteristics when the cracked structural element is kept in an aggressive environment.

In this paper, the results of an experimental study on the evolution of mechanical characteristics of pre-cracked elements conserved in aggressive environment are presented.

2 GENERAL CONSIDERATIONS ON UHPFRC

Ultra-High Performance Fibre Reinforced Concretes (UHPFRC) has been developed in France since 1991, first with the Reactive Powder Concrete (RPC) [Richard & Cherezy 1995]. Several other UHPFRC were developed afterwards and today these products are finding many industrial applications. One of them, Ductal® jointly developed by Lafarge, Bouygues and Rhodia, is available as a premix and has been licensed in Japan, France, USA, Canada and Australia [Casanova *& al* 2000]. First aimed at ultra-high mechanical performances and ductility [Richard & Cherezy 1995], these materials also present high durability performances, thanks to their particular microstructure [Vernet et al 1998 ; Vernet, 2003]. Nine years of accelerated tests have shown that UHPFRC could also be called “Ultra-High-Durability Concretes” (UHDC).

UHPFRC’s present high cement content, high amount of ultra-fine components like silica fume and quartz flour which extend the particle size range and small size aggregates or sand. UHPFRC has a very low water cement ratio. It shows a self-levelling behaviour, but a high viscosity. This water to cement ratio is always smaller than the stoichiometric value, required for complete clinker hydration. Therefore in UHPFRC, a large part of the clinker grains (typically in the order of 50%) remains unhydrated [Richard & Cherezy 1995 ; Vernet et al 1998]. These remaining clinker particles can be considered as surface-reactive micro-aggregates of high elastic modulus, (approx. 120 GPa). They are strongly bonded by low C/S calcium silicate hydrate (CSH) and improve the mechanical performance of the material.

A large part of the high durability of UHPFRC is due to a significant reduction in pore size and volume. The main part of the porosity is found at nanometric scale, within the hydrates nano-structure. Water porosity and permeability arealso close to the detection limit, in relation with the absence of capillary porosity and the disconnected nanopores. These performances are one order of magnitude better than HPC and two orders of magnitude better than ordinary concrete. RPC resistance to corrosion was studied by the CSIC in Madrid (Spain) [Roux *et al* 1996]. After one year of drying and wetting cycles, the metallic fibers in Ductal® FM remain sound, even in a mechanically pre-cracked sample. Only surface stains, approximately at 0.1µm depth, are observed [Roux *et al* 1996].

In the range of Ductal® formulations, two main families can be distinguished. On one hand, thanks to an optimised steel fibre content and a very high performance matrix, a strain hardening behaviour is obtained with this materials under flexural loading leading to a very fine multiple cracking. This range of product could typically be used in structural application where steel fibres can take the function of reinforcing bars according to Interim Recommendation of UHPFRC [AFGC 2002]. On the other hand, the use of organic fibre and again a very high performance matrix allows to obtain a non brittle material for thin sections under flexure. This range of product can be used in architectural applications, where organic fibre can provide the structural integrity in case of cracking [Perry & Zakariasen 2003]. In this paper, the results of an experimental study are presented on the evolution of mechanical characteristics of pre-cracked specimens belonging to previously described families, subjected to various conservation conditions.

3 EXPERIMENTAL PROGRAM

The formulations of the concretes used in the present study are presented in Table 1. For the Ductal® FM, the samples are 70x70x280 mm prisms, notched in the middle for 10 mm, and subjected to 3 point flexural test according to the Interim Recommendations of UHPFRC [AFGC 2002]

For the FO formula, the samples are plates of 10 mm x 50 mm x 300 mm subjected to 4 point flexural test according to the NF EN 1170-5 standard, applicable to GRC.

	Sand	Cement	Filler	Silica Fume	Fibre	Fiber (%vol)	W/C ratio
Ductal® FM	1.43	1	0.3	0.325	Steel	2.0	0.20
Ductal® FO	1.43	1	0.24	0.30	APV	4.2	0.27

Table1. Relative formulations of the concretes

Fig. 1 gives an view of the samples preparation, showing the good fluidity and high viscosity of the mixtures, even with high fibre content. 24 Ductal® FM specimens and 9 Ductal® FO plates were cast.

Fig. 1 : Overview of the samples preparation

The experimental program includes three main steps. First, after maturity, all specimens were pre-cracked under flexural loading. Then the samples have been unloaded and kept in different aggressive conditions for three months. Finally, the samples have been re-loaded according to the same flexural test procedures as the ones used during pre-cracking, in order to determine the residual mechanical strengths. For each condition, three specimens have been tested.

For Ductal® FM, the maturation followed the following procedure : after casting the material, 48 hours at 20°C and 100%RH, then after demoulding 48 hours at 90°C and 100%RH. For Ductal® FO, a cure of 28 days at 20°C and 100%RH after demoulding was used. This pre-cracking has been done under flexure by adopting displacement control technique. For the Ductal® FM formula, two different crack opening levels have been tested, 100 and 300 µm, both directly measured at the notch location. 100 and 300 µm crack opening levels are considered as the limits in SLS in case of a non critical cracking and very critical cracking states respectively . For the Ductal® FO formula, a cumulative crack opening of 300 µm over a base length of 90 mm in the middle of the plate has been retained. This crack opening has been considered of critical for the structural integrity of a thin element, which looks extremely damaged at that state. Then, the various aggressive environments used are :

- **Reference samples :** they are kept at 23°C and 50% relative humidity.
- **Hot water :** the samples have been kept immersed into water at 60°C
- **Wet-dry cycles :** The tests have been conducted according to the EN 494 Standard, either in water at 20°C for 18 hours and then 6 hours of drying at 60°C (60 cycles in total).
- **Chemical environment :** Continuous immersion into a 10% sodium chloride solution. (this test has only been done for Ductal® FM).

In addition, during these tests, resonance frequency and mass of the samples have been regularly measured.

4 Results and discussion

4.1 Retention of mechanical properties

For each configuration, three samples were tested. The deviation between them is small and then, only one curve will be presented, representing the mean behaviour. Each curve displays flexural stress, calculated with the applied load by considering the linear classical mechanics formulas in function of crack opening displacement.

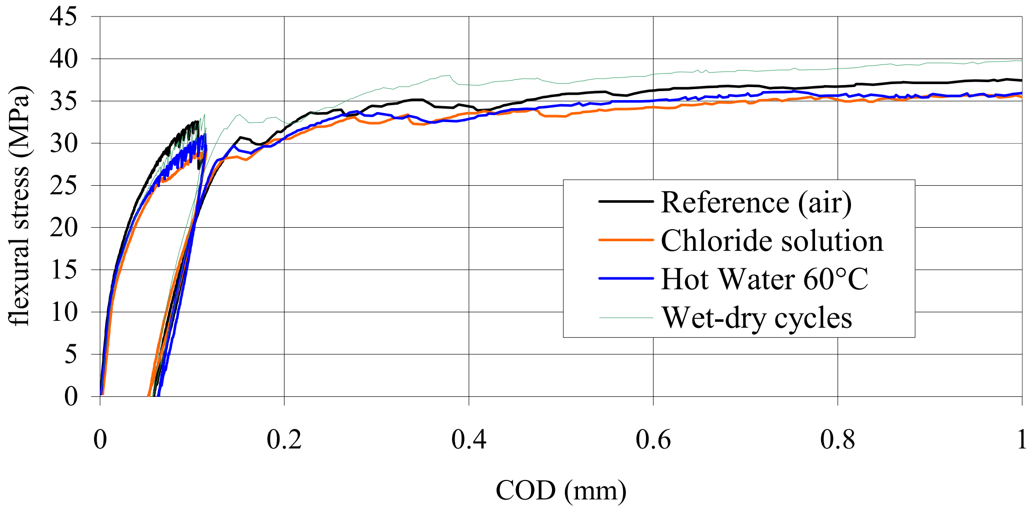

Fig.2 : Flexural Stress vs. COD curve for Ductal® FM, pre-cracking 100 µm

Figures 2 to 4 give the synthesis of the obtained results. The pre-cracking stage appears clearly as the first part of each curve and the re-loading corresponds to the second part. This two stage of loading have been connecting for the purpose of the presentation. In practice, the data acquisition was not set at the same rate in both loading stages, so that the noise on the curve could be different. However, the control rate of the test was kept constant at 60µm/min in both stages.

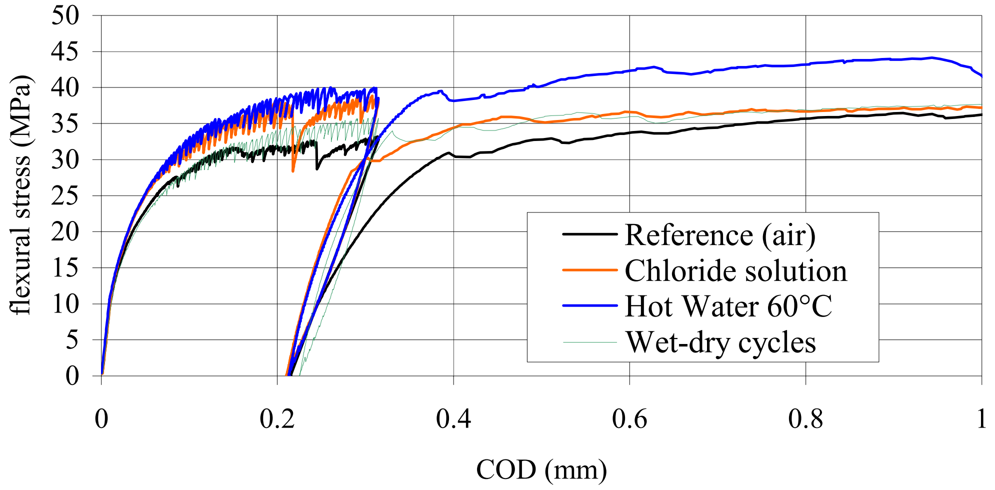

Fig.3 : Flexural Stress vs. COD curve for Ductal® FM, pre-cracking 300 µm

Then, it appears clearly that the mechanical characteristics of the cracked samples have not been influenced by the severe conditions of conservation used. In case of Ductal® FM (fig. 2 and 3) the continuity of all the curves is remarkable. The only point we can identify is the shape of the reloading curve for 300 µm pre-cracking in reference configuration. The slope of this curve is less stiff than in the other conservation cases. We can also notice that the reference configuration is the only one without contact of the cracked surface with water. Mechanisms of self sealing may be responsible of such behaviour, it will be discussed later in this paper.

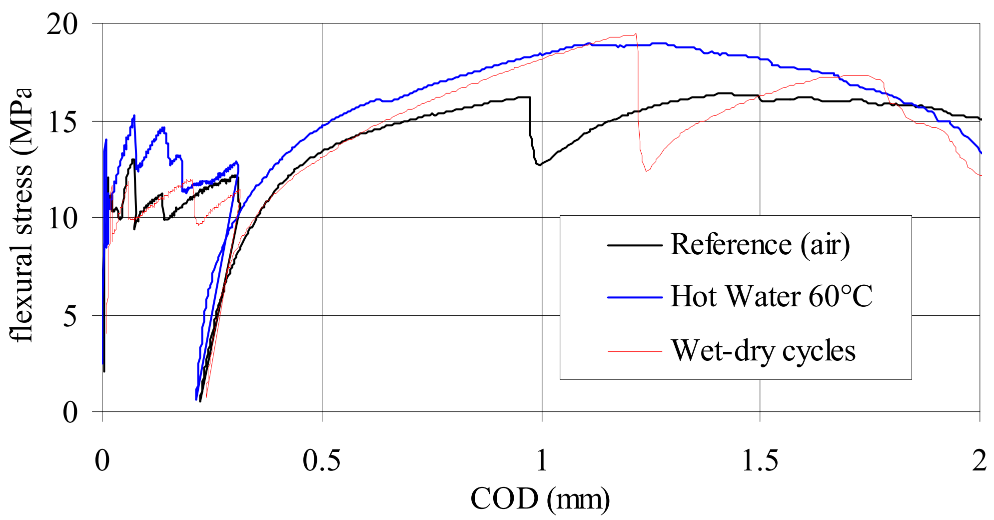

Fig.4 : Flexural Stress vs. COD curve for Ductal® FO, pre-cracking 300 µm

In the case of Ductal® FO, the same conclusion can be drawn. It is interesting to note that precise load drops can easily be identified on these curves (fig. 4). Each of these drops correspond to the localization of a new crack in the central part of the specimen under flexure. In addition, even after conservation in aggressive environment, the fibres bridge the cracks at a sufficient level so that a new crack is formed.

4.2 Resonance frequency and mass evolutions

Considering now the evolutions of resonance frequency and mass. For the reference environment, no notable evolution is measured on both measures. For the other environments, the evolution of the mass is still very small and it does not exceeded 0.5%. The reliability of the resonance frequency allows a more interesting analysis (fig. 5).

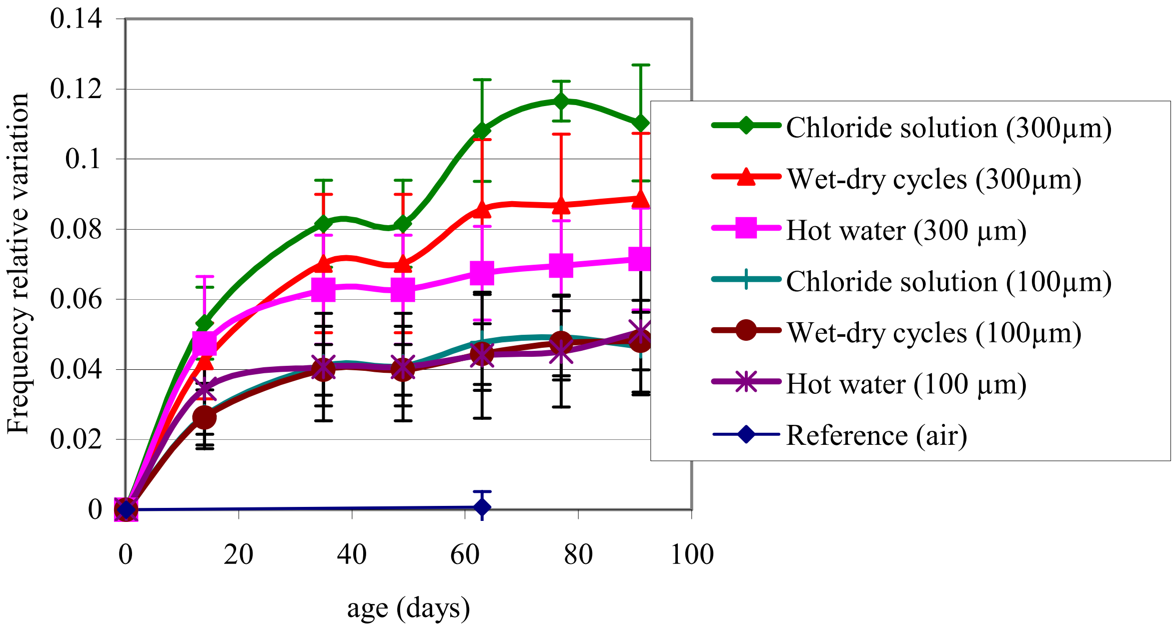

Fig.5 : Resonance Frequency vs. age for Ductal® FM

For Ductal® FM, it appears that the resonance frequency increases with duration of tests, showing a gradual stabilisation. In addition, this increase in higher for larger initial crack opening. It seams that any distinction between the various aggressive environment is not possible considering the standard deviation of the measure. However, it is well known that the resonance frequency is directly correlated with the dynamic elastic modulus of the material. Then, starting with a cracked sample, any increase in the resonance frequency indicates that the overall rigidity of the samples has increased.

One possible explanation of such phenomena is the self sealing of the microcracks. Indeed, we have already mentioned that a large part of the cement remains unhydrated in UHPC. Then, microcracks may open paths for the water which allows additional hydration of the cement. Such behaviour has already been observed in a previous study dealing with durability of the microstructure (fig. 6). Opening cracks of less than 40 µm were completely filled by hydration products after conservation under water [Ammouche & Martinet 2001].

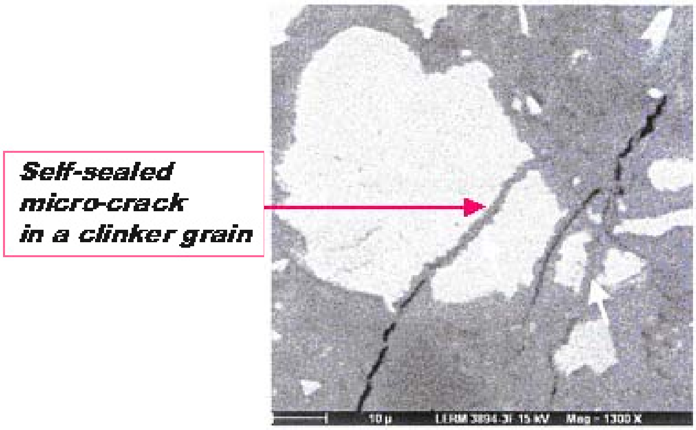

Fig. 6 : Self-sealing of microcracks by clinker hydration

As a consequence, the resonance frequency may be more sensitive in the case of 300µm crack opening than 100µm because the numbers of paths created during pre-cracking is larger. This is confirmed by the initial mean values of the resonance frequency (before conservation protocols), which are 3.46kHz (Sdt Dev 0.05kHZ) for Ductal® FM 100µm and 3.18kHZ (Sdt Dev 0.1kHZ) for Ductal® FM 300µm. Finally, such self sealing doesn't exist in case of conservation in air (the reference state). This may explain the shape of the reference curve compared to the others on fig. 3 and the previous consideration explaining why such effect is not visible on fig. 2.

Finally, this hypothesis on self sealing should be confirmed in the present study by additional works on the tested specimens, using SEM.

5 Conclusion

In this paper, the results of an experimental study on the evolution of mechanical characteristics of pre-cracked Ductal® specimens based on steel fibre reinforced and organic fibre reinforced formulas are presented. Various conservation conditions have been tested, including hot water, wet-dry cycles and sodium chloride solution.
After three months, it appears that the mechanical characteristics of the cracked samples have not been influenced by the severe conditions of conservation used. Then this retention of the mechanical properties shows that such material may be used in structural applications as far as the crack opening is less than 300µm. Possible self sealing of the microcracks has been proposed considering the evolution of resonance frequency. This point should be confirmed with further observations using SEM of the specimens' microstructure.

6 References

AFGC.2002, "*Ultra High Performance Fibre-Reinforced Concretes, Interim Recommendations*", AFGC Publication, France, January 2002

Ammouche, A. & Martinet, G. 2001 "*Durability of Ductal® and ancient BPR*" Rapport d'étude n° 98.3894.003.01.A, LERM, August 2001

Casanova, P., Dugat, J., & Orange, G. 2000 "*Ductal : a new generation of Ultra-High Performance Fiber Reinforced Concrete*" Proc. HPC Symp., Workability, Strength and Durability, Hong Kong, December 10-15, 2000. Vol. 2, pp. 853-859, C.K.Y. Leung, Zongjin Li, Jian-Dong Ding Editors.

Perry, V. & Zakariasen, D. 2003 "*An Overview of UHPC (Ductal®) Technology, Materials, Properties, Markets and Manufacturing*" PCI / FHWA 3rd International Symposium on HPC Orlando, Florida, 19-22 Oct. 2003

Richard, P. & Cherezy, M. 1995 "*Les bétons de poudres réactives*" Annales de l'ITBTP, N°532, Mars-Avril 1995, pp. 83-102.

Roux, N., Andrade, C., & Sanjuan, M.A 1996 "*Experimental Study of Durability of Reactive Powder Concretes*" Journal of Materials in Civil Engineering, Feb. 1996, pp. 1-6.

Vernet, C. 2003 "*UHPC Microstructure and related Durability Performances : Laboratory Assessment and Field Experience Examples*" PCI / FHWA 3rd International Symposium on HPC, Orlando, Florida, 19-22 Oct. 2003

Vernet, C., Lukasik, J., & Prat, E. 1998 "*Nanostructure, permeability, and diffusivity of Ultra-High Performance Concretes (UHPC)*". International Symposium on High Performance and Reactive Powder Concretes, Sherbrooke, Québec, Vol.3, pp.7-36.

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Durability vs. Reliability of RC Structures

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ABSTRACT

The failure criteria of Serviceability Limit States are linked to design service life. Moreover, different levels of reliability should be adopted for different types of Limit States. The choice of level of reliability for a particular structure and material should take account of the relevant factors, including possible consequences of failure and potential costs of safety measures. This principle is not fully accommodated in current codes.

In the context of durability of reinforced concrete structures, the European codes *EN 206-1* and *EN1992-1-1* together with *EN1990* provide recommendations for different exposition and structural classes as to the minimum content of cement, maximum water/cement ratio, and, optionally, minimum concrete strength class. Limiting concrete cover is required simultaneously. The relevance of these requirements and associated values of the index of reliability should be ensured. By utilizing analytical models for carbonation progress in concrete and taking account of the uncertainties of concrete mixture, carbon dioxide concentration and other input data, a statistical description of carbonation progress is gained. Random or deterministic input parameters are involved. Statistical analysis is performed allowing for calculation of the reliability index relevant to different service lifes. The time passing before the initiation of rebar corrosion is considered as the limiting condition.

The results of the investigation of the reliability index profiles and its trends concerning different conditions/classes are provided utilizing an interactive web-site covering service life, concrete cover and reliability index assessment. The non-uniformity of reliability level is shown.

Some conclusions (and generated questions) are discussed.

KEYWORDS

Concrete structures, durability, carbonation, reliability index, Eurocodes

1 INTRODUCTION

Design for durability is coming considerably into the focus of researchers and recently, designers too. This has been clearly demonstrated at several international and/or local conferences, and by numerous papers (let us mention e.g. [Siemes 2002]). ISO activity (TC98) is currently following this trend by working to produce a new document, ISO/NP 13823 "General principles in the design of structures for durability". Also, *Integrated Design* and its subsiduary, *Performance-Based Design* (PBD), are leading trends in structural engineering design; these approaches deal with *durability* and *reliability* issues, which rank amongst the most decisive structural performance characteristics.

Unfortunately, the prescriptive approach of current standards (Eurocodes) does not allow simply for design focused on specific service life and/or specific level of reliability. Such tasks necessarily require the utilization of stochastic approaches, analytical models and also simulation techniques. The theoretical apparatus of this approach has been developed already but the practising engineer is usually not equipped with the appropriate knowledge, routines and instruments or software. In order to partially solve this problem the authors have recently introduced [Teplý *et al.* 2004] a simple tool for the designing process for concrete structures taking account of the considerations of durability and reliability – thus attempting to furnish the designer with a user friendly instrument for dealing with such problems. It is an interactive web page called *RC_LifeTime,* freely accessible on http://rc-lifetime.stm.fce.vutbr.cz/.

The goal of the present paper is to assess the feasibility of modern codes of practice to design reinforced concrete structures for durability and also to demonstrate the features of the *RC_LifeTime* web page.

2 DESIGN FOR DURABILITY OF RC STRUCTURES

2.1 Eurocodes

The service life of a building or structure is determined by its design, construction, ageing and maintenance during use. The combined effect of structural performance and ageing should be considered.

The modern codes (Eurocodes) generally do not allow for a design subjected to a specific (target) service life. Some exceptions concern RC structures: combining tables 2.1 with tables 4.3N and 4.4N [EN1992-1-1] the value of nominal concrete cover can be determined with relevance to structural class (S1-S6) and exposition class (e.g. XC1–XC4 in cases of concrete carbonation). The indicative strength class is given too. Moreover, considering tables 1 and F.1 [EN206-1] the requirements of maximum water-cement ratio and minimum content of cement (together with minimum concrete strength class as an additional specification) are recommended in accordance with exposition classess referring to the use of CEM I and an intended working life of 50 years only! Immediately some questions appear: the direct dependance of strength class on water and cement content is not a unique one, and, the relation of concrete durability to strength is not generally to be taken as granted. The increasing proportion of cement may lead to concretes being more crack-prone. Also the type and the grain size of used cement, supplementary cementing materials and aggregatte may have a strong influence – see e.g. [Mehta 1997]. It should be noted also that the above-mentioned structural recommendations of EN1992-1-1 and EN206-1 do not take into account the indicative design working life categories – see the basic structural Eurocode EN1990, table 2.1 and the reliability requirement!

Structural design based on modern building codes deals with limit states (both ultimate and serviceability – ULS and SLS). The level of reliability in the context of durability is not stated there. When considering the degradation of reinforced concrete structures, the corrosion of reinforcement is the dominating effect. In the context of corrosion the following limit states can be recognized: (i) depassivation of reinforcement due to carbonation or chloride ion penetration; (ii) cracking; (iii) spalling of concrete cover; (iv) decrease in the effective reinforcement area (leading to excessive deformation and finally to collapse). Types (i)-(ii) are in the SLS category of limit states, (iii) might fall in both the ULS and SLS categories depending on the location and grade of degradation, and, (iv) is in the ULS (load bearing capacity) or SLS (deformation capacity) categories. As stated in the basic design code [EN1990], the recommended value of the reliability index for SLS (irreversible state) is β = 1.5, which is again relevant to a 50-year service life.

2.2 *RC_LifeTime*

Rather often, case (i), i.e. the depassivation of reinforcing steel due to carbonation (pH ≤ 9.5), is considered conservatively as a *limiting condition.* This condition is considered in *RC_LifeTime* as: "*the time to depassivation = initiation period → irreversible serviceability limit state*". In other words, at this stage the reinforcement is no longer secure against corrosion. Under the condition of the presence of a certain level of moisture in concrete (necessary for cathodic reaction), hydroxide ions form corrosion products.

RC_LifeTime performs statistical analyses and offers the following options:

(i) "Service Life Assessment" provides an evaluation of service life based on the equality of carbonation depth and concrete cover

$$x_c = c \tag{1}$$

The assessment of x_c is based optionally on one of the two models shown in [Papadakis *et al.* 1992 – (2a,b)]; both models are enhanced within *RC_LifeTime* by the humidity function extracted from [Matoušek 1977] – see also [Keršner *et al.* 2004]. The input data are the values of concrete cover c together with 12 model variables (or 5 – according to the chosen model; optionally deterministic or random variables). The statistical characteristics of relevant service life (mean and standard deviation) are the output data. The target value of the reliability index β may be the additional input value – describing the probability of reaching condition (1) – and then the corresponding service life is the output value.

(ii) "Concrete Cover Assessment" provides an evaluation of concrete cover appropriate to equality (1). The input data are the value of the target service life (as a deterministic value) together with model variables (again, deterministic or random). Statistical characteristics of relevant concrete cover (mean and standard deviation) are output data. The value of required concrete cover c may be inputted too and the relevant reliability index β is then an output value (describing the reliability of reinforcement depassivation).

It has to be noted: (a) both models are capable of describing the carbonation of concrete made from CEM I only, while other models are the focus of ongoing work; (b) the values of β should depend also on the consequences of failure and on the relative cost of safe design; (c) the condition (1) is assessed for the structure/member "in general", not considering the issue of a critical cross-section and the degree of statical redundancy; (d) within *RC_LifeTime* the reliability index is of the "Cornell´s" type.

In the view of comments (b) and (c) the reliability level given by β = 1.5 seems to be too "severe" as it is relevant for limit states described by e.g. excessive deflections or excessive crack width as well. This is why ISO 2394, table E.2, shows SLS values from β = 0 (for reversible) up to β = 1.5 (for irreversible states).

3 RELIABILITY STUDY

Utilizing *RC_LifeTime* and following the recommendations of the Eurocodes described above, parametric studies have been performed assessing the reliability index values for exposition clases and structural classes. The corresponding input data and statistical characteristics are listed in tables 1 and 2. Comments to Tab. 1: the mean values of cover are shown, the two-parametric lognormal probability density function is assumed (origin in zero point) with COV = 20%. In this case "slab geometry" COV

= 5% only. Table 2: the mean values of cement content are the minima and water-cement ratios the maxima recommended by EN 206-1; all probability distributions are normal.

Exposure class	*Structural class* S1	S2	S3	S4	S5	S6
XC1	20	20	20	25	30	35
XC2/XC3	20	25	30	35	40	45
XC4	25	30	35	40	45	50

Table 1. Nominal concrete cover in mm recommended by EN1992-1-1 for exposure and structural classes.

Exposure class	*Cement content* [kg/m^3] *Mean value*	*Standard deviation*	*Water/ Cement ratio* [–]	*Water* [kg/m^3] *Standard deviation*	*Relative Humidity* [%] *Mean value*	*Standard deviation*
XC1	260	7	0.65	4	55	10
XC2	280	8	0.60	4	85	5
XC3	280	8	0.55	4	75	10
XC4	300	9	0.50	3	65	10

Table 2. Random input variables

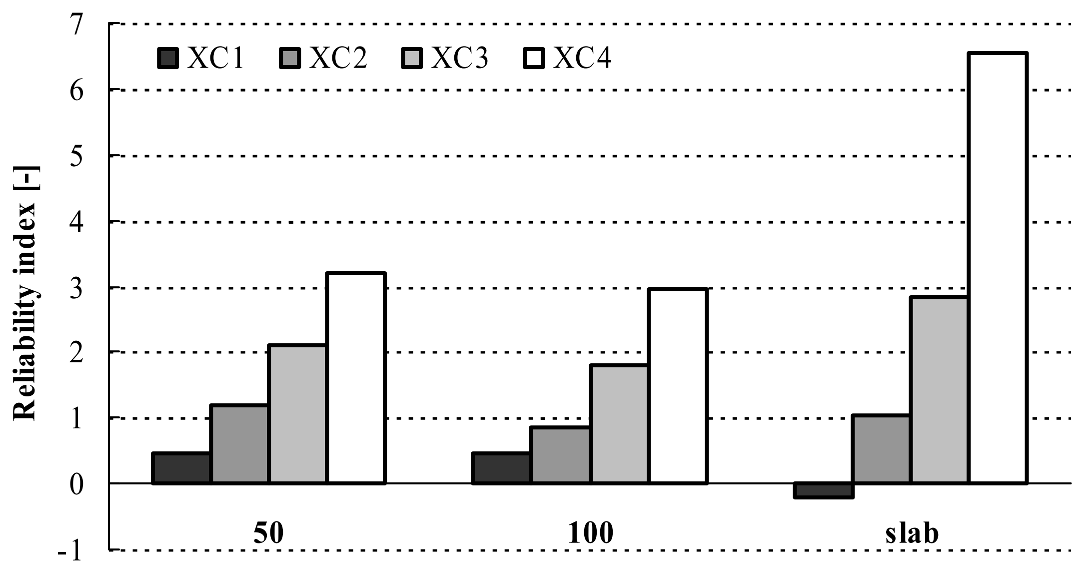

Figure 1. Reliability index for exposure classes

Fig. 1 depicts the reliability index for exposure classes and three cases with reference to EN1992-1-1: case “50” is the basic situation, i.e. the structural class S4, with a design working life of 50 years; “100” has a design working life of 100 years; “slab” is relevant to members with slab geometry (the position of reinforcement is not affected by the construction process or members where special quality control of concrete has been ensured – see table 4.3N of EN1992-1-1). Fig. 2 shows reliability profiles together with the reliability level of β = 1.5 demanded by EN 1990. All these results clearly document the remarkable non-uniformity of the reliability level “hidden” in the Eurocodes.

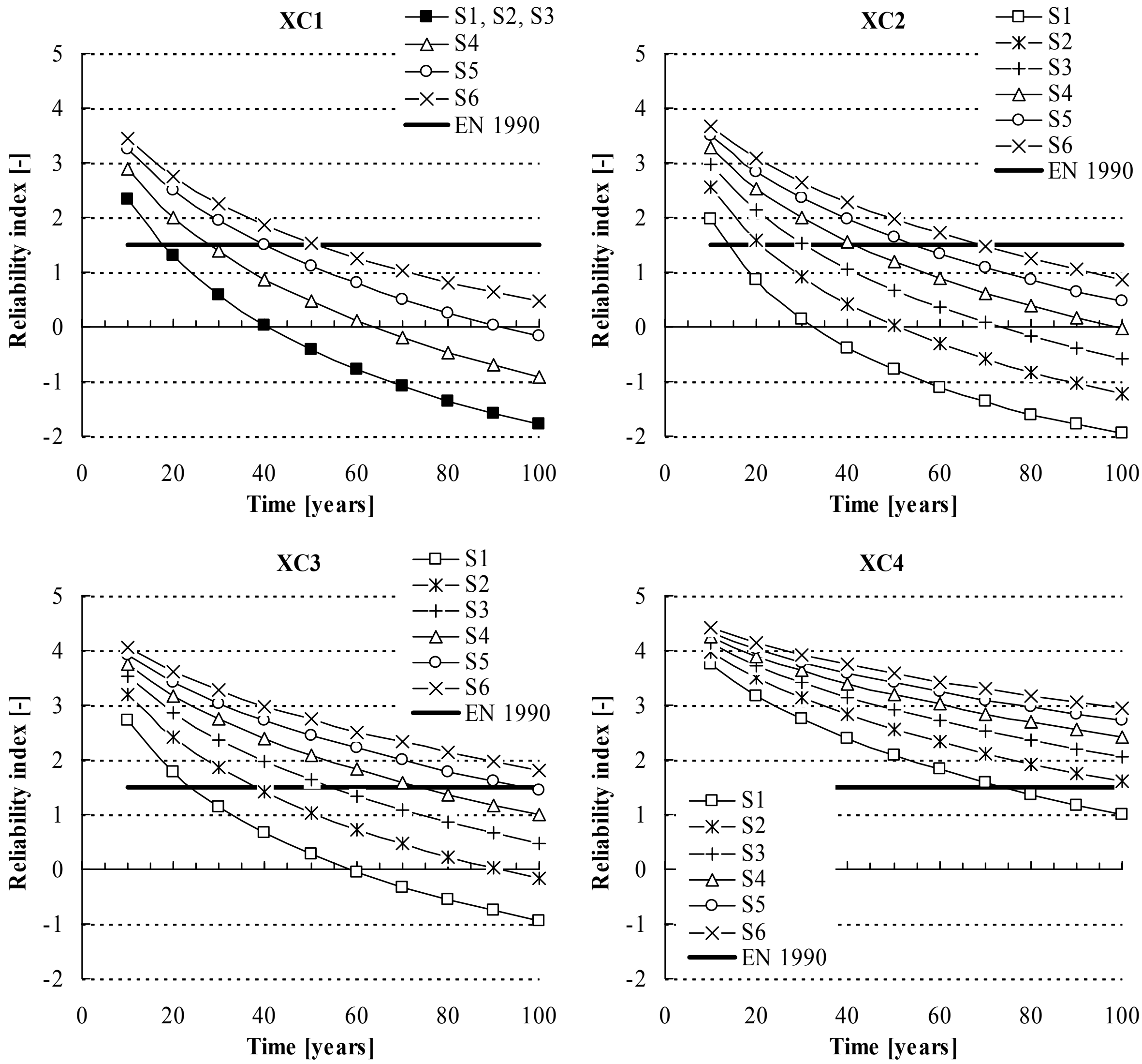

Figure 2. Reliability index vs. service life

4 CONCLUSIONS – QUESTIONS

Carbonation is a particularly important form of deterioration and its impact on existing or newly designed structures is evident. Its vast variability due to variability in concrete quality and environmental changes is remarkable and should be taken into account. It could be assessed by *RC_LifeTime* which serves as a simple tool for the Performance-Based approach of reinforced concrete structure design. However, it is based on a most conservative limit state.

The authors believe that apart from the verification shown in [Keršner *et al.* 2004], some additional research concerning the current model and especially other models for concretes from blended cements is necessary.

The results of the approach and the study shown above generate some questions:

$?_1$ …. how should relevant durability limit states of RC structures be defined?

$?_2$ …. what should the appropriate values of the reliability index associated with different durability limit states of RC structures be?

$?_3$ …. is it necessary to make required concrete cover thickness and required service life compatible with a certain balanced level of reliability?

Definitely, these questions require further study and discussion. The authors also believe that design for durability (and for a specific target service life) needs a probabilistic approach and appropriate/individual reliability assessments.

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6 REFERENCES

EN 1990:2002 Basis of Structural Design (European Standard)

EN 206-1:2000 Concrete – Part 1: Specification, performance, production and conformity (European Standard)

EN 1992-1-1:2003 (E) Eurocode 2: Design of concrete structures – Part 1.1: General rules and rules for buildings

Gehlen, Ch. 2000, 'Probabilistishe Lebensdauerbemessung von Stahlbeton bauwerken', *Deutsher Ausschuss fuer Stahlbeton*, Heft 510, Berlin.

Keršner, Z., Rovnaníková, P., Teplý, B. & Novák, D. 2004, 'Design for durability: An interactive tool for RC structures', Proc. Life cycle assessment, behaviour and properties of concrete structures – LC 2004, Brno, Czech Republic, 9 11 November 2004, (in print).

Maage, M. & Smeplass, S. 2001, 'Carbonation – A probabilistic approach to derive provisions for EN 206-1' *DuraNet workshop*, Tromso, Norway, June 2001.

Matoušek, M. 1977 *Effects of some environmental factors on structures*. PhD thesis, Technical University of Brno, Brno (in Czech).

Mehta, P.K. 'Durability – Critical Issues for the Future', *Concrete International*, July 1997, 27-33.

Papadakis, V. G., Fardis, M. N. & Vayenas, C. G. 1992, 'Effect of Composition, Environmental Factors and Cement-lime Mortar Coating on Concrete Carbonation', *Materials and Structures*, 1992, 25[149], 293–304.

Siemes, T. 2002, 'Overview Of The Service Life And Maintenance Problem Probabilistic Design', Proc. 9DBMC (CD ROM), Brisbane, paper No. 260, 10 pages.

Teplý, B., Králová, H. & Stewart, M. G. 2003, 'Ambient Carbon Dioxide, Carbonation and Deterioration of RC Structures', *International Journal of Materials & Structural Reliability,* 1, 31–36.

Teplý, B., Rovnaník, P., Keršner, Z. & Rovnaníková P. 2004, 'Support to durability design of RC structures', *Beton TKS*, 3/2004, 38–40 (in Czech).

Risk Based Maintenance Strategy for Coastal Concrete Structures

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ABSTRACT

Corrosion of reinforcing steel in concrete is the dominant cause of premature failures of reinforced concrete structures located in chloride-laden environments, leading to ultimate structural collapse. It is also observed that some severely deteriorated concrete structures survive for many years without maintenance. This raises the question of why and how to maintain corrosion affected concrete structures, in particular in the climate of the increasing scarcity of resources. The present paper attempts to formulate a maintenance strategy based on risk cost optimization of the structure during its whole service life. A time-dependent reliability method is employed to determine the risk of attaining each phase of the service life. To facilitate practical application of the formulated maintenance strategy, an algorithm is developed and programmed in a user-friendly manner with a worked example. A merit of the proposed maintenance strategy is that models used in risk assessment for corrosion affected concrete structures are related to some of design criteria used by practitioners. It is found in the paper that there exist an optimal number of maintenances for cracking and delamination that returns the minimum total cost for the structure of its whole life. The maintenance strategy presented in the paper can help structural engineers, operators and asset managers develop a cost-effective management scheme for corrosion affected coastal concrete structures.

KEYWORDS

Concrete structures, Maintenance, Reinforcement corrosion, Risk, Total cost.

1 INTRODUCTION

Corrosion of reinforcing steel bars [rebar] in concrete is the dominant cause of premature failures of reinforced concrete [RC] structures located in chloride-laden environments [ACI C222 1985, Broomfield 1997]. The corrosion induced structural deterioration is a gradual process, consisting of a few phases during the service life of RC structures. These include corrosion initiation; concrete cracking; delamination and the final rupture of the structure [member] due to loss of strength. Much of the maintenance for such structures is related to these phases of service life. With the increasing scarcity of resources, any maintenance of deteriorated structures needs to be evaluated cost-effectively, taking into account economic constraints. One solution could be to prolong the overall service life of corrosion affected RC structures by extending the time period of each phase through intermediate maintenances, provided that their safety is not compromised. A risk-cost optimized approach can achieve this.

The intention of this paper is to formulate a maintenance strategy based on risk cost optimization of the structure during its whole service life. A performance-based model is proposed to determine each phase of service life of corrosion affected RC structures. A time-dependent reliability method is employed to determine the risk of attaining the limit state of each phase. To facilitate practical application of the formulated maintenance strategy, an algorithm is developed. An example is given to illustrate the application of the proposed maintenance strategy to RC seawalls. A merit of the proposed maintenance strategy is that models used in risk assessment for each phase of service life are directly related to design criteria used by practitioners. The maintenance strategy presented in the paper can help structural engineers, operators and asset managers make decisions with regard to repairs, strengthening and/or rehabilitation of corrosion affected RC structures.

2 OPTIMISATION FORMULATION

In research project conducted at the University of Dundee, a quantitative risk-based maintenance strategy for coastal concrete structure is adopted. The goal is to maintain the safety margin of the structure above a prescribed acceptable level throughout the service life of the structure, while intermediate maintenance is carried out to meet the serviceability requirements of the structure. Types of maintenance will be determined by types of structural performance [response], including [i] superficial patching for concrete cracking, [ii] major repair for concrete delamination and [iii] overall structural strengthening for rupture [or end of service life]. The attainment of a limit state at each phase, i.e., cracking, delamination or rupture, is quantified by a probability p_c, $p_{\rm d}$ or p_r, respectively. Clearly only when p_c or p_d is greater than an acceptable limit respectively will the corresponding maintenance be warranted to achieve an cost effectiveness. Similarly, p_r has to be smaller than an acceptable limit to eliminate undue risk of collapse. With these constraints, the time and number of maintenances can be determined through conventional optimization in terms of a total cost, C_T. Mathematically this can be expressed as

Minimizing $$C_T(t_L) = \sum_{i=1}^{n_{mc}} C_c(t_c^i)\cdot p_c(t_c^i) + \sum_{i=1}^{n_{md}} C_d(t_d^i)\cdot p_d(t_d^i) + C_r(t_L)\cdot p_r(t_L)$$

Subject to $$p_c(t_c^i) \ge p_{c,a}, p_d(t_d^i) \ge p_{d,a}, p_r(t_L) \le p_{r,a} \qquad [1]$$

where C_c and C_d are maintenance costs of cracking and delamination and C_r is the cost due to structural rupture [collapse]. All costs are relative to the initial construction cost of the structure so that the data on costs are relatively easy to collect. The design variables in this optimization are t_c^i, t_d^i, t_L, n_{mc} and n_{md}. The variables t_c^i and t_d^i are the time of maintenances due to cracking and delamination respectively; t_L is the time of strengthening or end of service life; n_{mc} and n_{md} are the number of maintenances due to cracking and delamination respectively at different times and locations of a structure. For simplicity, interdependence between cracking and delamination is not included in

Equation [1] to achieve effective practical applications as will be shown in the example, but it can be dealt with in principle using conditional probability [Melchers 1999]. In Equation [1], the probability terms with subscripts *a* are acceptable limits. The outputs of the optimization will form the basis of a maintenance strategy for a structure. That is to determine when [t_c^i or t_d^i] and where [n_{mc} or n_{md}] and what types of maintenance [cracking or delamination] are necessary during the service life of the structure and the associated confidence of achieving each phase of the service life [p_c, p_d and p_r].

3 RISK ASSESSMENT

3.1 Corrosion induced concrete cracking

According to design codes and standards, e.g., ACI 318 [1999] and BS 8110 [1997], the performance criterion related to concrete cracking of practical RC structures is to limit the crack width to an acceptable level. In the theory of structural reliability, this criterion is expressed in the form of a limit state function as follows

$$G(w_{cr}, w, t) = w_{cr}(t) - w(t) \tag{2}$$

where $w(t)$ is the crack width [load effect] at the surface of concrete cover at time t and $w_{cr}(t)$ is a critical limit for the crack width. With this limit state function, the probability of structural failure due to corrosion induced concrete cracking, $p_c(t)$, can be determined by using time-dependent reliability method [Melchers 1999]. An analytical solution for upcrossing rate problem with a deterministic threshold has been derived by Li and Melchers [1993], therefore $p_c(t)$ can be determined by

$$p_c(t) = \int_0^t \frac{\sigma_{\dot{w}|w}(t)}{\sigma_w(t)} \phi\left(\frac{w_{cr} - \mu_w(t)}{\sigma_w(t)}\right) \left\{ \phi\left(-\frac{\mu_{\dot{w}|w}(t)}{\sigma_{\dot{w}|w}(t)}\right) + \frac{\mu_{\dot{w}|w}(t)}{\sigma_{\dot{w}|w}(t)} \Phi\left(\frac{\mu_{\dot{w}|w}(t)}{\sigma_{\dot{w}|w}(t)}\right) \right\} d\tau \tag{3}$$

where $\phi(\ )$ and $\Phi(\ )$ are the standard normal density function and the distribution function, and " | " denotes the conditional probability. When the stochastic model of crack width is established, all the variables in Equation [3] can be determined as shown in Li and Melchers [1993]. The crack width $w(t)$ can be expressed in terms of primary contributing factors, which are treated as basic random variables. The statistics of $w(t)$ can be obtained using the technique of Monte Carlo simulation. By introducing a random variable ξ_w in such a way that its mean is unity, i.e., $E(\xi_w) = 1$, the crack width $w(t)$ can be modeled as follows

$$w(t) = w_c(t) \cdot \xi_w \tag{4}$$

where $w_c(t)$ is treated as a pure time function of crack width. Li *et al.* [2003] developed an analytical solution to corrosion induced crack width w_c, based on fracture mechanics and the model of thick-wall cylinder as shown in Fig. 1 [Bažant 1979; Pantazopoulou and Papoulia 2001]

$$w_c(t) = \frac{4\pi d_s(t)}{(1-\upsilon_c)(a/b)^{\sqrt{\alpha}} + (1+\upsilon_c)(b/a)^{\sqrt{\alpha}}} - \frac{2\pi b f_t}{E_{ef}} \tag{5}$$

where a and b are the inner and outer radii of the thick-wall cylinder [Fig. 1], υ_c is Poisson's ratio of concrete, E_{ef} is the effective elastic modulus of concrete and f_t is the tensile strength. The key variables in Equation [5] are the thickness of the ring of corrosion products d_s, which is directly related to the corrosion rate i_{corr} as shown in Liu and Weyers [1998], and the stiffness reduction factor α, which is related to stress conditions and concrete property and geometry as shown in Li *et al.* [2003].

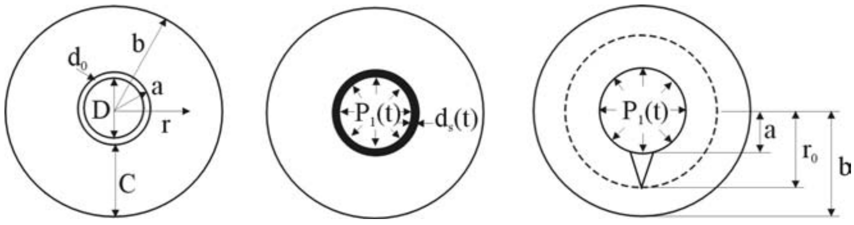

Figure 1. Schematic representation of cracking process.

In Equation [5] the mean and auto-covariance functions for $w(t)$ are [see, e.g., Li and Melchers 1993]

$$\mu_w(t) = E[w(t)] = w_c(t).E[\xi_w] = w_c(t) \quad [6]$$

$$C_{ww}(t_i,t_j) = \lambda_w^2 \rho_w w_c(t_i) w_c(t_j) \quad [7]$$

where ρ_w is [auto-]correlation coefficient for $w(t)$ between two points in time t_i and t_j and λ_w is coefficient of variation for ξ_w. Both of can be estimated using the technique of Monte Carlo simulation. With μ_w and $C_{ww}(t_i,t_j)$ and the assumption of Gaussian process, the other statistical parameters of $w(t)$ can be determine [Li and Melchers 1993].

3.2 Corrosion induced concrete delamination

The derivation of the probability of structural failure due to corrosion induced concrete delamination is similar to that for cracking failure once a model for delamination is established. Concrete delaminates from the structure when the corrosion induced cracks in the concrete propagate and the neighboring cracks join together to form a fracture plane that runs through the rebars. Once the fracture plane is formed, the concrete cover separates from the substrate concrete and falls off. When the following geometric condition for concrete section is met [Bažant 1979; Li *et al.* 2004], the cracks would propagate between the rebars and form a fracture plane parallel to the surface of concrete cover

$$S - D < 2C \quad [8]$$

where S is the spacing between rebars, D is the diameter of the rebar and C is the depth of concrete cover. Otherwise, the inclined fracture plane would be formed. This Section only concerns with the delamination from a fracture plane parallel to the surface of concrete cover. For inclined fracture plane, formulae in Bažant [1979] will be used.

For the concrete cover to separate from the substrate concrete, certain crack opening, i.e., crack width, at the fracture plane is required. When the crack width at any point along the crack $w(r,t)$ is larger than a critical limit w_d the layer of concrete [i.e., cover] will delaminate at the fracture plane. Since the corrosion induced crack is tapered from the rebar [see Fig. 1 [c]] it is sufficient that when the crack width at the point that two cracks join together, i.e., $r = b = S/2$, is larger than a critical limit the concrete cover delaminates. The limit state function for this condition is

$$G(w_d, w, t) = w_d(t) - w(t) \quad [9]$$

With this limit state function, the probability of structural failure due to corrosion induced concrete delamination can be determined. The rest is the same as that for cracking failure and hence will not be repeated here.

3.3 Corrosion induced structural rupture

Eventually, the rebar corrosion in concrete will reduce the strength of RC structures and leads to the rupture at the critical cross-section of a structural member. The limit state function for structural rupture can be expressed as

$$G(R, R_a, t) = R(t) - R_a(t) \quad [10]$$

where $R(t)$ is the residual strength at time t and R_a is a minimum acceptable strength. Equation [10] represents a typical downcrossing problem in reliability calculation. Mathematically it has approved [Melchers 1999] that the formulation and solution of a downcrossing problem are exactly the same as those of upcrossing, when the problem is looked at "upside-down". Therefore, $p_r(t)$ can be determined once the model of $R(t)$ is established. In analogy to the model for crack width of Equation [4], the residual strength $R(t)$ can be modeled as

$$R(t) = R_s(t) \cdot \xi_R \quad [11]$$

where $R_s(t)$ is the residual strength at the critical cross-section of a structural member, ξ_R is a random variable with $E(\xi_R) = 1$ and λ_R. The residual sectional strength $R_s(t)$ can be expressed, in terms of the net area of rebar $A_{net}(t)$, as

$$R_s(\mathbf{E}, t) = \psi \cdot f[A_{net}(\mathbf{E}, t)] \quad [12]$$

where $f[\ ]$ is provided by standard concrete design codes [e.g., ACI 318, BS 8110]. The use of code formula can facilitate the easy and direct application of the developed model by practitioners. In Equation [12], ψ is a coefficient to be determined from calibration of the code formula $f[\ ]$ against data from direct loading test [Li 2003]; and **E** is a vector of factors affecting cross-sectional area reduction of the rebar. The most significant factor is the corrosion rate i_{corr}. The statistics of $R(t)$ can be determined in a similar way as that of crack width.

4 WORKED EXAMPLE

To illustrate the application of the proposed maintenance strategy and in particular the developed algorithm to practical structures, a RC seawall is used as an example. As can be seen from the models of structural response in each phase of service, a key parameter in the models is the corrosion current density i_{corr}. In fact, it is an essential parameter for the assessment of corrosion induced structural deterioration. The measurement of it, however, is very site or structure specific and its accuracy affects the assessment to a great deal. A comprehensive test on a large scale RC seawalls has been undertaken from which the i_{corr} can be used. This is the primary reason to use a RC seawall as an example here although the data on i_{corr} is still preliminary [Table 1]. The wall has a dimension of 1000 [wide] x 2000 [high] x 150 [thick] mm. It is subjected to simulated seawater spray under simultaneous service load of 60% nominal strength. Other variables of the geometry and material properties of the wall are also shown in Table 1. With these inputs, the optimized time and number of cracking and delamination maintenances and the service life can be determined using the developed computer programme [more details are referred to Li *et al.* 2004]. The results in Fig. 2 demonstrate that the total cost is a function of the number of crackings and delaminations and there exists a minimum total cost for a given accepted risk. This can be the vindication of the formulation of Equation [1]. The results are shown in Fig. 3 [window image]. As can be seen, repairs for cracking for different rebars [location] and at different times are clearly marked. Same is for delamination. From Fig. 4, it is of interest to note that the cost of structural rupture affects the optimal number of cracking and delamination maintenances. Higher structural rupture cost leads to higher minimum total cost when the number of cracking and delamination maintenances is small. However, when the number of maintenances is large, the effect of structural rupture cost on the minimum total cost diminishes as shown in Fig. 4. Clearly the information in Fig. 3 can well equip coastal engineers and asset managers with a rational and practical maintenance strategy for coastal infrastructure subject to corrosion attacks and thereby provide a cost-effective asset management scheme.

Basic variables	*Symbol*	*Mean*	*COV*
Concrete cover	C	30 mm	0.2
Diameter of rebar	D	10 mm	0.15
Thickness of pore band	d_0	12.5 µm	-
Effective modulus of concrete	E_{ef}	30.12 GPa	0.12
Elastic modulus of steel	E_s	200 GPa	-
Compressive strength of concrete	f_c	30 MPa	0.15
Tensile strength of concrete	f_t	3.0 MPa	0.2
Yield strength of steel	f_y	543 MPa	0.15
Corrosion current density	i_{corr}	0.0652t+1.0105	0.2
Rebar spacing	s	186 mm	-
Poisson's ratio of concrete	υ_c	0.18	-
Relative cost for cracking repair	$C_c(0)$	10%	-
Relative cost for delamination repair	$C_d(0)$	20%	-
Relative cost due to rupture	$C_r(0)$	10	-

Table 1. Values of basic variables

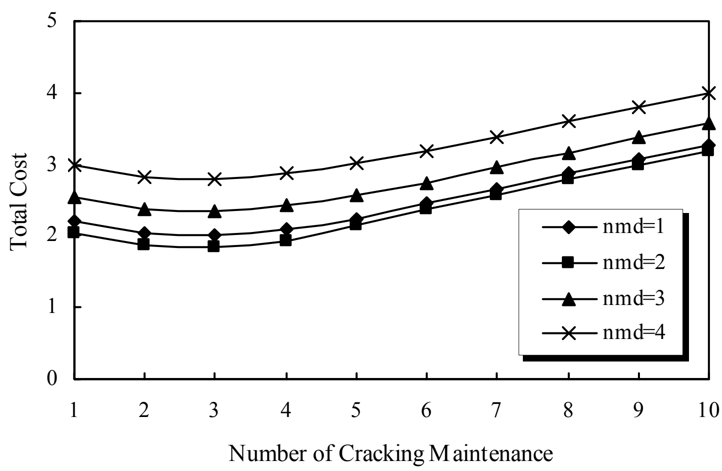

Figure 2. Total cost vs. number of cracking and delamination maintenance.

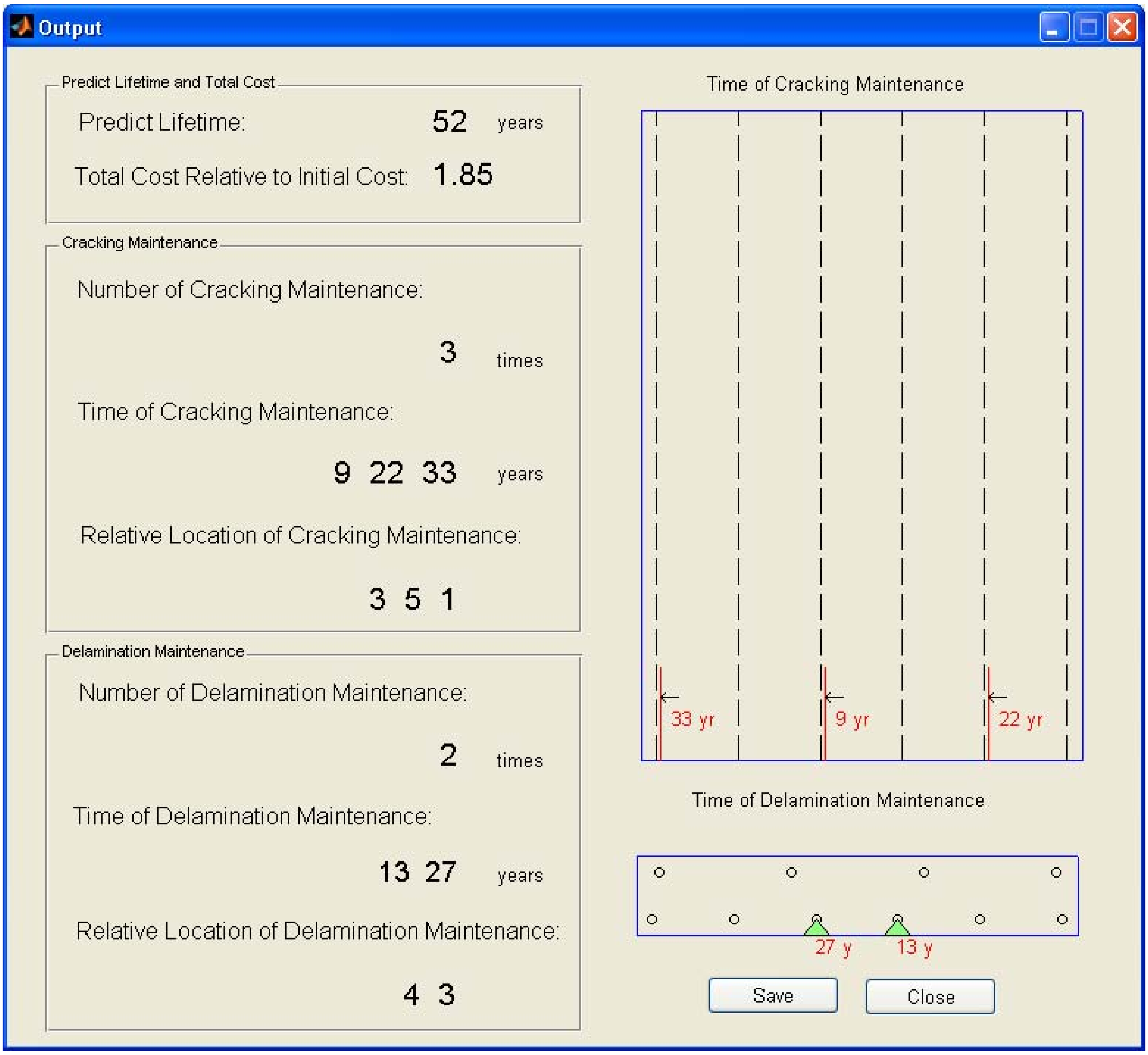

Figure 3. Outputs of optimization with optimal time and location of cracking and delamination [window image].

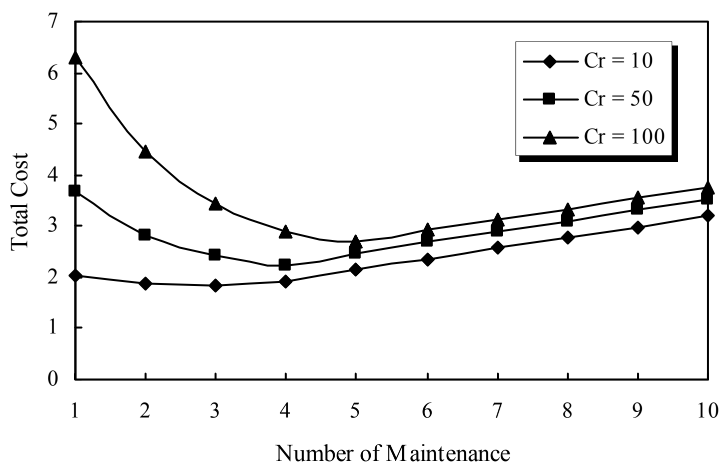

Figure 4. Effect of cost due to structural rupture to minimum total cost.

5 CONCLUSIONS

A risk cost optimized maintenance strategy for corrosion affected concrete structures has been formulated based on whole life behavior of the structure. A performance-based model for service life prediction has also been proposed. A time-dependent reliability method is employed in the paper to determine the risk of attaining the limit state of each phase of service life. To facilitate practical application of the formulated maintenance strategy, an algorithm has been developed and programmed in a user-friendly manner and on window base. The proposed maintenance strategy has the advantage that models used in risk assessment for each phase of service life are related to some of design criteria used by practitioners and that multiple limit states for the structure have been considered in the risk-cost optimization. It has been found in the paper that there exist an optimal number of maintenances for cracking and delamination that returns the minimum total cost for the structure of its whole life. It has also been found that the cost of structural rupture affects the optimal number of cracking and delamination maintenances. It can be concluded that the proposed maintenance strategy can help structural engineers, operators and asset managers make decisions with regard to repairs, strengthening and/or rehabilitation of corrosion affected concrete structures.

6 ACKNOWLEDGEMENTS

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7 REFERENCES

ACI Committee 222. 1985, 'Corrosion of metals in concrete', *ACI Journal,* **82**[1]**,** 3–32.

ACI 318. 1999, *Building Code Requirements for Structural Concrete with Commentary*, American Concrete Institute.

Bažant, Z.P. 1979, 'Physical model for steel corrosion in concrete sea structures – application', *Journal of Structural Division, ASCE*, **105**[ST6], 1155–1166.

Broomfield, J. 1997, *Corrosion of Steel in Concrete, Understanding, Investigating & Repair*, E & FN Spon, London.

BS 8110. 1997, *Structural Use of Concrete – Part 1: Code of Practice for Design and Construction*, British Standards Institution.

Li, C.Q. 2003, 'Life cycle modeling of corrosion affected concrete structures – propagation', *Journal of Structural Engineering*, *ASCE*, **129**[6], 753–561.

Li, C.Q. & Melchers, R.E. 1993, 'Outcrossings from convex polyhedrons for nonstationary gaussian processes', *J. Engrg. Mech., ASCE*, **119**[11], 2354–2361.

Li, C.Q., Zheng, J.J. & Lawanwisut, W. 2004, 'Concrete delamination caused by steel reinforcement corrosion', *ACI Structural Journal* (submitted).

Li, C.Q., Zheng, J.J., Lawanwisut, W. & Chen, Z.J. 2003, 'An analytical model for corrosion induced crack width in reinforced concrete structures', *ACI Structural Journal* (submitted).

Li, C.Q., Lawanwisut, W. & Mackie R.I. 2004, 'A risk cost optimized maintenance strategy for corrosion affected concrete structures', *Journal of Computer-Aided Civil and Infrastructure Engineering (CACAIE)*, (submitted).

Liu, Y. & Weyers, R.E. 1998, 'Modeling the time-to-corrosion cracking in chloride contaminated reinforced concrete structures', *ACI Materials Journal*, **95**[6], 675–681.

Melchers, R.E. 1999, *Structural Reliability Analysis and Prediction*, John Wiley and Sons, Chichester.

Pantazopoulou, S.J. & Papoulia, K.D. 2001, 'Modeling cover-cracking due to reinforcement corrosion in rc structures', *Journal of Engineering Mechanics*, *ASCE,* **127**[4], 342–351.

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Sustainable tools and methods for estimating building materials and components service life.

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ABSTRACT

The need of building materials and components service life assessment is strongly increasing in the actual building construction sector. This phenomenon is due to new techniques developments and new environmental requirement and sustainable development principles. The behaviour of material performance within time in several environments has been largely studied, and good models contribute to a reliable evaluation of service life of materials when environmental and external solicitations are well defined.
On the other hand, evaluation of components service life is not so well known. It can be explained by the complexity of the components and the lack of knowledge on the failure scenarios of these components. Furthermore, the feedback management is not efficient, and the knowledge is not enough collected and accessible.
Our main interest is the development of tools and methods for service life assessment which remain perfectly suitable to the possible evolution of research and standardisation on the numerical characterisation of the different factors impacting the components service life.
Our research and development activity is focused on three complementary tools to answer this task: (Failure Mode Effect and Criticality Analysis method; Product service life data base (including material service life data base) and associated tools compatible with ISO 15686; Data fusion).
The purpose of this paper is the presentation of the efficiency and the complementarities of these tools and methods. Indeed, by the use of these tools, we are performing a multi scale analysis of building product and component (at the scale of the product itself (data base, data fusion and F.M.E.A.), at the scale of the component (data base, data fusion and F.M.E.A.) and of the material (data base, data fusion). Further more, data base and data fusion are much more efficient at the material scale, especially if the material is subjected to one identified load. The F.M.E.A. allows us to evaluate the service life of the product from the service life values of the different materials loaded by one unique identified solicitation. We can thus use this result of service life at the scale of the product to feed the product service life data base.
This study highlight the need of performing F.M.E.A. analysis to obtain reference service life without performing long, complicated and onerous tests at the scale of the product, and to collect these data into a data base which take into account all the information accompanying the service life value. A project of F.M.E.A. software is on his way in order to facilitate the analysis.

KEYWORDS

Service life assessment, F.M.E.A., ISO 15686, data fusion

1 INTRODUCTION

The aim of this article is to propose a methodology to estimate the service life of products and components by the use of several specific tools developed by the CSTB and suitable with ISO 15686 standards application. Indeed, the development of the ISO 15686 standard is generating new approach for estimating the service life of product and component. We will first shortly introduce this method. This approach is highlighting the lack of building products and components service life data. The second paragraph will describe the interest of using F.M.E.A. to collect reference service life data (RSL).
Finally, we will present the tools developed in CSTB, and a general methodology which is using these tools to estimate service life data of building products in a specific case study. This methodology intends to be an operational tool for construction stakeholders, (Designers, managers, manufacturers…) compatible with recommendation of ISO 15686. Furthermore, according to the fact that knowledge in term of characterisation and quantification of factors impacting the service life is limited, this methodology try to propose a sustainable solution, which take into account the evolution of knowledge in this domain.

2 "FACTOR METHOD": ISO 15 686

2.1 Presentation of the method

The factor method is a simple system to estimate service life when there is limited knowledge of long-term performance of components. (ISO 15686-1, ISO 15686-2 and ISO DIS 15686-8)
This method is using the following equation to estimate service life of building product and component:

$$ESLC = RSLC \times FactorA \times FactorB \times FactorC \times FactorD \times FactorE \times FactorF \times FactorG \quad (1)$$

Where:

RSLC is the reference service life of component
ESLC is the estimated service life of component

The estimated service life of a component (ESLC) is a function of the reference service life (RSLC) and a number of factors:

A: Material/Component factor
B: Design factor
C: Workmanship factor
D: Internal environment factor
E: External environment factor
F: In-use factor
G: Maintenance factor

This formula acts as a reminder of what should be taken into account when estimating service life.

2.2 Reference Service life

Our capability to Collect RSL is the key issue for the performance of the factor method. A step further in the factor method is the calibration of the influence of the factors on the ESL.
At this stage the factor are supposed to be representative of the difference between the reference in use condition set and the condition of the case study. According to the fact that we can use several RSL for the same product, it is not possible to propose a unique quantified scale for each factor according to associated conditions. This highlight one of the interest of the notion of intrinsic reference service life (IRSL) describe in paragraph 4.2.

3 INTEREST OF MULTISCALE ANALYSIS TO COLLECT RSL

The behaviour of material performance within time under one identified sollicitation has been largely studied, and good models contribute to a reliable evaluation of service life of materials when environmental and external solicitations are well defined. This evaluation became more complex with the coupling of the external and environmental solicitations. Therefore there is still a strong need of experimentation in order to improve the models for complex and coupled cases study.
If the elaboration of models seems to be possible at the scale of materials, it is not as simple at the product scale. The evaluation of products and components service life is a difficult task, due to the complexity of the components and the lack of knowledge on the failure scenarios of these components.

Concerning experimentation:

> Taking into account that products have several functions, constitutive materials, and are submitted to complex environment, we do believe that realisation of accelerated ageing difficult is tricky. The size of the sample is also an obstacle for laboratory testing. The number of constitutive materials of the product create problem to accelerate ageing without changing any ageing mechanism

Concerning modelling:

> Numerical modelling of product ageing assumes to be able at least to perform thermo-hydro-mechanical modelling of heterogeneous components. It also imply to model interaction between materials.

Furthermore, the feedback management is not efficient, and the knowledge is not enough collected and accessible.

According to this context, we have developed a tool which allows us to get service life information at the scale of the building product by using the one at the scale of the material. This tool is based on F.M.E.A. (Failure Mode and Effect Analysis) (Talon [2004], Lair [2002]). Using first structural and functional analysis and then a Failure mode and effect analysis, this analysis provides us all the potential failure scenarios of the product in use in its environment. Failure event graph is then drawn for a better exploitation of the result. (Figure 1). By the use of this graph we can define for each scenario, several stage of degradation implying a material under a single solicitation.

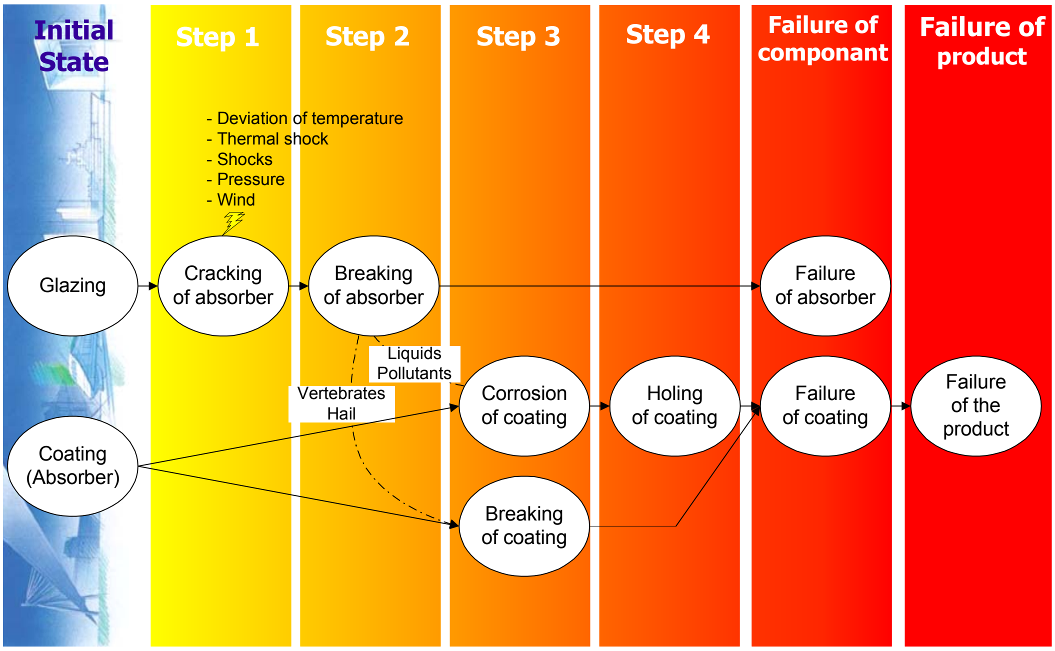

Figure 1: Failure event graph used to present result of F.M.E.A. Illustration with two components (Glazing and Coating) of a solar panel.

The determination of the kinetic of each of these degradations allows us to calculate the service life of each scenario. (Figure 2 and equation 2).

$$SL[Scénario(i)] = \sum_{j=1}^{n_i} [SL_{mat}(mat(j), perf(j))] \qquad (2)$$

Where:

- i is the number of the studied scenario
- n_i: successively degraded materials in scenario number i (stages of the scenario)

Considering the service life of each scenario, it is possible to obtain a service life of the product. Indeed, the characterisation of the criticality of each scenario allows us to select representative scenarios from the initial exhaustive list. Then, the determination of duration of these scenarios gather a service life data for the performance associated to the scenario (equation3):

$$SL[product, perf(j)] = \min(SL[Scenario_{selected}(i(perf(j)))]) \qquad (3)$$

If we consider the need for characterisation of durability and\or service life of the product. The task is largely simplified by the used of F.M.E.A.. Indeed, the challenge was to gather Service life data for complex products (several materials, complex geometry and environments, several possible uses and needs), it is now to use data of degradation kinetic of a material under one identified solicitation to provide the service life of this material under this solicitation according to the associated performance. The figure 2 illustrate how to get a service life value for a product in a specific case study, by staying at the scale of the product (factor method, ISO 15686), and by the use of F.M.E.A. which is using the information at the scale of the material.

It can be a suggestion to evaluate service life if the "factor method" is not applicable by lack of RSL data or knowledge on the factors.

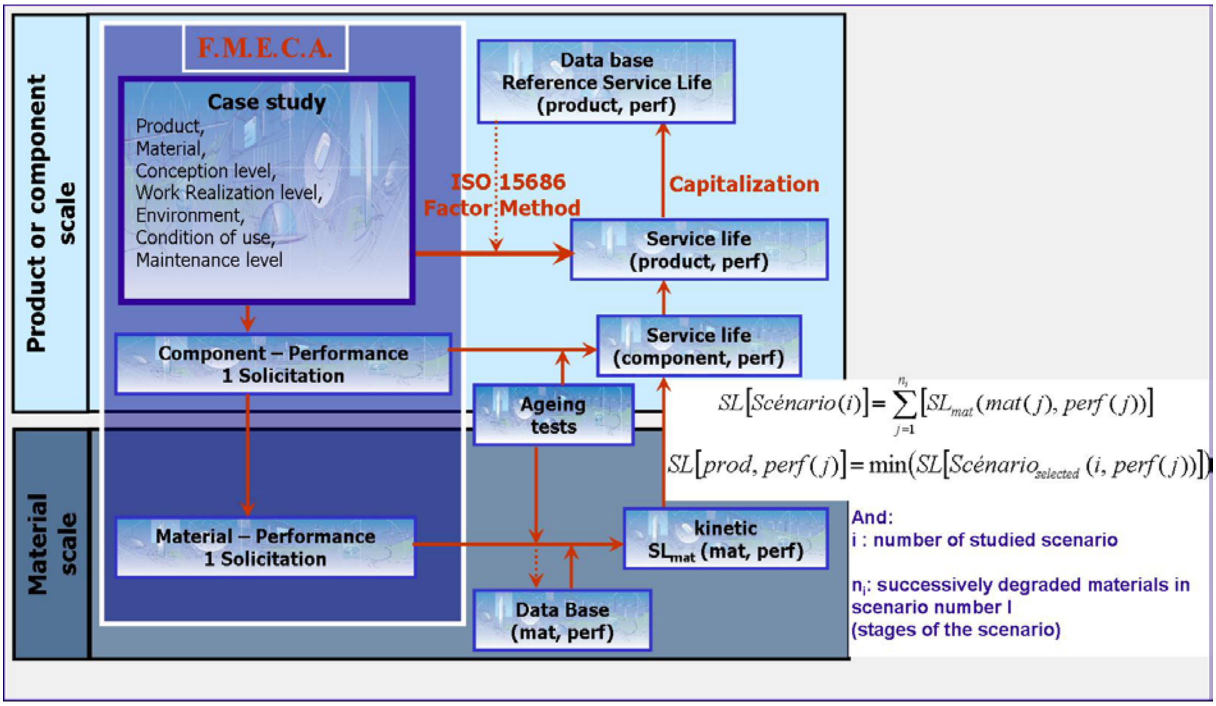

Figure 2: Methodology of service life estimation at both product and material scale. ISO 15 686 propose a direct methodology at the scale of the product (the "factor method") using the notion of RSL. The F.M.E.A. is using the service life data at the scale of the material to provide ESL which can be capitalize as a RSL for other case study.

4 PROPOSED TOOLS AND METHODOLOGIES TO ESTIMATE SERVICE LIFE OF PRODUCTS AND COMPONENTS

4.1 Reference Service Life Data base

The ISO 15 686 part 8 "Reference Service Life" provides guidance on the provision of reference service life (RSL) for use in the application of the "factor method". Methodology such as F.M.E.A. is recognised as a method to provide RSL data by the ISO 15686 standard. We have developed a data base which is able to collect RSL and its associated information (Source, factors A to G, data quality, references…). This SQL data base is accessible on the CSTB intranet, but could be proposed soon on a website. Some of the specificities of this data base are the following:

- It includes fields for Numerical Unit Spread Assessment Pedigree (N.U.S.A.P.) in order to perform data fusion (Lair et al [2001])
- It includes fields to collect information on A to G factors according to the data providers knowledge, but also includes a second data base which proposes a factors format for the product (according to the state of the art, or standardization on that product), and the initial information on the factors can be updated according to these formats. Initial information is kept and the proposed format can be updated too.

4.2 Tools to optimize the factor method

4.2.1 Suggestion of Intrinsic Reference Service Life (IRSL) notion

We suggest to use the notion of Intrinsic Reference Service Life (I.R.S.L.).The idea is to artificially use the same "intrinsic reference in use conditions" for all the IRSL value. This is applicable if we are able to quantify the impact of the factors on the service life. When Collecting the RSL corresponding to the reference in use conditions, we back calculate an IRSL, corresponding to the following equation

4, where A_R, B_R,C_R, D_R, E_R, F_R and G_R are the factor corresponding to the application of the factor method when replacing RSL by IRSL and ESL by RSL.

$$IRSL = \frac{RSL}{A_R \times B_R \times C_R \times D_R \times E_R \times F_R \times G_R} \tag{4}$$

Making the assumption that the factors can be quantified, each value of collected RSL is then able to provide an IRSL value.

4.2.2 Interest

The interest of using the IRSL value is the efficiency of the data fusion tool (Lair et al [2001]) on all the calculate IRSL to get a single CIRSL on each product (Consolidated Intrinsic Reference Service Life)

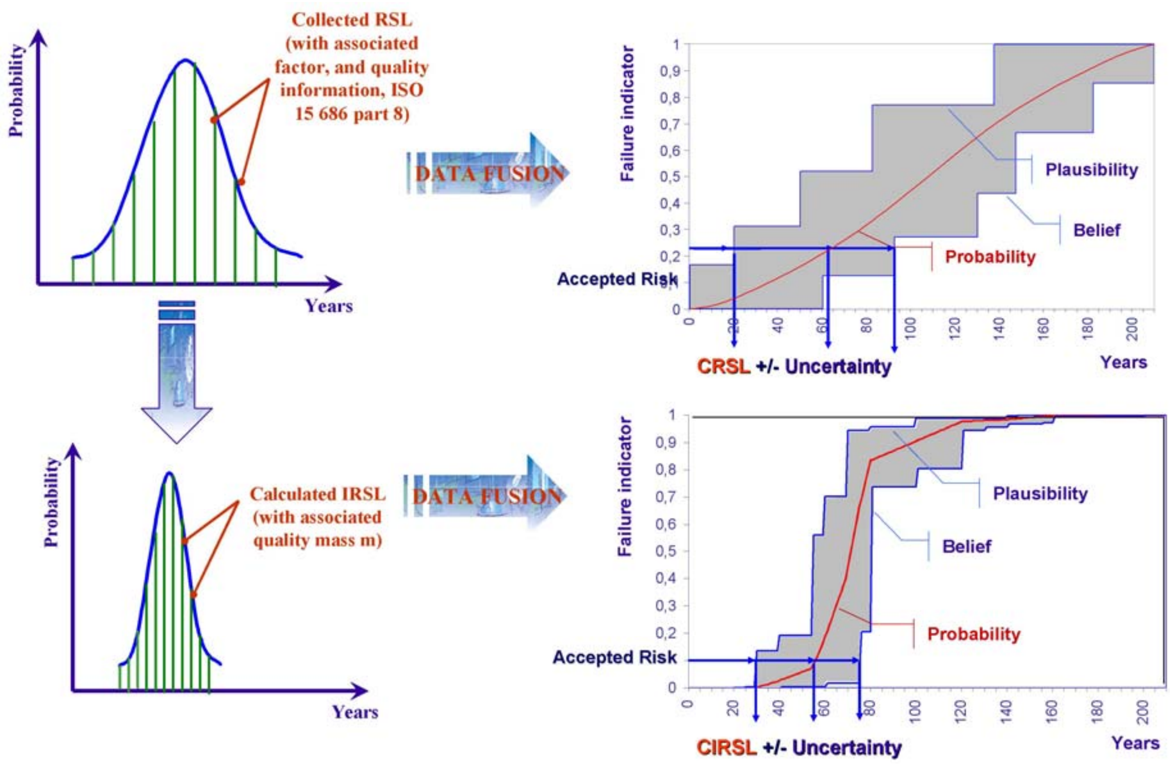

Figure 3: Theoretical illustration of the interest of IRSL and data fusion.
This sketch highlights the interest of the fusion on a IRSL set of data in comparison with a classic RSL set of data. The failure indicator curve is more exploitable with the IRSL set; indeed we observe that the failure indicator curve slope of the IRSL is nearly constant when the one of CIRSL is largely bigger between 60 and 80 years. The consensual value of a service life is thus easier to extract.

To each service life, we assign a mass m (which is similar to a probability assignment) according to the quality of the basic durability data. Thus *m* is a quality indicator according to the origin of the data. Theoretical structures, data input, set-up conditions… are quality criteria (Funtowicz & Ravetz, [1990]) valued on a five level scale.

The concept of IRSL and CIRSL is compatible with the RSL data recommendations of ISO 15686-8.

4.3 General methodology for service life estimation

The needs for estimating service life of building components and products can have several origins such as whole life costing analysis, life cycle analysis, Performance Based Building (PeBBu Network), PFI….
The tools developed (RSL data base, Data fusion and the analysis based on F.M.E.A. and associated tools) are complementary and supply a real scientific gain for the evaluation of service life by the use of the factor method.

The classical case study is the estimation of a product service life used in a constructed assets or a building where we precise information on the in use conditions as well as on material quality and conception level, protective coating etc…
According to these data and to the specific functional requirements due to building context (Sustainable building, Whole life costing, Life cycle analysis…) we are able to characterize the functions of the product by leading Functional analysis or F.M.E.C.A. (which is integrating a functional analysis).

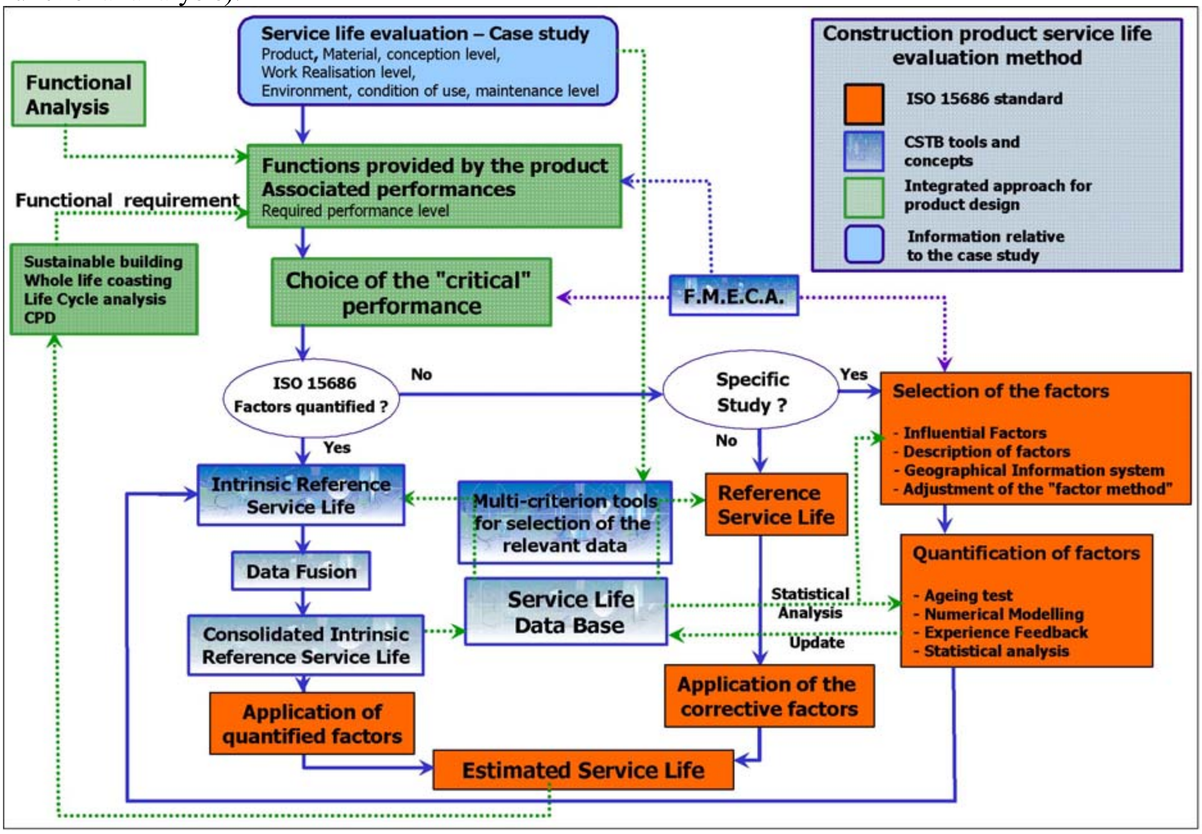

Figure 4: General methodology for service life estimation of a specific case study.

To estimate a service life of this product it is necessary to identify the "critical" function and his associate "critical" performance". Indeed, we are estimating the service life of a product for one given function the performance of which is the reason of the failure. At this stage we can start the process of service life estimation according to the "factor method".

This first way is corresponding to the case where the factors are not quantified and where no specific study is or has been led. One can see the interest of a reference service life data base and selection tool to choose the appropriate reference service life for the case study. Indeed, the "factor method" efficiency is strongly link to the ability of supplying a reference service life with reference in use conditions as close as possible to the one of the case study; in that case the corrective factor are less influential and the uncertainty on estimated service life value is smaller.
But there is an interest to perform specific study on the product to determine and quantify the value of the factor impacting on the service life. The aim is to obtain a scale of every factor which is based on the intrinsic reference in use conditions.

If such analysis is done, we had the opportunity to calculate intrinsic reference service life (IRSL) and to realize a more efficient data fusion in order to obtain a consolidated intrinsic reference service life (CIRSL).
The CIRSL can be reused for any case study concerning this product and provides a calculation of the estimated service life without using subjective corrective factor.
However this theoretical methodology needs to be validated by practical application

5 PERSPECTIVES AND CONCLUSION

Providing RSL data is the first step for service life planning. We are focusing our research in the use of data at the scale of the material to estimate service life of component or product. The F.M.E.A. is a very powerful tool to realize this task. Indeed, it explains the behaviour of products in their environment, by giving the detail of each step of each failure scenario. The scenario is then scattered in several well defined degradation.

6 GLOSSARY

ESLC: Estimated Service Life of Component
RSLC: Reference Service Life of Component
IRSL: Intrinsic Reference Service Life
RSL: Reference Service Life
CIRSL: Consolidated Intrinsic Reference Service Life
FMEA: Failure Mode Effect Analysis
FMECA: Failure Mode Effect and Criticality Analysis

7 BIBLIOGRAPHY

ISO 15686-1,2000, Building and constructed asset - Service life planning – Part 1: General principle
ISO 15686-2,2001, Building and constructed asset – service life planning – Part 2: service life prediction procedures
ISO DIS 15686 -8, Building and constructed asset – service life planning – Part 8: Reference service life

Talon, A. Boissier, D. Chevalier, J.-L. Hans, J. 2004, "A methodological and graphical decision tool for evaluating building component failure", Proc. CIB World Building Congress 2004, Toronto, Canada, 2 7 may 2004.

Lair, J., Chevalier J.L. (2002). "Failure Mode Effect and Criticality Analysis for risk analysis (design) and maintenance planning (exploitation)". 9th Durability of Building Materials and Components (9th DBMC) Brisbane, Australie – 17/21 Mars 2002.

Lair, J., Chevalier J.L., Rilling J. (2001). "Operational methods for implementing durability in service life planning framework. CIB World Building Congress". WELLINGTON, (April 2001).

Funtowicz, S. O. and Ravetz J.R. (1990). "Uncertainty and Quality in Science for Policy". Dordrecht (The Netherlands). Ed.: Kluwer Academic Publishers. 229 pages.

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The Factor Method – a simple tool to service life estimation.

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ABSTRACT

Over the last decade, there have been extensive research and development activities in many countries regarding service life prediction (SLP) methods for building materials, products and components. Internationally, the outcome of many of these efforts have been reported and applied within two bodies, the working commission CIB W80/RILEM 175-SLM "Service life methodologies" and the standardisation technical committee ISO/TC 59/SC 14 "Design life".

Since 1999, the commission CIB W80/RILEM 175-SLM has been working on three state-of-the-art reports describing three levels of SLP methods. These are probabilistic based methods, that are based on probability and statistical data, simpler engineering methods based on a reference service life and probability distribution functions of some key factor values, and the simple factor method that is based on the same reference life and simple deterministic values of key factors. The Factor Method is presented in the International Standard ISO 15686 Part 1. Increasing application of the Factor Method over the last years has shown that there is an urgent need for input data, both for the reference service life as well as rules for selection of the factor values. It has also shown that there is a need for the Factor Method in its original version as a tool for simple service life estimates and to demonstrate what are the main conditions influencing the service life.

This paper explains the Factor Method and refers to some of the work going on internationally regarding the development and evaluation of the method. Further, the paper presents requirements of design lives of buildings, building components and services, and explains the meaning of each of the factors involved. The main part of the paper is to give some general examples of how to select appropriate values for each of the factors of the Factor Method. Some tables are given to illustrate a range of values in the interval 0.2 to 5, and under what circumstances different values may be chosen. Tables show general conditions for selection of factor values, as well as basis for a more specific selection of values for the factors A (quality of components) and E (outdoor environment), respectively. Finally, the paper stresses the need for presentation of reference service life of building materials and components, and for further development of the system of factor value selection as mentioned above.

KEYWORDS

Service life estimation, Factor Method, factor values.

1. INTRODUCTION

Over the last decade, there have been extensive research and development activities in many countries regarding service life prediction (SLP) methods for building materials, products and components. Internationally, the outcome of many of these efforts have been reported and applied within two bodies, the Working Commission CIB W80/RILEM 175-SLM "Service Life Methodologies" and the standardisation technical committee ISO/TC 59/SC 14 "Design Life". This is further described in clause 2.3. The scope of this paper is to present a tool for choice of practical factor values as input to the Factor Method.

2. STATUS OF THE FACTOR METHOD

2.1 Definitions of service life

For the discussion of service life in this paper, the following definitions apply [ISO 2000]:

Design life: intended service life, expected service life or service life intended by the designer.

Reference service life: service life that a building or parts of a building would expect (or is predicted to have) in a certain set (reference set) of in-use conditions.

Estimated service life: service life that a building or parts of a building would be expected to have in a set of specific in-use conditions, calculated by adjusting the reference in-use conditions in terms of materials, design, environment, use and maintenance.

2.2 Presentation of the Factor Method

Since the publication of ISO 15686 Part 1 in 2000, the Factor Method has gained increasing interest, and it has been studied in many research and development projects. The method is becoming more and more familiar to people being involved in studies of durability and service life of building materials and components, and the number of publications presenting and discussing the method has been increasing. However, as a basis for the further discussion, the method is also presented here.

An Estimated service life (ESL) of a material or component is calculated based on a Reference service life (RSL) and a series of factors, A-G, by combining them as shown in equation (1).

$$ESL = RSL \times A \times B \times C \times D \times E \times F \times G \qquad (1)$$

where A = quality of components
B = design level
C = work execution level
D = indoor environment
E = outdoor environment
F = in-use conditions
G = maintenance level

2.3 Development and application of the Factor Method

The Factor Method in its present form was first published in the international standard ISO 15686 Part 1 in 2000. The method is based on a similar method published in Japan some decades ago [Architectural Institute 1993]. However, the Japanese method was some more sophisticated in the way that the type, number and application of the various factors were altered dependent on the material or component to be evaluated, or on the specific application of the actual material or component. This may give more correct results for each situation, but it will also require more information about various materials or components and the actual situation where they are to be applied.

ISO 15686 Part 1 gives a comprehensive presentation of service life prediction (SLP). In ISO 15686 Part 2 [ISO 2001], a procedure for SLP was presented based on alternative studies of climate exposure and degradation of materials and components. This procedure is regarded to be the most extensive and correct procedure to determine the Reference service life (RSL) of a material or component. The RSL is the basic value for application of the Factor Method, together with specific values of the individual factors included.

In 2004, CIB W80/RILEM 175-SLM published a state-of-the art report describing two alternative versions of the Factor Method [Hovde & Moser 2004]. The first part of the report (Part A) presented the background, development and evaluation of the Factor Method in its original version, whereas the second part of the report (Part B) presented a more specific selection of factor values based on statistical distributions of each of the factors. The scope of the latter part was to show how the Factor Method could be applied as an engineering design method for SLP of building materials and components. A brief summary of the state-of-the-art report may be found in separate papers, [Hovde 2002], [Moser & Edvardsen 2002], [Lacasse & Sjöström 2004].

Further, papers discussing the Factor Method may be found in journals and conference proceedings, showing that there is an increasing interest in the method and need for practical tools for SLP. Examples of such papers are [Teply et.al. 2003], [Cusamo & Lucchini 2003], [Re Cecconi 2003], [De Pascale 2003], [Re Cecconi 2004]. Marteinsson [2003] has performed a more extensive study of SLP and the role of the Factor Method in such work. He presents some examples of practical application of the method, and he states that practical solutions of SLP have to be based on a good knowledge in the field, but also on a sound working strategy, to ensure that different design scenarios can be compared in a standardised and structured way. The Factor Method is a promising tool for such an evaluation and comparison, but there is still a lot of work to do to establish the necessary input data for reference service life and factor values, and to introduce the Factor Method into the design process.

Design life of building	**Inaccessible or structural components**	**Components where replacement is expensive or difficult (incl. below ground drainage)**	**Major replaceable components**	**Building services**
Unlimited	Unlimited	100	40	25
150	150	100	40	25
100	100	100	40	25
60	60	60	40	25
25	25	25	25	25
15	15	15	15	15
10	10	10	10	10
NOTE 1: Easy to replace components may have design lives of 3 or 6 years. NOTE 2: An unlimited design life should very rarely be used, as it significantly reduces design options.				

Table 1. Design lives of building components, services and buildings (years) to be selected for different types of buildings [ISO 2000].

3 FURTHER APPLICATION OF THE FACTOR METHOD

3.1 Design life

In the planning and design process of a new building, design lives of components, services and the whole building have to be chosen. In ISO 15686 Part 1, a table with proposal of appropriate design lives is given. A similar table has been published by the European Organization for Technical Approvals (EOTA) [EOTA 1999].

3.2 Reference service life

As mentioned in clause 2.3, the most comprehensive and laborious procedure for determination of reference service life (RSL) is presented in ISO 15686 Part 2. This procedure is based on different types of testing (field, laboratory), test houses, field investigations, etc. A procedure for provision of RSL of a building material or component has been presented in a Draft International Standard [ISO 2004]

3.3 Evaluation of the factors

In a total determination of the service life, it is important to be aware of how the Reference service life (RSL) as well as each of the factors are determined. Care should be taken to make sure that conditions influencing the specific value of each of these parameters are not mixed and taken into consideration multiple times. One example is the effect of incompatibility of materials (or a material and a surface treatment), which may be taken into consideration in relation to factor A, F or G. If a specific factor does not apply in determination of service life in a certain situation, the actual factor value is set to 1.0.

Factor A: This factor expresses the quality of the actual material used in a specific component. This may be related to the material quality itself, or to treatment (surface, impregnation) in order to protect the material against climate exposure (outdoor, indoor).

Factor B: This factor expresses the design level of the actual component or structure of a building. Of special interest is how the component or structure are designed in order to be protected against weather exposure, wetting and drying, etc.

Factor C: This factor expresses the skill of the construction workers, and to what extent the work is done according to accepted procedures (drawings, standards, guidelines, handbooks, etc.).

Factor D: This factor expresses the indoor environment exposing the actual component or structure. The factor usually does not apply for service life estimation of exterior materials, components or structures.

Factor E: This factor expresses the outdoor environment exposing the actual component or structure. It may be necessary to select different factor values for one specific climate exposure, depending on the type of material applied (wood, concrete, metals, polymers, etc.). This is due to the fact that different climate components (UV radiation, moisture, temperature, freeze/thaw) will be critical to different materials.

Factor F: This factor expresses the in-use conditions of the actual material or component. Such conditions may be general wear and tear, type of building where the material or component is applied, etc.

Factor G: This factor expresses the extent of maintenance of the actual material, component or structure. Poor maintenance may reduce the factor value to be selected, but in some situations also too intensive maintenance (e.g. short intervals of painting of exterior wood panels) may reduce the service life.

3.4 Selection of factor values

Much of the discussion and criticism of the Factor Method has been related to the fact that it is necessary to select a specific value for each of the factors, and that the multiplication of all the factor values makes the value of the Estimated service life (ESL) very sensitive to slight variations of each factor value. This is discussed in more detail in Hovde [Hovde 2002].

Factor value	General conditions for selection of factor values						
	A	B	C	D	E	F	G
5.0	Treated material				One relevant climate component is lacking		Maintenance with best available procedures
3.0	Excellent quality		Very good execution level				
2.0	Very good quality				Mild climate		Very good maintenance
1,5	Good quality		Good execution level				
1,2							
1.0	To be applied if conditions are similar to the RSL conditions, or if a specific factor does not apply.						
0,85							
0,67	Reduced quality		Bad execution level				
0,5	Poor quality				Severe climate		Poor maintenance
0,33	Very poor quality		Wrong mounting and fixing				
0,2	Material not applicable				Extreme climate		Lack of maintenance

Table 2. Guideline for selection of factor values (examples).

The studies of the Factor Method so far have proved that it is necessary to have simple procedures for selection of factor values. One important aspect is then the relation to the actual design life as shown in Table 1. A building may have a design life of 60, 100, 150 or even an unlimited number of years, and it is not possible to prescribe the future conditions of such buildings in detail. For estimation of service life of a material or component that is to have a rather short design life, it is important to be able to obtain a value as correct as possible regarding the actual number of years. For larger design lives, however, there will always be a number of uncertainties, and it is not necessary to estimate the service life to the nearest year or even 5 to 10 years. Therefore, some general values for selection of each of the factors should be developed, and they will be a practically applicable choice of values. Examples of such values are given in Table 2. The table has to be further developed based on existing knowledge, and it may be refined in the future based on further experience.

Factor A: Quality of components	
Factor	Qualities for selection of factor value

value	Concrete	Steel	Wood	Polymer
5.0		Stainless steel	Heartwood	
3.0	High strength concrete with reduced porosity		Impregnated wood Tropical wood	
2.0		Surface treatment, type A	Surface treatment, type A	Extra UV stabiliser added
1,5	Concrete with low w/c ratio	Surface treatment, type B	Surface treatment, type B	
1,2				
1.0	To be applied if conditions are similar to the RSL conditions, or if a specific factor does not apply.			
0.85				
0.67			Sapwood	
0.5	Concrete with high w/c ratio			
0.33		Incompatible surface treatment	Incompatible surface treatment	No UV stabiliser applied
0.2	Concrete with high w/c ratio and reactive aggregate			

Table 3. Guideline for selection of factor values in relation to material qualities (examples).

Factor D: Outdoor environment				
Factor value	Outdoor climate conditions for selection of factor value			
	Concrete	Steel	Wood	Polymer
5.0	Dry climate, no pollution	Dry climate, no pollution	Dry climate, no pollution	
3.0	Dry climate	Dry climate	Dry climate	
2.0				Low UV radiation
1,5				
1,2				
1.0	To be applied if conditions are similar to the RSL conditions, or if a specific factor does not apply.			
0.85				
0.67	Freeze/thaw conditions apply			
0.5		Industrial atmosphere		High UV radiation
0.33		Marine climate	Wet climate	High UV radiation, ozone
0.2	Severe freeze/thaw conditions	Severe industrial atmosphere		

Table 4. Guideline for selection of factor values in relation to outdoor environment (examples).

Each value may be related to specific conditions for each of the factors. If the applicant of the Factor Method has more specific information for one or more of the factors, it will be possible to select a factor value from interpolation between the two nearest values given. The application of such pre-selected values for each of the factors will also be a good basis for evaluation of the importance of each factor and an important support in understanding the applicability, limitations and practical use of the Factor Method.

The system of selection of factor values as shown in Table 2 is very general. It may be a first, crude system for application to all types of materials and components in buildings. However, the practical application of the system may reveal that it will be necessary to refine the system to take care of different types of materials or components. This can be further developed as shown in Table 3 and 4. The tables may be expanded for other types of materials or components, and similar tables have to be developed for each of the factors A-G. When applying values as given in table 3 and 4, it is important to be aware of the initial value of the RSL. It should be stressed that the text given in the tables 2-4 are just examples. These examples have to be verified or ajusted, and further examples have to be stated for the the empty boxes.

3.5 Estimated service life

Calculation of Estimated service life (ESL) is performed by use of the Factor Method as shown in clause 2.2. Selection of appropriate values for Reference service life (RSL) and each factor, respectively, may be done as explained in clause 3.2 and 3.3. For evaluation of the outcome of the calculation, it is important to be aware of the description of ESL as given in ISO 15686 Part 1. ESL does not represent an exact value of the service life, and the Factor Method merely gives an empirical estimate of the service life of a building material or component.

4 NEEDS FOR INPUT DATA

The main parts of the Factor Method are the Reference service life (RSL) and the factor values. In order to apply the method in a practical and reliable way, it will be necessary to establish data sources presenting specific values for these parameters. RSL values have to be furnished by manufacturers of building materials and components. In the future, publication of such values should be a regular part of material or component data sheets etc. It may also be possible to publish more general data for RSL of types of materials or components, as an input to a first calculation of Estimated service life (ESL). A database for selection of factor values has to be developed as illustrated in clause 3.4. The development of such a database should be a joint task for industry, testing and research institutes, etc., because it requires input data and evaluation of a variety of conditions and parameters.

5 CONCLUSIONS

The paper presents a new proposal for selection of input data for practical application of the Factor Method. The proposal will be a support for users of the Factor Method in estimating the service life of a building material or a component in the design and engineering phase of a building. The proposed system also underlines the need for joint efforts to establish available data bases for Reference service life (RSL) of building materials or components, as well as an appropriate range of values for the different factors A-G.

REFERENCES

ISO 2000, ISO 15686 Part 1. Buildings and constructed assets – Service life planning – Part 1: General Principles. International Organization for Standardization, Geneve, Switzerland.

Architectural Institute 1993, The English edition of Principal Guide for service life planning of buildings. Architectural Institute of Japan, Japan.

ISO 2001, ISO 15686 Part 2. Buildings and constructed assets – Service life planning – Part 2: Service life prediction procedures International Organization for Standardization, Geneve, Switzerland.

Hovde, P. J. & Moser, K. 2004, Performance based methods for service life prediction. State of the art reports, Part A and Part B. CIB Report: Publication 294. CIB, Rotterdam, The Netherlands. ISBN 90-6363-040-9.

Hovde, P. J. 2002, The factor method for service life prediction. From theoretical evaluation to practical implementation. Paper 232, Proceedings of 9th International Conference on Durability of Building Materials and Components (9DBMC), Brisbane, Australia, 17.-20. March 2002.

Moser, K. & Edvardsen, C. 2002, Engineering design methods for service life prediction. Paper 222, Proceedings of 9th International Conference on Durability of Building Materials and Components (9DBMC), Brisbane, Australia, 17.-20. March 2002.

Lacasse, M. & Sjöström, C 2004, Recent advances in methods for service life prediction of building materials and components – an overview. Proceedings, CIB World Building Congress, Toronto, Canada, 2.-7 May 2004.

Teply, B., Kersner, Z. & Rovananikova, P. 2003, Risk analysis using factor method. Proceedings, International Workshop Management and Durability in the Building Process, Milan, Italy, 25.-26. June 2003.

Cusmano, G. & Lucchini, A 2003, Implementing the Factor Method for service life estimation using weighed-factors. Proceedings, International Workshop Management and Durability in the Building Process, Milan, Italy, 25.-26. June 2003.

Re Cecconi, F. 2003, Service life planning: further development of the Factor Method. Proceedings, International Workshop Management and Durability in the Building Process, Milan, Italy, 25.-26. June 2003.

De Pascale, A. 2003, Proceedings, International Workshop Management and Durability in the Building Process, Milan, Italy, 25.-26. June 2003.

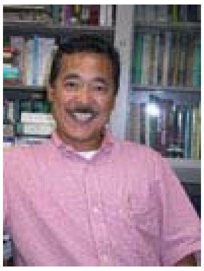
Re Cecconi, F. 2004, Engineering method for service life planning: the evolved Factor Method. Proceedings, CIB World Building Congress, Toronto, Canada, 1.-7. May 2004.

Marteinsson, B. 2003, Assessment of service lives in the design of buildings – Development of the Factor Method. Licentiate thesis. KTH Research School – HIG Center of Built Environment, Gävle, Sweden.

EOTA 1999, Assumption of working life of constructional products in guidelines for European Technical Approval, European Technical Approvals and harmonized standards. EOTA Guidance Document 002, European Organization for Technical Approval, Brussels, Belgium.

ISO 2004, ISO/DIS 15686 Part 8. Buildings and constructed assets – Service life planning – Part 8: Reference service life. International Organization for Standardization, Geneve, Switzerland.

Computer 3d Model of Concrete Chloride Penetration



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ABSTRACT

The durability of concrete is an important issue, which has to be considered at the initial stage of design by taking into account performance specifications. One of the durability aspects is the concrete resistance towards chloride ion penetration, i.e. concrete chloride diffusivity. Factors like water-cement ratio, degree of hydration, volume of the aggregates and their particle size distribution are with a significant effect on concrete chloride diffusivity. The use of polypropylene fibers (especially very fine and well dispersed micro fibers) is shown to produce a significant reduction in the concrete permeability.

The main objective of this study is to evaluate how fiber inclusion (in terms of volume and size) influences the concrete chloride diffusivity by applying a 3D computer modeling for the composite structure and simulation of the chloride penetration. The approach includes a generation of multi-scale microstructural computer model of both plain and polypropylene fiber concretes, and simulation and evaluation of chloride diffusivity of these systems by applying random walkers algorithm. Multi-scale microstructural model at micro millimeter scale describes the hydration development in the modeled structure and at millimeter scale – its percolation and the chloride diffusivity. Fiber volume, fiber size (3D), degree of hydration are considered as major factors, and the modeled parameters, i.e. concrete chloride diffusivity, are compared to the experimental data obtained in parallel conducted chloride migration test experiment of these concrete mixtures.

In a result a good agreement is found between multi-scale microstructural model and predicted through it concrete chloride diffusivity and experimentally measured one, they were found to be of the same order. The results of the simulation when only fiber volume was considered showed a slight decrease of concrete chloride diffusivity with a fiber volume increases from 0 to 1%. It could be explained by the fact that when volume of aggregates increases until certain limit this leads to both a reduction of diffusivity in the bulk paste and an increase of volume of interfacial transition zone, but the former effect is stronger than the latter by this limit. On the other hand the simulation results showed when both fiber volume and fiber geometry were considered that the fiber size has an important role for more precise simulation of concrete chloride diffusivity. The described above approach can be applied in the process of prediction of concrete chloride diffusivity and to be used in durability and service life design of concrete with polypropylene fibers.

KEYWORDS

Chloride penetration, computer model, concrete chloride diffusivity, polypropylene fiber.

1 INTRODUCTION

The durability of concrete is an important issue, which has to be considered at the initial stage of design by taking into account performance specifications. The lasts are still under development and approval, but having durability-based design codes would in great extent assure better service life performance of concrete structures. One of the durability aspects is the concrete resistance towards chloride ion penetration, i.e. concrete chloride diffusivity. Factors like water-cement ratio, degree of hydration, volume of the aggregates and their particle size distribution are with significant effect on concrete chloride diffusivity. The use of polypropylene fibers (especially very fine and well dispersed micro fibers) is shown to produce a significant reduction in permeability through a modification of crack topography. At the same time, the use only of macro polypropylene fibers produces higher permeability, mostly because of poor dispersion.

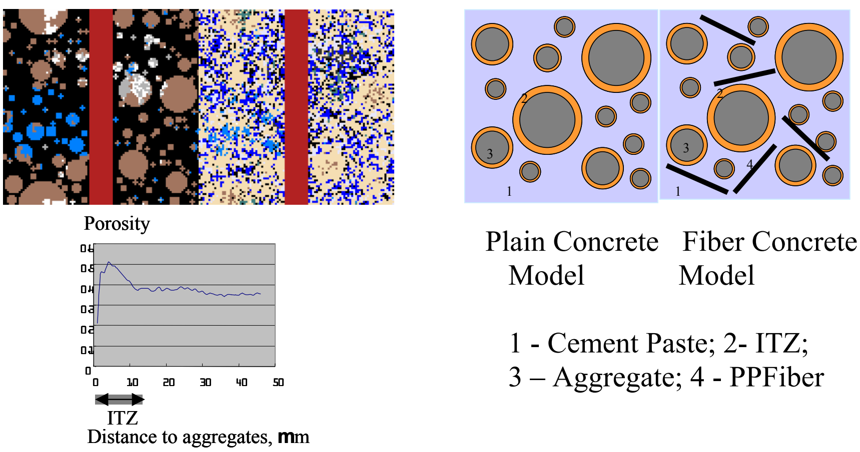
The main objective of this study is to evaluate how fiber inclusion (in terms of volume and size) influences the concrete chloride diffusivity by applying a 3D computer modeling for the composite structure and simulation of the chloride penetration.

2 MULTI-SCALE MODELING AND TECHNIQUES

The multi-scale modeling approach has been proposed and applied by Bentz [2000] for prediction of chloride ion diffusivity of plain concrete. Here this model has been extended to both plain and fiber reinforced concretes.

2.1. Modeling steps

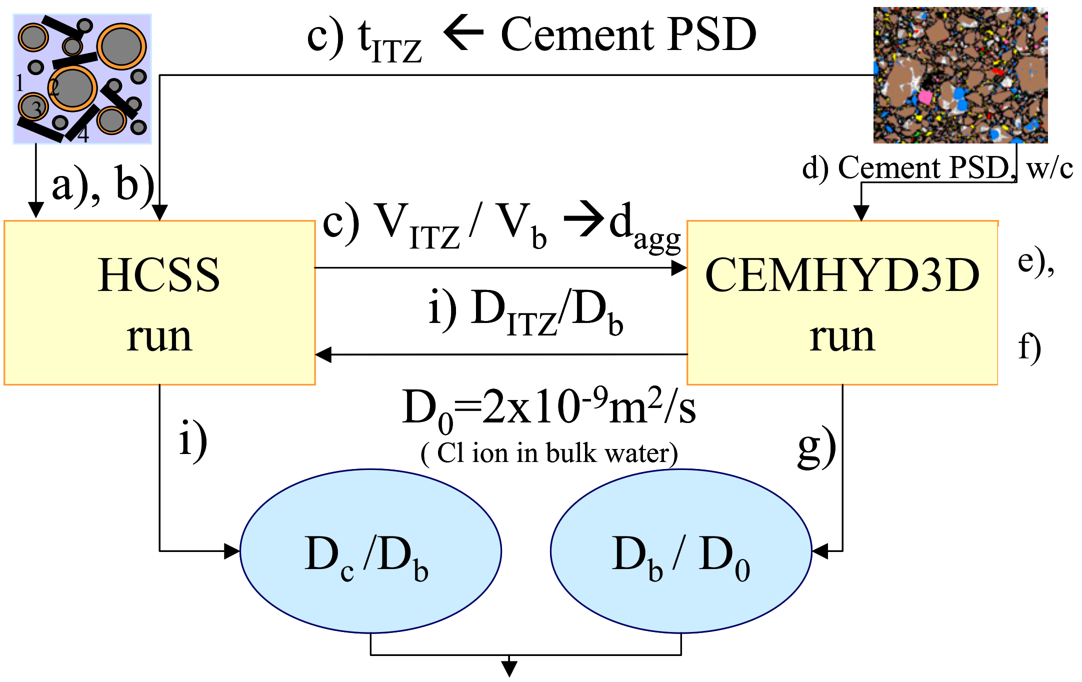
The approach includes a generation of multi-scale microstructural computer model of both plain and polypropylene fiber concretes, and simulation and evaluation of chloride diffusivity of these systems by applying random walkers algorithm. Multi-scale microstructural model at micro scale describes the cement paste surrounding a single aggregate, the hydration development and percolation in cement paste, and in the Interfacial Transition Zone as it was proposed by Benz and Garboczi [1998]. The model at a millimeter scale describes the aggregate particles in the considered volume, as well as the fibers when they are employed. The multi-scale model is illustrated in Fig. 1. The models at these two scales are interconnected together with the technique for computation of relative diffusivity of a three-dimensional microstructure and finally to compute the diffusivity of a concrete [Bentz et. al. [1997]].



a) Micro scale model (CEMHYD3D)   b)Millimeter scale model(HCSS)

Figure 1. Multi-scale microstrutural concrete model.

The simulation procedure includes a cellular-automation based 3D model for cement paste hydration and microstructure development (CEMHYD3D)[Bentz, 2000] and a 3D hard core/soft shell (HCSS) model for concrete [Bentz et. al., 1999]. The modeling approach consists of next steps (illustrated in Fig. 2 as well): **a)** aggregate particle placement in the computational volume of modeled concrete, as the placement is random from largest to smallest particles and the distribution follows the supplied sieve analysis classification; **b)** fiber particle placement as cylindrical shape and according to the designed volume of fibers for each fiber type considered; **c)** execution of HCSS model with initially established thickness of ITZ, t_{ITZ} to be equivalent of the median cement particle diameter known from cement particle size distribution; **d)** cement particle placement in correspondence with desired w/c ratio and the thickness of a single aggregate particle (d_{agg}) is assumed to be the ratio of calculated in previous step c), volume of ITZ, V_{ITZ} to the volume of bulk (cement) paste, V_b; **e)** execution of CEMHYD3D and simulation of cement paste microstructure hydration until the degree of hydration of interest is achieved; **f)** analysis of the porosity in the bulk (cement) paste as a function of distance from the aggregate surface; **g)** calculation of local relative diffusivity in the bulk (cement) paste and in ITZ region according to the suggested by Garboczi & Bentz [1992] relationship (1), seen in Fig. 2; **i)** execution of random walker algorithm [Garboczi et. al., 1995] in HCSS model considering the ratio of average ITZ diffusivity, D_{ITZ} to the average bulk (cement) paste diffusivity, D_b determined in the previous step in order to determine the effective diffusivity of concrete system relative to the average bulk (cement) paste diffusivity (D_c/D_b); **j)** finally, a calculation of the absolute chloride ion diffusivity for the concrete, D_c according to the relationship (2) (Fig. 2).



$$\text{j)} \quad D_c=(D_c/D_b)(D_b/D_0)D_0 \quad \ldots(2)$$

$$\text{g)} \quad \frac{D_b}{D_0} = 0.001+0.07\Phi^2+1.8H(\Phi-0.18)^3 \quad \ldots(1)$$

$$\Phi\text{-Porosity, H-Heaviside function} \begin{cases} 1, \text{ for } \Phi>0.18 \\ 0, \text{ otherwise} \end{cases}$$

Figure 2. Simulation flow and steps.

2.2. Variables of the model

Previous research [Bentz et. al., 1998] on the identification of significant factors influencing concrete diffusivity has shown that with the highest effect are the w/c ratio, degree of hydration and volume fraction of aggregates. Factors like aggregates' PSD, t_{ITZ}, and air content also have an influence on the diffusivity, but in less extent. In this study, the w/c ratio and volume fraction of aggregates were kept constant, as that was the case in the experimental mixtures as well. Degree of hydration was selected as an equivalent to the ages of 28 days, 60 days and 90 days. Fiber volume and fiber size are considered as major factors. Three different polypropylene fiber types were employed. Additionally, the overall effect of the fibers classified as micro fibers and macro fibers was simulated. For that purpose the millimeter scale model, namely hard core-soft shell model was handled in two different series of simulation. In the first series of simulation fiber volume and size were considered, and aggregates and fibers were strictly distinguished as separate phases. In the second series of simulation only fiber aspect ratio was considered. That was expressed through fiber volume addition to the aggregate volume for the case of macro fibers, and the presence of micro fibers was expressed by the difference in the thickness of interfacial transition zone, t_{ITZ}. Details about the mixture proportions, cement type, and fiber type and volume are presented in Table.1. The information on the performed simulations in Series I (by fiber volume and degree of hydration) and Series II (macro and micro fibers through the aspect ratio) are summarized in Table 2.a) and Table 2.b) respectively.

Mix	*Fiber Type*	*Fiber Size* *L, mm*	*d, mm*	*Aspect ratio, A_f*	*W, kg*	*C, kg*	*S, kg*	*G, kg*	*S/A, %*
PC	-	-	-	0					
PP12M	Mesh	12	0.31	38.7	175	350	913~927	865~878	51
PP10S	Monofil.	10	0.23	43.5					
PP30S	Monofil.	30	1.0	30					

Table 1. Fiber type, size, mixtures proportions (w/c=0.5 for all mixtures).

Fiber Volume, % / *Fiber Type*	*0.1*	*0.3*	*0.5*	*1.0*	*1.5*
PP12M	V	V	V		
PP10S		V	V	V	
PP30S			V	V	V

Table 2.a) Series I. Fiber volume and size (Degree of hydration – at 90 days).

	Macro fibers			*Micro fibers*		
A_f	30	38.7	43.5	60	400	860
V_f, %	1.0	0.1	0.5			
t_{ITZ}, mm				55	37	19
d_f, mm	1.0	0.31	0.23	0.086	0.050	0.014

Table 2.b) Series II. Macro and micro fibers by aspect ratio, volume and t_{ITZ} (Degree of hydration – at 28, 60 and 90 days).

The ordinary portland cement used in all of the computer runs is with Blaine fineness of 350 m^2/kg and has the following composition on a volume basis: C_3S – 0.702, C_2S – 0.132, C_3A – 0.083, C_4AF – 0.084. The cement paste (micro scale) model is with a resolution 1 mm/pixel. The concrete system (millimeter scale) model is a cube of 100 units on a side and a real size of 30 mm (and 35 mm in case of polypropylene fiber of 30 mm length). The simulation results received in Series I are compared with the experimental results obtained in a experimental setup explained below, but those in Series II are not, as the experiments with very fine micro fibers are not completed yet.

2.3. Chloride migration experimental procedure

The experimental program and procedures are explained in details by Antoni [2004], but in brief the test setup is presented in Fig. 3. A major concern for selecting a chloride migration test based on Standard of NordTest Build 492 is that this is a rapid test and produces more consistent results, which can be used directly for service life prediction. Chloride diffusion coefficients of plain and polypropylene fiber reinforced concretes (the mixtures are presented in Table 1) were measured for both non-loading and under-loading conditions, but here only the former are of interest.

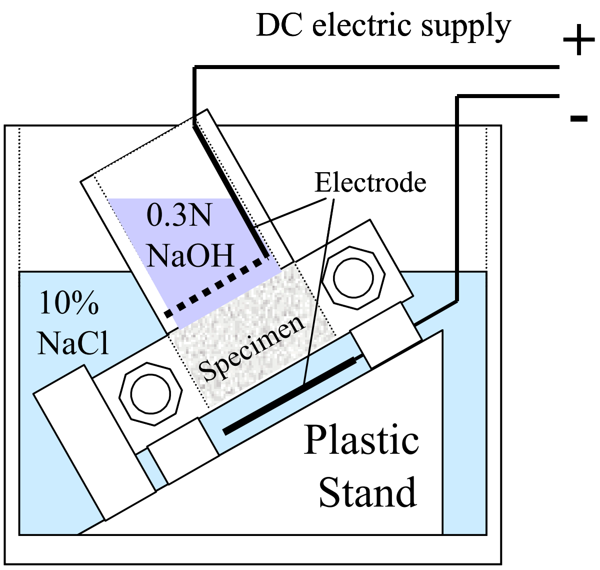


Figure 3. Migration test setup.

The fresh concrete was cast into 100x100x400 mm rectangular prism, unmolded 24 hours later and cured in water for 28 days. After the water curing, the prismatic specimens were cut into 50 mm thick specimens with 100x100 mm cross-section. All specimens were placed in room condition for additional 60 days before the migration test.

3 RESULTS AND DISCUSSION

3.1 Simulation results of model in Series I and migration experimental test

The multi-scale model was executed as explained in the previous section. Predicted values of absolute chloride ion diffusivity for the concrete, D_c at 90 days and their graphical representation in relation with the fiber type and fiber volume are shown in Table 3 and Fig. 4.a) respectively.

Fiber Volume, % / *D_c,(*$10^{-12}m^2/s$)*	*0*	*0.1*	*0.3*	*0.5*	*1.0*	*1.5*
PC	13.004					
PP12M		13.094	13.019	13.144		
PP10S			13.009	13.072	13.008	
PP30S				12.936	13.025	13.030

Table 3. Simulation results for absolute chloride ion diffusivity of concrete, D_c (*10^{-12} m^2/s).

The measured and determined chloride ion diffusivity for concrete D_{cm} after the migration test and reported by Antoni [2004] are presented in Table 4 and Fig. 4.b) respectively.

Fiber Volume, % / D_{cm}, ($*10^{-12}m^2/s$)	0	0.1	0.3	0.5	1.0	1.5
PC	16.099					
PP12M		16.701	16.634	22.551		
PP10S			14.378	15.388	15.662	
PP30S				13.966	14.973	14.423

Table 4. Experimental results for absolute chloride ion diffusivity of concrete, D_{cm} ($*10^{-12}$ m^2/s).

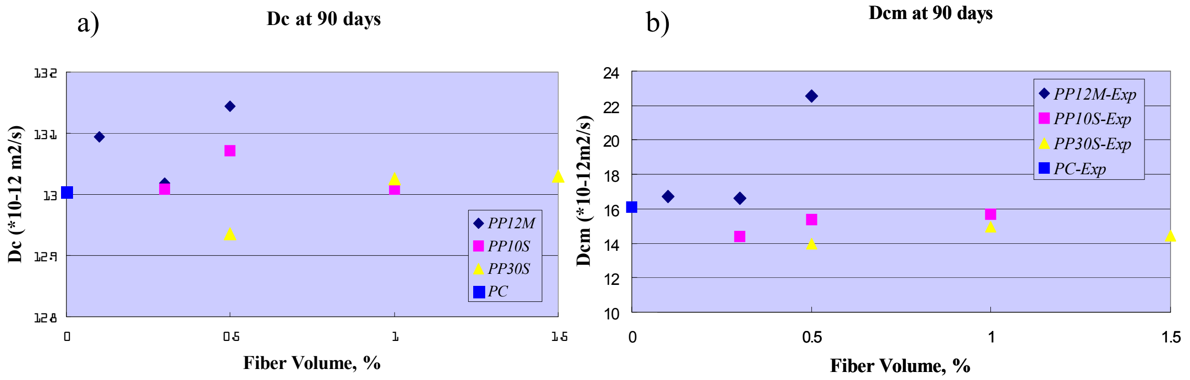


Figure 4. Effect of fiber volume and type on the absolute chloride ion diffusivity of concrete: a) Multi-scale model simulation results; b) Migration test.

The multi-scale model results show that concrete diffusivity is highest in polypropylene mesh type fiber reinforced concrete, and with the increase of the fiber content there is an ascending trend for D_c to increase as well. Similar dependence is found in the experimental results (Fig. 4.b)) for PP12M type fiber. The simulation output for the other two fiber types does not show any particular trend for an increase or decrease of D_c. A comparison of the values of D_c in PP10S and PP30S mixtures with that of plain concrete shows that they are of the same order independently of the fiber volume employed in the concrete. Experimental results show similar relationship but the diffusivities of PP10S and PP30S are reduced when compare them with that of plain concrete. And moreover, diffusivity of PP30S is less than the one of PP10S. The main reason for that is the influence of casting direction of fibers, which is higher in the case of longer fibers [Antoni, 2004].

The simulation values and experimentally measured values are found to be of the same order and to show the same trend of influence of the fiber type or volume. The interval range of experimentally measured diffusivity is higher due to the effect of various factors (i.e. mixing, casting, preparing of test specimens etc.), which were not entirely considered and reflected in the simulation process.

3.2 Simulation results of model in Series II

It is of interest to find out what is the effect of fiber shape and size expressed through the fiber aspect ratio on the chloride penetration of the concrete. Here, the macro fibers are considered to be with an aspect ratio up to ~50 and those with aspect ratio more than 50 to be considered as micro fibers. The results of multi-scale model simulation in these cases are presented in Fig. 5. The absolute chloride ion diffusivity of the concrete for macro fibers is calculated by adding fiber volume to aggregate volume. The results (in larger scale on right side of Fig. 5.) show a slight trend of concrete chloride diffusivity to decrease when the fiber volume increases from 0 to 1%. This could be explained by the fact that when the volume of aggregates increases until certain limit this leads to both a reduction of diffusivity in the bulk (cement) paste and an increase of volume of interfacial transition zone, but the former effect is stronger than the latter by this limit.

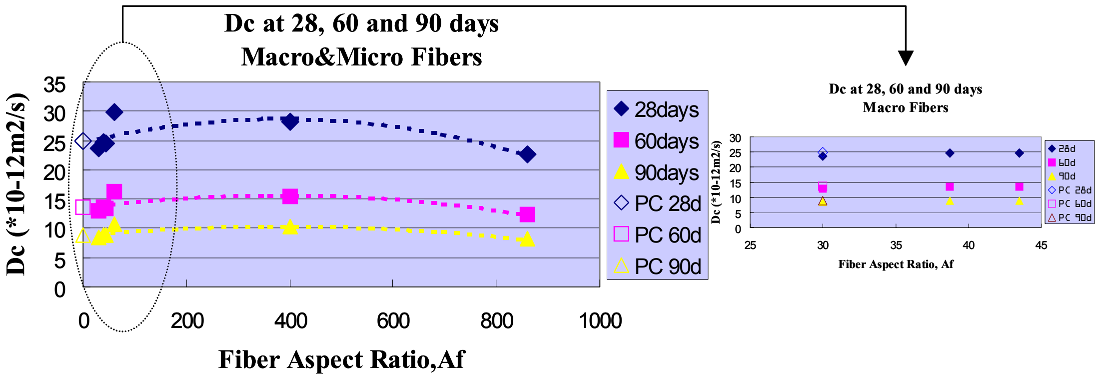


Figure 5. Effect of fiber aspect ratio on the absolute chloride ion diffusivity of concrete.

The parabolic curves which fit the data at 28, 60 and 90 days show that the application of very fine micro fibers could be very beneficial in order to keep or improve the concrete resistance to chloride penetration due to the reduction of the porosity of interfacial transition zone. The effect of degree of hydration (here at the age of 28, 60 and 90 days) is significant as already reported by Bentz et. al. [1998]. At early age the porosity in ITZ is quite high compare to the one of the bulk (cement) paste. Both, ITZ porosity and bulk (cement) paste porosity, tend to become closer as the hydration proceeds and therefore the reduction in D_c is expected.

A comparison the results of multi-scale model application (Series I and Series II) in the part of effect of fibers on the absolute chloride ion diffusivity shows that taking in a consideration both fiber volume and fiber size would produce more closer system representation to the real one. The average D_c in Series I at age 90 days is of ~$13*10^{-12}m^2/s$, the same of Series II is of ~ $8*10^{-12}m^2/s$ and experimental one is of ~$16*10^{-12}m^2/s$.

4 CONCLUSIONS

Based on the simulation, the models established, and the comparison with the experimental results, it could be summarized that computer multi-scale based models are good tool to predict transport behavior of the concrete.

The simulation values and experimentally measured values are found to be of the same order and to show the same trend of influence of the fiber type and fiber volume. The simulation results are with about 20% less than experimentally measured one. On the other hand the simulation results when both fiber volume and fiber geometry were considered show that the size of the fibers has an important role for more precise simulation of concrete chloride diffusivity.

The simulation model confirm the significance of the degree of hydration and highlight the possibility that the application of very fine micro fibers could be very beneficial in order to keep or improve the concrete resistance to chloride penetration. The experimental results do not show any significant change in the concrete chloride diffusivity when different volume or type of fibers is employed.

The described above approach can be applied in the process of prediction of concrete chloride diffusivity and to be used in durability design and service life design of concrete with polypropylene fibers.

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6 REFERENCES

Antoni, 2004, *Chloride Penetration into Fiber Reinforced Concrete under Loading,* High Performance Concrete Laboratory, PhD. Thesis, Hokkaido University, Japan, August.

Benz, D.P., 2000, 'Influence of Silica Fume on Diffusivity in Cement-Based Materials. II. Multi-scale Modeling of Concrete Diffusivity', *Cement and Concrete Research*, Vol.30(7), 1121-1129.

Benz, D.P., Garboczi, E.J. & Lagergren, E.S., 1998, 'Multi-scale Modeling of Concrete Diffusivity: Identification of Significant Variables', *Cement, Concrete, and Aggregates*, 20(1), 129-139.

Bentz, D.P., Detwiler, R.J., Garboczi, E.J., Halamickova, P. & Schwartz, L.M., 1995, 'Multi-scale Modelling of the Diffusivity of Mortar and Concrete', Proc. RILEM International Conference on Chloride Penetration into Concrete, Paris, France, pp. 85-94.

Bentz, D.P., 2000, *CEMHYD3D: A Three-Dimensional Cement Hydration and Microstructure Development Modelling Package. Version 2.0,* NISTIR Report 6485, NIST, Gaithersburg, USA, April.

Benz, D.P., Garboczi, E.J. & Snyder, K.A., 1999, *A hard core/soft shell microstructural model for studying percolation and transport in three-dimensional composite media,* NISTIR Report 6265, NIST, Gaithersburg, USA.

Garbotzi, E.J. & Bentz, D.P., 1992, 'Computer Simulation of the Diffusivity of Cement-Based Materials', *Journal of Materials Science*, 27, 2083-2092.

Garboczi, E.J., Schwartz, L.M., & Bentz, D.P., 1995, 'Modelling the influence of the interfacial zone on the D.C. electrical conductivity of mortar', *Advanced Cement-Based Materials*, **2**, 169-181.

Durability experimental evaluation of building components in sustainable production process: The Italian research program

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ABSTRACT

The paper presents a research program on durability and an experimental evaluation of building components used within a sustainable production process supported by the Italian Ministry of education, universities and research (MIUR). The proposed research program intends to develop methodologies applicable to the design and evaluation of an initial set of components categories: load-bearing and non load-bearing walls, external windows, and flat and inclined roofs. Six Italian Universities are involved including the: Polytechnic of Milan, Polytechnic of Torino, and the Universities of Naples, Palermo, Brescia, and Catania. The expected results will provide relevant methodological and experimental benchmark from which to initiate a national network of experimentation focused on the durability of building components. This network will draw upon the resources available in the respective technical laboratories at which accelerated aging tests will be conducted on the different components in reference test conditions. As well, outdoor tests will be conducted to help determine the accelerated aging effects on the components when subjected to different weather induced stress levels associated with a given climate. Additionally, the work will be relevant to determining a component's conventional performance limits from which may be estimated the Reference Service Life. The information derived from this work will be used to support a software tool that will be very useful for building or component designers. The development of this software tool is specifically aimed at aiding design decision makers and building maintenance planners. This work will increase knowledge on the durability of building components, and as well, contribute to related CIB Research, and in turn, through collaboration with the Italian Institute for Standardisation (UNI), aid the development of relevant ISO standards.

KEYWORDS

DURABILITY, SERVICE LIFE, BUILDING COMPONENTS, MAINTENANCE.

1. THE ITALIAN RESEARCH PROGRAM

The research program intends to develop methodologies to be applied to design and to experiment the durability evaluation of building components and to test the evaluation on a first set of components categories: load-bearing and not load-bearing walls, external windows, flat and inclined roofs.

On this theme the International Council for Research and Innovation in Building and Construction (CIB) is developing, through the Working Group CIB W 80 –RILEM 175 "Service life methodologies" a hard work to examine and co-ordinate the researches that are being worked out in Europe and in the World, with a pre-normative scope and the International Standard Organisation (ISO) with the sub-committee TC 59 /SC14 is working to set up standards related to service life prediction of products and buildings. The knowledge about construction materials durability has been structuring and developing for innovative materials. It's not like that for the knowledge about complex building components durability that is the subject of this research program, i.e. building components aimed to have different technological functions and then to supply various performances related to the multiplicity of technological requirements, specific for the each component's category. In this case the time decay of each performance appears in different ways. It's to note that durability, intended as the attitude of the component to maintain the initial performance level substantially constant, is a technological requirement transversal with regard to the other requirements that have to connote the initial technological quality(at time zero) of the component. In fact the European Directive n. 106-1989, regarding construction products, requires for the Products Technical Approval, the fulfilments of the Essential Requirements for an economically reasonable working life. This is to guarantee the building component's technological quality and the whole building's sustainability. Taking into account these considerations, we have involved in the Research Program six Research Units of so many Universities (Politecnico di Milano, Politecnico di Torino, Università di Napoli, Università di Palermo, Università di Catania, Università di Brescia), that with the Research Unit of Politecnico di Milano have followed the development of CIB international research and ISO standardisation works.

The results we intend to obtain aim to set up a relevant methodological-experimental reference to start with a systematic experimentation about building components' durability on a National level to be worked out in a national network of technological laboratories (accelerated aging tests in reference conditions) and outdoor monitoring for the component's time re-scaling in different stressing weathering contexts, with the scope to supply an increasing knowledge about durability, to contribute to the CIB International Research, and also to the development of ISO standards on these themes, through UNI work. In fact UNI (Italian Standardisation Institute), starting from a proposal of the Research Unit of Politecnico di Milano, has constituted during 2002, an "ad hoc" Working Group to elaborate a draft standard on building components durability.

A very relevant and innovative scope that is foreseen in the research, through the set up of methodologies applicable for the durability experimental evaluation of building components is also to determine the component's conventional performance limits to estimate the ending time of Reference Service Life, also looking at the proposal of an Engineering Method (as it's suggested in ISO standard) to evaluate the component's Service Life on the base of the specific stressing indoor and outdoor environmental context where the building work is located.

This method will be supported by a software tool, which will be very useful for designers of the components and of the whole building, in order to obtain a durability prediction of the components and of the whole building, in the framework of an integrated sustainable building process, and specifically of a sustainable production process.

The research program is structured into three development stages that will last 24 months totally, so organised to reach the scope to set up a relevant methodological-experimental reference, to start with a systematic experimentation about building components' durability on a national level, hopefully co-ordinated by the Italian Universities. The research program stages will be worked out in a strict co-ordination among the six involved research units and with the Program Scientific Coordinator .

Stage 1 – Preliminary set up of the durability evaluation methodology for building envelope components

The Milan Research Unit will deal with the category of non load-bearing external walls and for this category will define the relevant technological requirements, will analyze the component's initial performance levels and the functional characteristics, will determine the functional and performance thresholds, will analyze the relevant agents and the degradation effects. The Brescia Research Unit will deal with the category of load-bearing external walls and for this category will work as the Milan Research Unit. The Turin Research Unit will deal with the category of external windows and for this category will work as the Milan Research Unit. The Palermo Research Unit will deal with the category of inclined roofs and for this category will work as the Milan Research Unit. The Naples Research Unit will deal with the category of horizontal roofs and for this category will work as the Milan Research Unit. The Catania Research Unit will deal with the different classes of building's components considered by the other research units in order to obtain a durability estimation, specific for the context of Mediterranean Area. The expected results in this first Research Program stage will consist in the development of the durability evaluation methodologies for building components of load-bearing and not load-bearing external walls, external windows, horizontal and inclined roofs. These methodologies will be the subject of a first research report that will constitute the starting point for the experimental stage of the research program, report that will be presented to the sector operators at the end of the first research year.

Stage 2 – Experimental program for building components ageing

The Milan Research Unit will deal with the experimentation in technological laboratory, following the methodology defined in the first research stage, for the accelerated ageing with testing cycles specifically planned, on building components of non load-bearing external walls and will deal with the outdoor monitoring of the same kind of components, to reach the time re-scaling of time durability in regard with the technological laboratory results; the relevant standards and technical scientific documents on durability for the specific building components' category will be considered as a starting point. The Brescia Research unit will do the same as for building components of load-bearing external walls category , the Turin Research unit as for building components of external windows category, the Palermo Research unit as for inclined roofs category, the Naples Research unit as for horizontal roofs category and the Catania Research Unit, as for the durability evaluation of the components' classes considered by the other research units in the Mediterranean Area. In particular the Palermo Research Unit will work at the audits of buildings where the tested components are. The expected results of this second stage of the Research Program will consist in the data acquisition from technological laboratory experimentation and from outdoor monitoring of components' samples. All the data obtained from experimentation will be communicated through a second research report which will be the base for the third stage of the research, regarding with the data analyse and results' interpretation, report that however will be communicated to the sector's operators at the end of the second research stage.

Stage 3 – Analysis and results' interpretation

During this third and last stage of the Research Program all the involved research units will work in a strict co-ordination to analyse the results obtained in the second stage and to their interpretation, in particular to compare the data from laboratory accelerated ageing and outdoor weathering in order to estimate the performance decay during time of the tested components and to predict service life of various category of components. It's to notice that during this third and last stage of the Research

Program the Naples unit intends also to apply the NIC method at the results obtained in the second experimental stage, and then the corrective coefficients will be defined to be introduced to the value of mean reference service life. The result of this third research stage will consist in the indication for each studied building component of the time behaviour of tested components and in the durability prediction, i.e. of Reference Service Life. This will be contained in the third research report, that will be communicated to the sector's operators at the end of this last research stage. The three reports at the end of the three stages of research program once systematically revised are intended to be published and diffused in the national and international Scientific Community.

2. SPECIFIC RESEARCH PROGRAM OF THE SIX RESEARCH UNITS

2.1Research Unit of Politecnico di Milano

The specific subject of the program is related to <u>Methodological development in order to design and to estimate the durability of building components: experimental evaluation of durability for non load-bearing external walls.</u> The program aims to set a methodology for designing and estimating the durability of building components, specifically for the components' class of non load-bearing external walls, starting from the durability concept as the attitude to maintain unchanged over time the performances' levels. The obtained research results as reference service life will be considered by the designer as an input to be corrected in order to take into account the actual context of the designed building, its use destination and other environmental factors that will influence the design service life prediction for a specific building, considering management and maintenance. To do this an engineering method to determine component's conventional performance limits is being developed in order to recognize the reaching the end of Service Life, as a function of the specific use contest of the component.

The method will be supported by a software tool, which will enable its application to the components' durability by designers. In future the availability of certified data for different products, (requested for example in the CE marking of construction products) about the reference time behaviour of performance characteristics, will allow the designer to choose on the market the fitted products for the specific work, considering the specific stressing context and the foreseen use conditions. Besides on the base of these data the designer comes to a service life prediction and to a maintenance planning, once durability evaluation method has been set up and validated experimentally. This performance-based approach has been developed in coherence with the factorial and engineering methods studied at CIB W80 RILEM 175 and discussed at ISO TC 59 SC14.

To reach this goal:

- In the first stage the durability evaluation methodology will be developed for the specific components category. In particular the modelling of relations between functional characteristics' decay and technological performances which influence the environmental conditions, will be used at the design stage to simulate the time decay of technological performances (and the variation of indoor conditions) and then to predict the service life during real building design (method to determine the performance's limits);

- In the second stage tests will be scheduled and accomplished through laboratory-based artificial ageing trials on external non load-bearing walls made of insulated bricks with different protective paintings. The assessing and forecasting procedures could be performed only when precise technical solutions have been envisaged and analysed. Furthermore, the Research Unit's tasks include investigations about how the technical solutions (previously built) are performing and have been affected over time when a natural ageing process has been triggered. Contemporary natural weathering testing will be developed outdoor in different locations, depending on the various kinds of weathering conditions; such task obviously entails a close partnering attitude and a special commitment among all the Research Units joining the Research Programme;

- In the third stage the Research Units will make a comparison between the results deriving from the artificial ageing actions and the ones gathered by the natural ageing trials, on the behalf of the whole national-spread Research Team.

The ultimate goal – the assessment of the service life of the tested components – could be attained only when a deal of data concerning different external stressing conditions will be available. The obtained data captured have to be analysed taking into account the various climates and exposures, too.

2.2 Research Unit of Politecnico di Torino

The specific subject of the program is related to <u>Durability of windows and direct tests for re-scaling of wall painting during time.</u> The program aims to focus and tune the validation (by testing) of a durability-purposed assessment methodology.

In the first stage the durability evaluation methodology will be developed for the windows components category. In the second step, for wall paintings, performing the on field natural weathering ageing trials, correlated with laboratory-based artificial ageing trials. The assessing and forecasting procedures could be performed only when precise technical solutions have been envisaged and analysed.

The Durability Evaluation Method is based on the idea that durability is the component's attitude to maintain unchanged during time the performance characteristics. The obtained research results as reference service life will be considered by the designer as an input to be corrected in order to take into account the actual context of the designed building, its use destination and other environmental factors that will influence the design service life prediction for a specific building, considering management and maintenance. In particular the modelling of relations between functional characteristics' decay and technological performances which influence the environmental conditions, will be used at the design stage to simulate the time decay of technological performances (and the variation of indoor conditions) and then to predict the service life during real building design. Besides wear control is required for elements endowed also with repeated movement (linked to decay control).

In future the availability of certified data for different products, (requested for example in the CE marking of construction products) regarding with the reference time behaviour of performance characteristics, will allow the designer to choose on the market the fitted products for the specific work, considering the specific stressing context and the foreseen use conditions. Besides on the base of these data the designer come to a service life prediction and to a maintenance planning, once durability evaluation method has been set up and validated experimentally.

This performance based approach has been developed in coherence with the engineering methods studied at CIB W80 RILEM 175 and discussed at ISO TC 59 SC14. Aging tests on external fitting complex element are particularly regulated by Nordic countries control agencies (Norway, Sweden) but rarely employed in different European countries. A degradation phenomena disaggregating is anyway possible for external fitting specializing them on choices of more critical goods proper for the solution (e.g. polymers decay due to sun radiation, packing elastomers unbrittlement due to the effect of atmospheric ozone).

Contemporary testing actions will be developed in different locations, depending on the various kinds of weathering conditions; such task obviously entails a close partnering attitude and a special commitment amongst all the Research Units joining the Research Programme.

Both the 2nd Engineering Faculty labs of Politecnico di Torino based in Vercelli, and I.T.C. (CNR Institute for construction technology, previously I.C.I.T.E.) in S. Giuliano Milanese are available for cooperation and in particular many experimental results already acquired by I.T.C. could undergo statistical analysis.

2.3 Research Unit of Università di Brescia

The specific subject of the program is related to Methodological development in order to design and to estimate the durability of building components: experimental evaluation of durability for load-bearing external walls. The program aims to focus and tune the validation (by testing) of a durability-purposed assessment methodology in order to settle some criteria for sustainable design. _Such a methodology impinges upon the durability evaluation of technical components belonging to the load-bearing external walls. The assessing and forecasting procedures could be performed only when precise technical solutions have been envisaged and analysed. Indeed, tests will be scheduled and accomplished through laboratory-based artificial ageing trials on reinforced concrete-made external walls (gypsum-based mortar internal plastering and inner layer for thermal insulation made by expanded polystyrene).

Suitable devices and test equipments – in order to performing the scheduled test program – will be made available to the Research Unit by the Polytechnics of Turin (Faculty of Engineering based in Vercelli). The artificial ageing tests will be planned and accomplished through the close partnership of the Turin-based Research Unit. Furthermore, the Research Unit's tasks include investigations about how the technical solutions (previously built) are performing and have been affected over time when a natural ageing process has been triggered.

Contemporary testing actions will be developed in different locations, depending on the various kinds of weathering conditions; such task obviously entails a close partnering attitude and a special commitment amongst all the Research Units joining the Research Programme. Finally, the Research Unit will make a comparison between the results deriving from the artificial ageing actions and the ones gathered by the natural ageing trials, on the behalf of the whole national-spread Research Team. The ultimate goal – the assessment of the service life of the tested components – could be attained only when a deal of data concerning different external stressing conditions.

2.4 Research Unit of Università di Napoli

The specific subject of the program is related to Methodological development and experimental evaluation of durability for horizontal roofs. The program aims to define an experimental method for durability evaluation of a specific technical solution, selected in Horizontal Roof class.

In particular, the Research Unit adopts a program based on tests in external environment and laboratory trials. In the first case, the data will be obtained from sample buildings, chosen for homogeneity, quality and quantity of the available information and reliability of results, and from test pieces (of technological solution) exposed in an external environment. In the second case, accelerate aging tests in laboratory will be worked out, following the ISO 15686 dispositions and the technical code.

At the same time, always following the other Units programs, our Unit of Research propose an application of the NIC Method, developed, in 1998, by Prof. Maurizio Nicolella of the D.IN.E. (Department of Building Engineering) of the University of Naples Federico II.

The method is based on the assumption that the service life of a building component can be estimated in any environmental context, considering the peculiarity of the case as shunting deviation from a mid-normal value obtained on an experimental basis. This reference value (mid-normal Service Life) is corrected, for the specific case, by modifying factors which are associated to every group of agents that influence the service life of the considered building component. The determination of these modifying factors is based on a double approach: empiricist, with data collected on the field;

experimental, with data, carried out from laboratory tests, aimed to evaluate the difference, in the influence of the same degradation agent, in various conditions.

The program, developed for the application of the NIC Method, agrees with the guidelines of the second part of ISO 15686, diverging from it in some points: for example, about the time rescaling. The NIC Method, in fact, searches for a relative value and not for an absolute Service Life and so it isn't important the conversion of the data from laboratory into real temporal parameters.

2.5 Research Unit of Università di Palermo

The specific subject of the program is related to Methodological development and experimental evaluation of durability for pitched roofs. The program aims to define an experimental method for durability evaluation of technical solution selected in Pitched Roof class.

Experiences prove that along the time performance decay of a technical element is inevitable. A study of durability can help us to understand on the one hand how long performance is keeping at a fixed level and on the other the way and the signs of decay. There is no quality absolute value, and neither durability absolute value. There are only values relative to a single building with its employ and stress context. Then acceptability threshold of a technological performance for a technical element is strictly linked with stress agents intensity of a specific context and with its design performance. As building elements are not generally bistable it's necessary to establish decay acceptability threshold on which natural service life of a product can be provided. But that's very difficult because of many factors affecting evaluation: employ model of users, design performance load, stress intensity, etc.; for all these reasons evaluation can be carried out considering technical elements class or a specific technological performance of a class or specific context conditions.

A durability evaluation (as service life performance) can be carried out considering future potential effects in a specific context. These effects prove a performance drop from initial performance (time=0). Then, how can we discover an evaluation criterion to determine a performance drop? In this particular case sandwich panels are tested: shaped panel surfaces can be made of: steel sheets protected by hot galvanizing and organic resin coating, aluminium or stainless steel or copper foils. While, insulating interposed material consists of polyurethane foam, expanded and extruded polisthyrene, phenolic foam, mineral wool.

Experimental tests will be carried out on a double-track:
- natural ageing of samples, through exposition to natural agents in our specific enviromental context,
- accelerated ageing of lab samples subjected to artificial conditions simulating real enviromental context.

It's necessary to analyze functional characteristics (correlated with fulfilment of basis functions determined in relation with requirements) of the product. Parameters of durability evaluation are essentially: reliability trend, failure rate and average period of acceptable working. In this university set a monitoring of technical solutions (investigated also in other university seats with different context conditions) will be also carried out.

2.6 Research Unit of Università di Catania

The specific subject of the program is related to Evaluation of durability of building components in Mediterranean context.. The program aims to develop experimental method for durability evaluation on typical flat covering building systems of Aeolian architecture.

The research program will be developed in continuity with the study till today conducted. Starting from the study of durability conducted on typical flat covering system of Aeolian architecture, the objects of the future research will concern essentially the following points:

- predisposition and execution of a campaign of monitoring on exposed test pieces, exposed to specific climates of the investigated area (Aeolian islands). Such monitoring will concur to correlate the time of decay found in laboratory with the real time of "natural performing decay" (temporal re-scaling).

- updating of the sector standards and, in particular, related to the predisposition of guidelines for European technical fitness and within CEN.

- definition of a methodology for a systematic experimentation aimed at the valuation of the building components durability. The outputs of the first phase of experimentation (currently in course in the laboratories of Leuven University - Belgium) will be compared and made up with those obtained through new equipments. For this reason the realization of a new laboratory of accelerated aging tests is being previewed inside Department of Architecture and Urban planning of Catania.

- determination of the conventional performing limits of the components. This is a particularly complex phase in which it will be tried scientifically to characterize the ending time of reference service life of a building component in function of specific context of environmental stress. As this regards it will be useful to make reference to specific ISO standard (to the so-called engineering methods to estimate the service life in work starting from the reference service life).

The experimentation for valuation of durability could be increased to other technical elements for which it is previewed to use materials of Etna tradition (Etna basalt, for example). The production in the building field, in fact, if on one side tries to promote the circulation in building market of new more and more qualified products, on the other side tries to revalue the use of typical materials of the local context. According this is correct to value the peculiarities of the Etna territory and to try to recover the delay in the modernization of the local manufacturer companies, promoting the spreading of technologically developed and certified products. That is online with the new European Union norms that for all "materials" introduced in the market and marketable to international level expect adequate "technical specifications" that define the quality (of materials and components) and therefore durability.

3. CONCLUSIONS

It's important to notice that the experimental Research Program the involved Italian Universities are developing is pursued in strict coordination, creating a relevant organic experimental research network in order to the diffusion in progress of research results, both in Italy towards building process stakeholders about building components' durability, both abroad towards CIB Working Groups and for pre-standardization aspects towards ISO.
We wish also that it will be possible as Italian Durability Research Network to contribute to the creation of an International Durability Data Base, and to participate to Research Programs on durability evaluation at international level and specifically al European level.

4 REFERENCES

Maggi P.N., B. Daniotti; M.G. Rejna; F. Re Cecconi; G. Rigamonti; A. Jornet; T. Teruzzi; T. Poli 1999,. *Experimental programme to evaluate building elements service life: first results on brickwork*. In International Congress on Durability of Building Materials and Components. Vancouver (Canada).
B. Daniotti, T. Poli, F. Re Cecconi, G. Rigamonti. *Methodology and experimental programme to evaluate building components service life*. CIB World Building Congress 1998. Gavle, Sweden
Maggi P.N., 2002 *Building components' durability, maintenance planning and sustainability* in 9th International Conference on Durability of Building Materials and Components. 17/21 March 2002. Brisbane, Australia.
Maggi P.N., B. Daniotti 2003 "*Management of durability in the building process*". 25-26 June 2003. Milan. Italy 2003
Maggi, P.N., 2004 *Technological design of building components and sustainable construction* World building CIB Congress, May 2004, Toronto, Canada

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Durability rankings for building component service life prediction

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ABSTRACT

As part of a research project for the UK Housing Corporation, Building LifePlans have made available on the internet durability data for over 900 building components — the Construction Durability Database; available at www.componentlife.com. For each component there is a set of durability descriptions which define, through their key durability criteria, the commonly available component options. In all over 3000 durability rankings are defined in the use context of social housing in the UK. For each component, material or assembly the key failure modes and durability issues are presented, which provides a framework for Failure Mode Effect and Criticality Analysis (FMECA) which underlies the durability rankings assessed for each component.

Reference is made to international standards specifically ISO 15686 on service life planning.

This paper describes the process of allocating durability rankings and explores the potential and practice of using durability rankings to estimate service lives. Consideration is given to using durability rankings as reference service lives in accordance with ISO 15686-1 and the developing guidance in other parts of the ISO.

The paper then explores the relationship between durability rankings and estimated service lives and to compare and validate the estimates assigned to components. In a separate commercial survey project component condition, failure modes and expected service for over 200,000 building components were recorded. This project builds on an earlier thesis methodology, but with a dataset of statistically significant size. The rankings and estimated service lives can be compared and there is a strong indication that an adjustment factor of 1.2 can be used to relate durability ranking to surveyor's estimates of residual service life.

KEYWORDS

Durability rankings, internet database, service life estimation

1 INTRODUCTION

As part of a research project for the UK Housing Corporation, Building LifePlans (BLP) have made available on the internet durability data for over 900 building components — the Construction Durability Database (CDD); www.componentlife.com. For each component there is a set of durability descriptions which define, through their key durability criteria, the commonly available component options. In all over 3000 durability rankings are defined. For each component, material or assembly the key failure modes and durability issues are presented, which provides a framework for FMECA and underlies the durability rankings assessed for each component.

Component durability descriptions are an attempt to define those criteria and properties — generally chemical and physical — which influence its durability (defined in ISO BS ISO 15686–1 [2000] as the "capability of a building or its parts to perform its required function over a specified period of time under the influence of the agents anticipated in service")

Durability rankings are a method of listing the component options in a heirarchy based on durability qualities for different specifications estimated to result in a given durability in the context of social housing in the UK and associated experience-based assumptions on use, agents, installation etc. Durability rankings are allocated in the context of the UK construction and environment, based on a building design life of 60 years.

The durability rankings list components on the basis of their durability qualities.

The Housing Corporation project provides core information for providers of social housing in the UK to:

- base component specifications and enable choices to be made in the context of components with equal expected service lives.
- have a durability justification for the use of sustainable materials and components.
- develop maintenance plans and property assessments which will inform business planning, investment options and risk management.
- provide a quantitative measure of best value and performance improvement by incorporating the information in life–cycle costing models and whole–life performance assessments.

2 THE CONSTRUCTION DURABILITY DATABASE

Durability data is listed for fabric and building services components. Durability data which was already in a published format [CAL. 1992, BPG 1999, BPG 2002, BLP 2001] was input into a database and an opportunity taken to cross link durability data with construction industry standard codes.

2.1 Component hierarchy

The durability data is structured hierarchically:

- **Sections**; each representing a major building element
- **Component types**; the principal components comprising a section. For example the section on roofing includes membrane coverings, decking and structural timbers.
- **Component subtypes**; a division of component types typically distinguished by material or function or some visually recognisable difference. For example roof membranes include asphalt and double layer bitumen membranes.

The structure broadly corresponds to the UK industry standard – Uniclass structure.

2.2 Data coding and links

The durability data is coded and cross linked using the principal building industry coding systems in the UK: Uniclass, and BCIS – Building Cost Information Service.
The following Uniclass codes are included:

- **Elements** — major physical parts of buildings; for design and cost information.
- **Work sections** — based on Common Arrangement of Work sections for building works (CAWS); for specifications and bills of quantities based on construction operation.
- **Construction products** — for classifying trade literature, design and technical information relating to construction products.
- **Materials** — for classifying different kinds of material

2.3 Component durability data

Component durability data is focused at the component sub–type level and includes:

2.3.1 Durability descriptions and durability rankings
Formal definitions of durability descriptions and durability rankings are given in the introduction. Durability descriptions distinguish the different 'qualities' of building components commercially available based on properties which influence component durability. Durability descriptions are generic and worded to reflect the key durability criteria such as grade of steel, thickness and type of protective coating.
Each durability description is allocated a durability ranking. The durability rankings are a judgement (with associated risk) used to indicate the relative durability of components. Components with higher durability rankings have physical, chemical or construction properties which are judged to have improved durability for a given environment.

2.3.2 Adjustment factors
The default durability rankings may be adjusted positively or negatively. The adjustment factors are based on the material of the component and its location to account for project-specific conditions.

Where the durability of a component is likely to vary depending on its location in the building for example: internal, external, in private areas or public areas alternative durability rankings are given. This is achieved either; by a separate component type specific to different locations or using an adjustment factor

2.3.3 Maintenance and inspection requirements
The durability data presented is based on the assumption that a certain minimum level of maintenance and inspection is carried out; to good industry practice.
While it may be convenient to define a given frequency for maintenance or inspection activities it is not always possible to be prescriptive. In practice maintenance may be carried out when necessary rather than at a given interval. This approach is increasingly being used where there is a condition based maintenance policy.

2.3.4 Design and detail assumptions
Key design and detailing issues that are important to, or might affect, the durability of components are identified. This section summarizes good practice requirements and gives recommendations for enhancing system life or reliability.

2.3.5 Installation, commissioning and operational assumptions
Key installation, commissioning and operational issues that are important to, or might affect, the durabilty of components are identified. The intention is not to produce a comprehensive guide to the

installation, commissioning and operation of building components, but to identify specific factors of relevance to component durability to avoid premature failure. Further explanation and details on maintenance frequencies may be included.

2.3.6 Key failure modes and Key durability issues
The potential causes of failure of components are identified as well as the factors which influence the durability of the component life. These two sections provide a framework as described by Lair [2003] for FMECA and underly the durability rankings assessed for each component

2.3.7 Notes and References
Useful information relating to component durability which would not naturally fall into the previous sections is included in the notes. The list of references associated with each component include only those which are directly related to the durability data.

3 DURABILITY RANKINGS AND COMPONENT SERVICE LIFE

Durability ranking have been used for more than 12 years in the context of insurance , known as insured lives. Bourke [1996] has noted how the insured life concept was developed as part of a risk management process underpinning a long term latent defects insurance scheme.
A commercial insurance product managed by Building LifePlans BLP [2004] has developed the insured life concept into durability rankings.

3.1 Component durability rankings

Durability rankings are numeric and generally expressed in multiples of 5 which are treated as years. The allocation of a durability ranking is a balance between: distinguishing classes of component by durability and determining a durability ranking value that is proportionate to the expected service life. Durability rankings are generally equivalent to 'insured lives' which have been used in published component life manuals [CAL. 1992, BPG 1999, BPG 2002, BLP 2001]. Insured lives represent cautious assessments of component durability [HAPM 1995] — they represent a time where failure would be considered premature.

3.2 The relationship between durability rankings and service lives

The relationship between service lives durability rankings is a matter of professional interpretation based on evidence from performance of components in practice and laboratory test data. However there is a body of evidence and research [Mayer 2003, Bourke 1996] which shows a predictive relationship between durability rankings and estimated service lives.
In general service lives will be greater then the durability rankings. The critical issue is 'how much greater'. There is no simple answer to this question. Key issues which influence the outcome include: defining service lives, end–of–life condition and making adjustments based on factors which influence service lives

3.2.1 Defining service life and describing end–of–life condition
A clear understanding and definition of the service life is required. Defining service lives and the criteria which describe end of service life is not an easy task. nor is there ever going to be one answer. The service life and end–of–life condition will depend on the policy of the person or organisation making a decision about replacement. To take two extremes:

- The residual service life for components of an historic building may be indefinite where the policy is to preserve and respect the historic fabric of the building.
- On the other hand components used in a retail building may be replaced long before their estimated service is reached if the retail organisation has a policy is in place of replacing

components as soon as the surface appearance deteriorates or a policy of total refurbishment after ten years, say.

In most cases the determination of an estimated service life lies somewhere between these two extremes.

3.2.2 Factors influencing service lives

The factors which influence service life are listed in [BS ISO 15686–1:2000], however the mathematical relationship between the degree of any one factor and service lives is the subject of much research as reported by [Hovde and Moser 2004]. Alternative approaches include:

- [BS ISO 15686–1:2000] advocates a factorial approach to adjusting the reference service life to determe the estimated service lives of components.
- The insurance approach of arithmetic adjustments has been found to work in practice as described by [Bourke 1996 and Mayer & Wornell 1999]

In either case the BLP Construction Durability Database durability rankings provide a useful starting point to provide reference service lives and distinguish components based on durability criteria.

3.3 A model for relating durability rankings to service lives

[BLP or BPG] have been involved in building surveys which have recorded expected service lives and compared these results with durability rankings. Consistently the surveyor's estimated service life is a factor of 1.2 times the durability ranking. An alternative approach is considered for permanent components.

3.3.1 Surveys relating durability rankings to expected service lives

A report by Mayer [2003] where some 220,000 components were surveyed found that the mode value for expected service life is a factor of 1.2 times the durability rank. The median expected service life is 1.3 time the durability rank and the mean expected service life is 2.0 times the durability rank. The range of expected service lives was between 0 and 9.6 times durability ranking. Further research is necessary to disaggregate the data to establish factors for specific components.

An earlier study by Bourke [1996] came to similar conclusions; that for a range of components durability rankings should be multiplied by a factor of 1.2 to arrive at mean expected service lives.

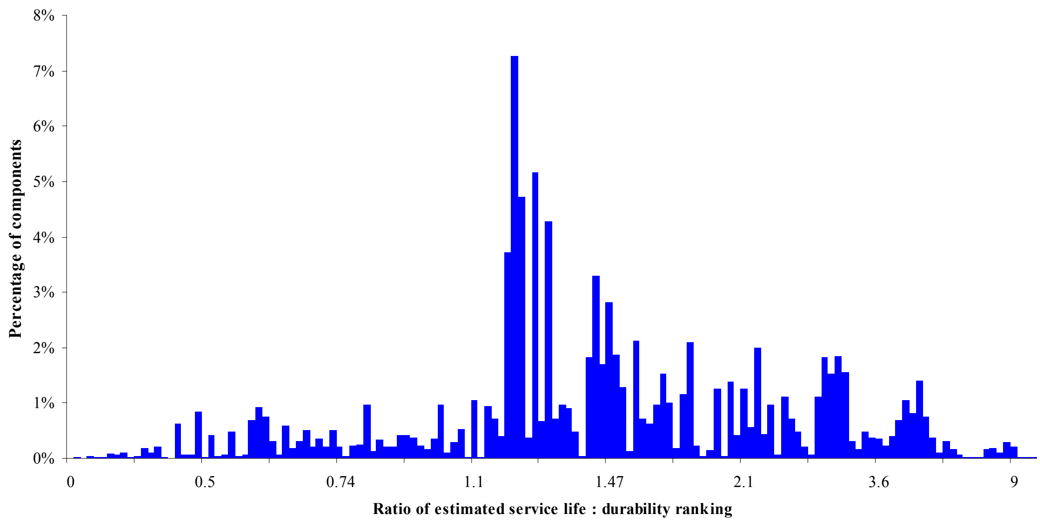

Figure 1 Frequency distribution of estimated service lives to durability rankings

3.3.2 Replaceable and permanent components
Replaceable components would be expected to be replaced some time in the life of the building. Multiplying the 'durability rank' by 1.2 gives an indication of the resultant estimated service life for replaceable components.

Insurance code	Insurance life (years)	Durability ranking (years)	Durability rankings factored by 1.2	Durability rankings factored by 1.2 and rounded
A	35+	40 – 60	48-72	50-70
B	35	35	42	45
C	30	30	36	35
D	25	25	30	30
E	20	20	24	25
F	15	15	18	20
G	10	10	12	15
H	5	5	6	5

Table 1 Summary of durability rankings, insured lives and factored durability rankings

Permanent components are long life components which are generally structural components;expected to last the life of the building; 60 years, 100 years or longer. Where structural components are assessed the durability rankings indicate the relative durability of component alternatives.

Taking for example steel lintels which are allocated durability rankings from 60 years to 15 years. The durability rankings are based on the degree of protection afforded by the coatings and represent times to first maintenance. Where maintenance, if necessary, is carried out the lintels will last the life of the building. The durability rankings represent the risk of failure associated with not carrying out the maintenance should it be required. One interpretation of the lintel durability data is that all mild steel lintels will last 60 or even 100 years but the lintels with lower durability rankings are more likely to require maintenance input to achieve the full design life of the building.

A higher factor than 1.2 may be applied to the durability ranking of structural components to determine an estimate of service life value, particularly where service lives in excess of 60 years are planned. When applying factors to durability rankings to arrive at service lives the factor applied should also consider the maintenance regime. For example: brickwork might be expected to last indefinitely and will require no regular maintenance — but it might need repointing within a 100 year period.

3.4 Applying adjustment factors in practice

With more detailed information a more realistic adjustment may be made when considering service lives. A common adjustment is to decrease the durability ranking where the component is exposed to an industrial, polluted or marine environment, typically an adjustment of minus 5 or 10 years is made to the durability ranking.

Table 2 shows the calculated lives expected for a zinc protective layer 49 microns thick in various environments based on corrosion rates. The zinc coating is typical for a component allocated a durability ranking of 10 in the Construction Durability Database. For example "Profiled mild steel cladding: Hot dip galvanized steel to BS EN 10142 or BS EN 10147, minimum 350g/m^2 zinc coating weight". Applying the 1.2 factor to convert the durability ranking into an expected service life gives 12 years. This corresponds with a minimum service life in an industrial inland or urban coastal environment of 12 years.
An adjustment factor of –5 where the component is located in an industrial environment with high humidity or high salinity coastal environment is confirmed by applying the maximum corrosion rate which would give a minimum service life of 6 years. A lower corrosion rate may be justified and the factor applied to the durability ranking may be larger to achieve a longer service life.

Environment		Average annual zinc corrosion rate (µm/year) based on BS EN ISO 14713:1999		Mild steel coated with 49µm zinc – time to corrosion of zinc protective layer (years)	
		Minimum	Maximum	Maximum	Minimum
C2	Rural	0.1	0.7	490	70
C3	Urban inland or mild coastal	0.7	2	70	25
C4	Industrial inland or urban coastal	2	4	25	12
C5	Industrial with high humidity or high salinity coastal	4	8	12	6

Table 2 Corrosion rates and time to corrosion of protective zinc coatings

An alternative approach to additive or subtractive adjustment factors is to apply ratios which would work well in the case of zinc corrosion rates as the adjustment factor should be? proportion to the zinc corrosion rate. Indeed one of the planned future improvements of the Construction Durability Database is to incorporate experimentally proven adjustment factors where these are available.

4 DEFINING DURABILITY RANKINGS

Durability rankings are determined from an assessment of information about individual components. The key information underlying the durability data concerns:

- Understanding of the component: the materials which make up the component, the process of manufacture and performance data relating to the component.
- Performance of component durability in practice; where modes of failure and lifespan data are examined in the light of experience and published research.
- Factors which influence the durability of components such as design, installation, commissioning and maintenance.

Information from a range of sources is distilled and presented in a common format in the BLP Construction Durability Database.

The information sources used to determine durability rankings have been described by HAPM [1995] and are based on information generally available in the public domain. Broadly listed in order of reliability:

1. International, European and British Standards
2. Authoritative publications and independent certifications
3. Trade associations
4. Manufacturers
5. Practical and professional experience

5 ACKNOWLEDGMENTS

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6 REFERENCES

BLP 2001. Building LifePlans *BLP Building Services Component Life Manual*. Oxford: Blackwell Science. ISBN 0-632-05887-0.

BLP 2004 Building LifePlans *Construction Durability Database* BLP CDD www.componentlife.com

Bourke, K.P. 1996 *Estimating building component durability for the purpose of risk management*. Unpublished Master of Philosophy thesis.

Bourke, P.K. 1996. *Model for insuring durability of building components*. 7DBMC Stockholm, Sweden. E & F N Spon, London, UK.

BPG 1999. The BPG Building Fabric Component Life Manual. London: Spon Press.

BPG 2002. Building Performance Group *Durability of Building Services Plant*. DTI PII funded research Project reference: CI/39/3/5/cc 2036.

BS EN ISO 14713:1999 *Protection against corrosion of iron and steel in structures. Zinc and aluminium coatings*.

BS ISO 15686–1 2000 *Buildings and constructed assets – service life planning General principles*.

CAL 1992. HAPM *Component Life Manual*. London: Spon Press. ISBN 0-419-18360-4.

HAPM 1995 *Technical Note 6: Lifespans of building components* London : Housing Association Property Mutual

Hovde P G and Moser K (2004) *State of the Art Report Factor methods for service life prediction and Engineering design methods for service life prediction* CIB W080 / RILEM 175 SLM

Lair, J., 2003, Failure Mode Effects and Criticality Analysis (FMEA), report submitted to the CIB W80 / RILEM 175-SLM, CIB, Rotterdam, the Netherlands.

Mayer, P D. and Wornell, P. 1999 *Assessing the remaining service life of existing building components for insurance*, 8DBMC, NRC Research Press, Ottawa, Canada, pp1447 – 1456.

Mayer, P.D. 2003 *HAPM claims reserving survey* Commercial report

Temporal quantification method of degradation scenarios based on FMEA

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ABSTRACT

The introduction of sustainable development principles in the construction field and the wish to optimize the functionning of buildings lead us to develop methods that allow to design and manage buildings, during their whole life cycle, by taking into account the objectives of the users. Those methods are mainly based on the apprehension of degradations and failures of the building products. However there is no global capitalization and management method of degradation kinetics of building products.
In this context, we develop a method that allows us to identify and capitalize all the degradation scenarios of a building product and then to quantify their degradation kinetic. Finally, we can generate its multi-performance profiles.

Firstly, the system analysis provides us with a functional model of the building product. Then the Failure Modes and Effects Analysis (FMEA) allows us to determine and capitalize its whole potential degradation scenarios.
The second step is the quantification of the degradation kinetic, i.e. the research of degradation dates and scenario times. First of all, we capitalize all available information on degradation state curves. Then we select and unify a part of this information from various sources, for each degradation phenomenon of the studied building product. After that, we determine the degradation scenario times, from the degradation phenomenon times.
Thirdly, we evaluate the functionnal performances of the building product. We start on the degradation scenarios related to the considered use function. Then, the correlation between the degradation states and the performance levels leads us to evaluate the multi-performance profiles of a building product during its exploitation stage.

The temporal quantification and the multi-performance profile evaluation both provide us with the service life of building products. On the one hand, it corresponds to the quickest degradation scenario, on the other hand it corresponds with the time until the first performance threshold is reached.
Lastly, we present the main forecasted developments and perspectives of this method.

KEYWORDS

Degradation kinetic, degradation scenario, FMEA, multi-performance profile, service life.

1 INTRODUCTION

Since twenty years, the building field tries to integrate sustainable development principles, in order to improve the welfare of present and future generations. Those principles influence on the building design and management, notably the inspection - maintenance - repair, that are expressed as the will to hold the functional performances of buildings. As a matter of fact, to improve the design and the management of buildings, one has to know "how" and "when" buildings and their components (building products) will be degradated or will fail, in order to know "how", "when" and "on which" one should intervene. Without this information, the design and the management are not optimum and generate significant exploitation costs that could be reduced.
In the industry field, a lot of failure analysis methods has been developed; their description can be found in [Desroches *et al.* 2003] and [Zwingelstein 1996]. Jérôme Lair [Lair 2000] selected one of those methods, the Failure Modes and Effects Analysis (FMEA), and adapted it to the building field specificities. We have completed his research ([Talon *et al.* 2003], [Talon *et al.* 2004]) so as to have a progressive view of failure by introducing the notion of degradation and to automate this analysis. Indeed, this method allows us to be exhaustive in the search of degradations and degradation scenarios, but its use application needs a quite long time.
Moreover, there is a significant knowledge of states and kinetics of degradations of the materials used in the building sector. We can extract from a lot of studies the ones on the concrete degradations ([Vu & Stewart 2002], [Taylor 1997] and [Crane 1983]). However, those specific studies are relevant to a material and not to a building product that is generally composed of several materials.
The performance evaluation is a significant thematic of the building field, as the PeBBu (Performance Based Building) network proves it. On the same topic, we can notably notice the Performance Limits Method [Re Cecconi & Iacono 2003] that allows estimating the service life of a component by definition of the limits of its performance characteristics. However, this method is interested in the performance characteristics of a specific function of the building product and consequently not in all the functions of the studied building product.
That are the reasons why we propose a temporal quantification method of degradation scenarios that allows us to evaluate the multi-performance profiles of building products during their exploitation stage, and consequently to determine their service life. We aim this method to be applicable to all building products.

2 IDENTIFICATION AND CHARACTERIZATION OF DEGRADATION SCENARIOS

The aim of this phase is to obtain the degradation chainings that could damage the studied building product during its exploitation stage. The system analysis and next the Failure Modes and Effects Analysis (FMEA) provide us with the list of the degradation scenarios.

2.1 System analysis

The system analysis consists in modelizing the building product behaviour submitted to stresses that could undergo during its exploitation stage.
The first step of this analyis is to build up the structural model. It needs the determination of mecano-physico-chemical and geometrical characteristics of the components that constitute the studied building product, and the determination of links between all those components.
Then we characterize the stresses. This step including the determination of the building product media (for example the inside and the outside media for a front wall) and the identification of their environmental agents (for example: the snow, the wind, the solvent...).
The final step is the functional model building, based on the identification of the functions that are ensured by the building product and its components. It allows us to know the building product behaviour submitted to the previously defined stresses.
In order to aid the user in that phase, we developed both a database of environmental agents and a database of the functions that are ensured by building products.
The functional model is the starting point of the Failure Modes and Effects Analysis.

2.2 Failure Modes and Effects Analysis

The interest of the Failure Modes and Effects Analysis (FMEA) is to obtain the more complete as possible list of degradations and degradation chainings that could damage the building product during its exploitation stage. The FMEA is a risk analysis method, developed during the 1970's and still used in industrial fields, such as spatial, nuclear, medical,...The application of this method to the building field was initiated by Jérôme Lair [Lair 2000] who modified the table format that allows to capitalize the FMEA results.
First of all, the FMEA principle consists in defining the potential degradation modes, the causes (stresses, incompatibilities between materials) and the consequences for each function/component pair. It is to be noticed that all the function/component pairs and the stresses had been identified during the system analysis.
The second step consists in determining the degradation scenarios in an iterative way. The iterative principle, from a step i (step i=0: beginning of the exploitation stage) to a step $i+1$, is to determine if the degradation consequences of the step i could be the origin, that is to say the causes, of the degradations at the step $i+1$.
As far as the capitalization of the results and the automation of the FMEA are concerned, we created databases of material/stress pairs and incompatibilities between materials. Moreover we are developing a database of degradation phenomena.
For more information about the identification and the characterization of degradation scenarios, one can refer to the papers [Talon *et al.* 2003] and [Talon *et al.* 2004] that especially detail and illustrate this step within our global methodology.

3 TEMPORAL QUANTIFICATION OF DEGRADATION SCENARIOS

This paragraph presents the method that gives us the time evaluation of all the degradation scenarios of the studied building product, and then the evaluation of its service life from the knowledge of degradation states. We aim this method to be applicable to all the building products. As a matter of consequences, we present how we capitalize data about the states of degradation, relevant to all the potential degradations of building products. Then the way to exploit this data is exposed, in order to obtain the evaluations of degration scenario times and the service life of building products.

3.1 Capitalization of degradation state data

From the system analysis and the FMEA, the material and stresses associated to each degradation scenario are notably identified.
Consequently, we need to capitalize all the available information relative to the degradation states, for all the building products and for all the potential stresses that we defined in the database of environmental agents (cf. Paragraph 2.1).
The degradation state data may proceed from several origins (fundamental study, ageing testing, feedback, expert judgement,...), so the data may be heterogeneous, imprecise, uncertain and incomplete. That are the reasons why we evaluate the quality of each of the degradation state data.
In order to exploit this rough data, we have to define a common format. In this context, we propose the use of the Weibull laws (presented in [Murthy *et al.* 2004]). The obtained degradation state data are then named formalized data.
The following Fig. 1 schematizes the main stages of capitalization of degradation state data.

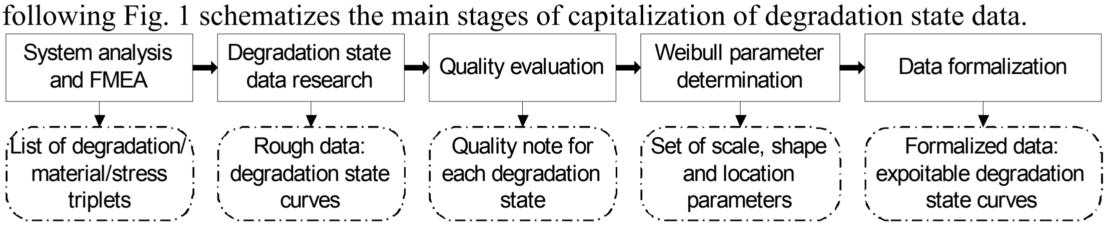


Figure 1. Capitalization principle of degradation state data.

3.2 Exploitation of degradation state data for a specific building product

The exploitation of degradation state data for a specific building product includes the selection and the unification of the formalized data as shown on paragraph 3.1.
The selection of data consists in drawing out all the formalized data that refers to the constitutive materials of the studied building product. For each triplet material/degradation/stresses, all the data could differ, owing to the fact that they come from different origins and that they are not wholly relevant to the product. Indeed, included into two different products the same material should not ensured the same functions. However, it could be damaged by the same degradation, but its kinetics should be different. As a matter of fact, the data will be all the more relevant as they will be associated to the same quadruplet material/degradation/stresses/function(s).
For all the potential degradations, the unification of data aims to provide us one degradation state, from all the available and selected formalized data. The unification of data (detailed in [Lair 2000]) allows us to take into account the quality of the data (with reference to the origin) and the relevance of the data to the studied case.

3.3 Evaluation of degradation scenario times

This phase consists in evaluating the degradation scenario times from the degradation states that have been previously determined. The principle of this evaluation is illustrated on Fig. 2.
One degradation scenario is a succession of degradations. Moreover, we can measure the degradation state - $E_i(t)$ - of a degradation *i* by its degradation rate, τ_i. Consequently, for a given degradation scenario S_j, the transition time $t_{i+1}^{S_j}$ from a degradation *i* to a degradation *i+1* is achieved when the degradation rate associated to the degradation *i+1* is reached.
As a consequence, the degradation scenario time Ts_j is the sum of times separating the degradations of this scenario. This can be formalized as follows :

$$Ts_j = \sum_{i=0}^{n-1} t_{i+1}^{S_j}$$ where *n* corresponds to the number of degradations of the *j* scenario.

Remark: *this result is applicable under the hypothesis that degradation states do not have mutual influence.*

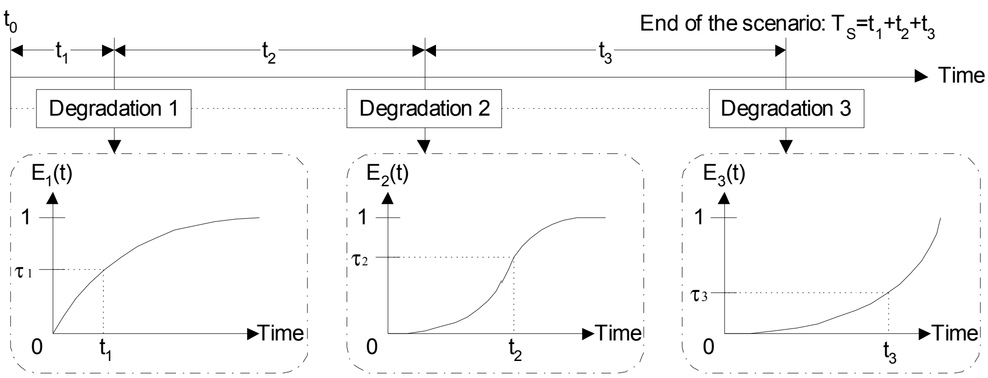


Figure 2. Evaluation principle of degradation scenario times.

3.4 Evaluation of building product service life

The building product service life, here named RSL as defined in the ISO 15686 Standard, can be considered as the time of the quickest degradation scenario. Consequently it is the minimum of the duration of all the determined degradation scenarios as presented in the paragraph 3.3. That can be formalized as follows :

$$RSL = \min_{j \in [1 \dots m]} (Ts_j)$$ where m corresponds to the number of degradation scenarios.

Remark: *this result is applicable only if one considers that each degradation has the same coming out possibility and that the performance thresholds of the use functions are zero (cf. paragraph 4.4).*

It is to be noticed that the determination of the service life of building products, when the degadations coming out possibilities differ ones from the others, is another part of the research – the criticity analysis – not presented in this paper.

The building product service life, as presented in this paragraph, does not take into account the users' waits. Indeed the performance thresholds of the use functions are zero. Consequentlythe way to evaluate the performance levels of the use functions will be presented in the following paragraphs. As a matter of fact, it will be then possible to evaluate a more relevant service life, i.e. when the performance thresholds are not zero.

4 EVALUATION OF MULTI-PERFORMANCE PROFILES

At a specific exploitation stage date, a multi-performance profile of a building product, is the representation of the performance state of all its use functions. The performance level of a use function corresponds to the capacity of the components to ensure this use function. The multi-performance profiles are evaluated from the knowledge of the degradations and of the correlation between degradation states and functional performance levels.

4.1 Selection of degradations

The system analysis and the FMEA provide us with the triplet material/stresses/functions for each degradation. Consequently, one scenario may be associated to several use functions (responding to the needs of the user) or to technical functions (allowing the building product to achieve the use functions). As a matter of consequences, one needs to select the degradations associated to each use function in order to evaluate the functional performance of this use function.

4.2 Relation between degradation states and functional performance levels

The aim of this step, as illustrated on Fig. 3, is to determine the correspondence between the use function' performance levels and the degradation states, for all the use function/degradation pairs previously selected.

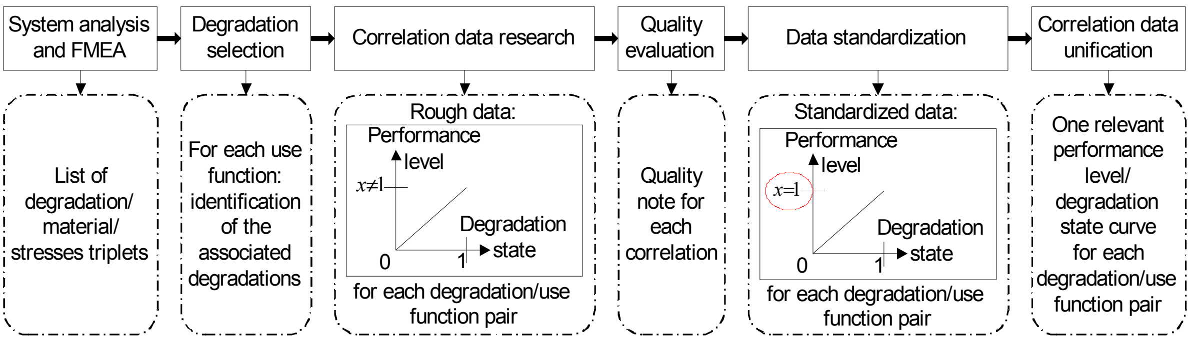


Figure 3. Relation principle between degradation states and functional performance levels.

We aim to capitalize the correlation data in order to facilitate the evaluations of the multi-performance profiles. This correlation data, as well as the degradation state data, may proceed from several origins. The performance levels could notably be evaluated according to different scales. That are the reasons why we evaluate the quality of rough data and standardize it, i.e. we represent them whith the same format in order to facilitate their exploitation.

The exploitation principle of correlation data is based on the unification of all the available data; it is the same as the exploitation principle of degradation state data that is detailed in paragraph 3.1.

4.3 Evaluation of the performance level of a use function at a specific exploitation stage date

The principle of evaluation of the performance level of a use function at a specific exploitation stage date is illustrated on Fig. 4.

The degradation times of the studied building product can be ranged on a time axis. The rectangles on the Fig. 4 represent the degradations and the arrows correspond to their chainings. The grey rectangles schematize the degradations relevant to the studied use function, here named F_1. At a T date, we have to consider the selected degradations that are anterior to T to evaluate the performance level of the F_1 function, here named $\mu^{F_1}(T)$. In the example of the Fig. 4, the degradations D_1 and D_5 are to be taken into account.
The degradation state curves of each degradation are supposed to be known; consequently one can determine their degradation rates, corresponding to this T date ($\tau_{D_1}(T)$ and $\tau_{D_5}(T)$ rates in our example).
Next, the performance levels corresponding to each of those degradation rates can be evaluated. Indeed, the way to correlate the performance levels and the degradation states of each degradation/use function pair has been detailed in paragraph 4.2.
The performance level of the considered use function, at a T date, is the minimum of the performance levels ($\mu_{D_1}^{F_1}(T)$ and $\mu_{D_5}^{F_1}(T)$ performance levels in our example) evaluated for all the considered degradations. That can be formalized as follows :

$$\mu^{F_1}(T) = \min_{i \in [1 \ldots r]} (\mu_{D_i}^{F_1}(T))$$ where r corresponds to the number of degradations to be considered.

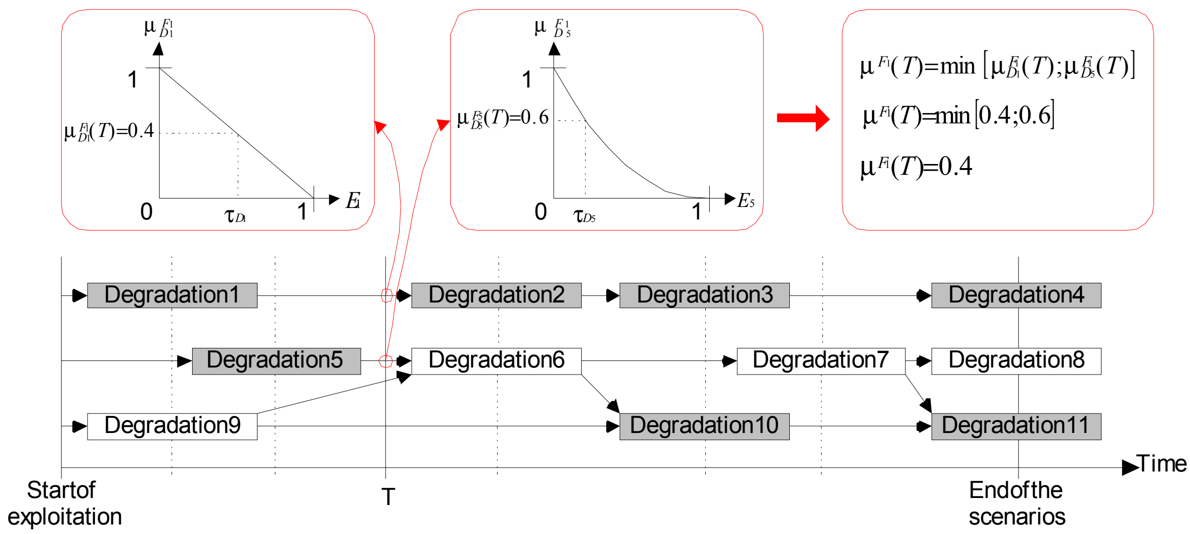


Figure 4. Evaluation principle of the performance level of one use function at a specific exploitation stage date. Grey degradations are the ones relevant to the F_1 use function.

4.4. Evaluation of multi-performance profile at a specific exploitation stage date

The evaluation of a multi-performance profile of a building product at a specific exploitation date, consists in determining the performance levels of all its use functions, at this date. Consequently, it corresponds to the realization of the step described in the paragraph 4.3 for each use function.
Two kinds of building product failures can be distinguished.
On the one hand, a building product fails when one of its use functions is no more ensured. The failure occurs when the performance threshold associated to this use function is reached. For a F_i use function, at a T date, that can be formalized as follows:

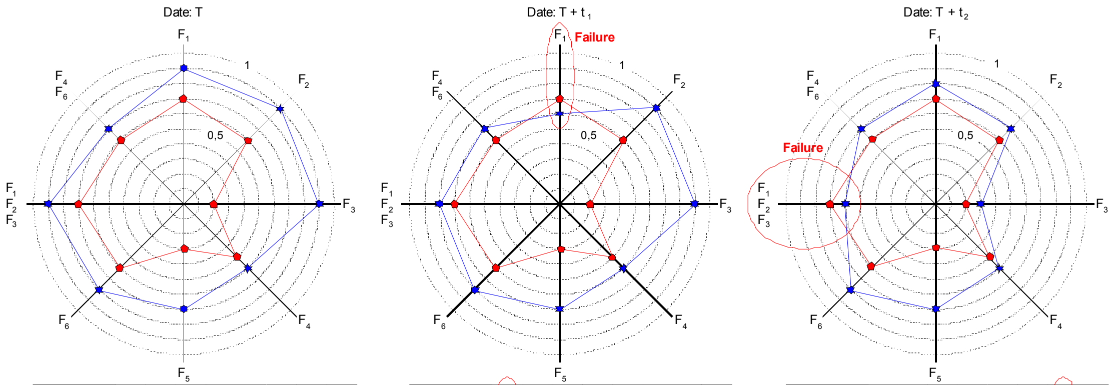
If $\mu^{F_i}(T) > \mu^{F_i}_{seuil}$ **then** there is no failure **otherwise** building product fails.

On the other hand, a building product fails when the performance level of a combination of its use functions (one global performance) is above the global performance threshold of this combination, even if all its use functions are individually ensured. For three F_i, F_j and F_k use functions, at a T date, that can be formalized as follows:

If $f(\mu^{F_i}(T),\mu^{F_j}(T),\mu^{F_k}(T)) > \mu_{seuil}^{[F_i,F_j,F_k]}$ **then** there is no failure **otherwise** building product fails.

Where $f(\mu^{F_i}(T),\mu^{F_j}(T),\mu^{F_k}(T))$ is a standardized function corresponding to the global performance of the F_i, F_j and F_k use functions. It is a combination of their performance levels.

As we have standardized the performance level data of all the use functions (cf. paragraph 4.2), we can have a global vision of the multi-performances of a building product at a specific exploitation stage date, by representing it with a multi-performance rosace. Examples of multi-performance rosaces are presented on Fig. 5.



Use function	F1	F2	F3	F4	F5	F6	F1 F2 F3	F4 F6
Performance level	0.9	0.9	0.9	0.6	0.7	0.8	0.9	0.7
Performance threshold	0.7	0.6	0.2	0.5	0.3	0.6	0.7	0.6

Use function	F1	F2	F3	F4	F5	F6	F1 F2 F3	F4 F6
Performance level	0.6	0.9	0.9	0.6	0.7	0.8	0.8	0.7
Performance threshold	0.7	0.6	0.2	0.5	0.3	0.6	0.7	0.6

Use function	F1	F2	F3	F4	F5	F6	F1 F2 F3	F4 F6
Performance level	0.8	0.7	0.3	0.6	0.7	0.8	0.6	0.7
Performance threshold	0.7	0.6	0.2	0.5	0.3	0.6	0.7	0.6

Figure 5. Multi-performance rosace examples. Red curves represent the performance thresholds, when blue ones are the performance levels. On the left rosace there is no failure, on both the middle and the right ones the building product fails.

4.5 Building product service life taking into account performance levels

In this context of multi-performance profile evaluation, the building product service life may be either the first time one use function performance threshold is reached, or the first time one global performance threshold – relative to a combination of use functions – is reached.

5 DEVELOPMENTS AND PERSPECTIVES

Our contributions on system analysis, on FMEA and on databases facilitate each step from the description of the building product to the more complete as possible FMEA. Nevertheless, it still takes a long-time for manually carrying out them.

That is the reason why the FMEA generation is being automated. We aim this software to be an FMEA realization aid, during the achievement of a “classical” FMEA by the experts of the building product field.

Moreover, this paper was focused on the temporal quantification of degradation scenarios of building products, but we are also extending this analysis to the building scale.

A main perspective is to propose a method to manage the muli-performance profiles of the building products. Indeed, at the present time, we are able to evaluate a multi-performance profile at a specific exploitation stage date. The aim of this management method would be to evaluate the influence of the change of use function performance thresholds on the service life of building products.

6 CONCLUSION

We proposed a temporal quantification method of degradation scenarios, applicable to building products, during their exploitation stage. This method provides us with the degradation scenario times, the multi-performance profiles of the building products at each date of their exploitation stage and their service life.

Firstly, the temporal quantification method is based on the identification and characterization of degradation scenarios, which are obtained by a system analysis and a FMEA.

Then, the capitalization, the formalization and the unification of degradation state data allow us to evaluate degradation scenario times and service life of building products. This service life corresponds to the quickest degradation scenario without taking into account the performance thresholds of use functions.

Finally, we are able to evaluate multi-performance profiles by correlating degradation states to functional performance levels and product service life including users' waits on performance levels.

This method provides the building actors with a precious help for building product design, by identifying the causes and the consequences of potential degradations, and building product management, by knowing "when" and "how" the building products could fail.

7 REFERENCES

Crane, A.A. 1983, *Corrosion of reinforcement in concrete construction*, Ellis Horwood, London.

Desroches, A. Leroy, A. Vallée, F. 2003, *La gestion des risques – principes et pratiques*, Lavoisier, Paris.

Lair, J. 2000, *Evaluation de la durabilité des systèmes constructifs du bâtiment*, thèse préparée au CSTB et au LERMES.

Murthy, D.N.P. Xie, M. Jiang, R. 2004, *Weibull Models*, Wiley Interscience, U.S.A.

Re Cecconi, F. Iacono, P. 2003, ' "Performance Limits Method" (PLM) A New Way To Service Life Prediction', Proc. International Workshop on Management of Durability in the Building Process, Milano, Italy, 25 26 June 2003.

Talon, A. Boissier, D. Chevalier, J.-L. Hans, J. 2004, 'A methodological and graphical decision tool for evaluating building component failure', Proc. CIB World Building Congress 2004, Toronto, Canada, 2 7 may 2004.

Talon, A. Boissier, D. Chevalier, J.-L. 2003, 'Failure Mode, Effects and Criticality Analysis: risk analysis methodology for capitalization and use of experience and knowledge on building products degradations', Proc. 2nd International Symposium on Building Pathology, Durability and Rehabilitation, Lisboa, Portugal, 6 8 november 2003.

Taylor, H.F.W. 1997, *Cement chemistry*, Thomas Telford, London.

Vu, K.A.T. Stewart, M.G. 2002, 'Service Life Prediction Of Reinforced Concrete Structures Exposed To Aggressive Environments', Proc. 9th International Conference on Durability of Building Materials and Components, Brisbane, Australia, 17th 21st March 2002.

Zwingelstein, G. 1996, *La maintenance basée sur la fiabilité*, Hermès, Paris.

Real-time simulations of the durability of Insulating Glass Units

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ABSTRACT

The current methodology of reliability modeling and simulations is dominated by qualitative analysis methods. The methods include Failure Modes and Effects Analysis (FMEA), Event Tree diagrams etc. Quantitative analysis is mostly done by measurements in the field and in the laboratory (such as the methodology of Accelerated Ageing Testing). These real-time methods usually require relatively long time of observations and are impractical in terms of providing fast feedback in the design process.

In recent times, the availability of significant and inexpensive computational power has made it possible to consider real-time simulations of the relevant physical processes as major tools in the design and engineering of systems. In this paper, a computational simulations based methodology for quantitative analysis of durability (reliability) of Insulated Glass Units (IGU), is presented.

The physical model of IGU is given by a set of coupled differential equations. Thermal, structural and mass diffusion models are solved simultaneously for a given time period and for a given time step. The simulations are real-time and, with a proper choice of time-step unit, can provide results equivalent to extremely long field observations. The failure modes are identified and incorporated in the model. The results of the simulations are subject to life data analysis. Right censored life data analysis is used as a natural choice for real-time simulations.

The possibility of utilization of real-time simulations for FMEA is discussed. Real-time simulations can be used selectively on separate units of the system. This way, the probabilities of failure of separate units can be estimated and can be incorporated in the FMEA. Real time simulations therefore can provide a method for obtaining additional quantitative accuracy in the qualitative methodology of FMEA.

KEYWORDS

Insulating Glass Units, Real-Time Durability Simulations, Event-Tree Diagrams, Life-Data Analysis.

1. INTRODUCTION

The durability of Insulating Glass Units (IGU) can vary significantly depending on the climate conditions they are exposed during their life. An accurate prediction of the life-time of an IGU under certain climate conditions is therefore essential when choosing the optimal IGU in a geographic area. The life-time of an IGU cannot be tested in the field, simply because there are too many possible climatic conditions to which the IGU would be exposed.

The climatic conditions can on the other hand be easily simulated by using available climate data i.e. a Typical Meteorological Year. In this paper we describe a model for real time simulation of the life time of an IGU realized in the simulation package **SealSim** [Velthuis *et al*. 2004]. The IGU is represented by a physical model that incorporates its permeation, thermal and structural behavior. The external conditions are taken from field observations compiled in a meteorological database. The results from real-time simulations are in principle subject to the same kind of analysis as the results from any field observations. The real-time simulations, however, deal with a finished product. Their usage in the design of a novel product is limited to simulations of a completed design solution. Traditionally, the actuarial approach of Failure Modes and Effects Analysis (FMEA) is used to identify the possible improvements in the design that can lead to higher durability of the product [Rausand & Hoyland 2004]. These FMEA methods unfortunately lack the quantitative accuracy of the real-time approach. In this paper we propose an approach of integrating the real-time simulations with the FMEA methods with the purpose of obtaining quantitative methods for design improvement.

In chapter 2 of this paper we describe the two classes of IGUs considered. In chapter 3 we explain the physical model representing the IGU. In chapter 4 we propose integration of the real-time simulations with the FMEA methods. In chapter 5 we conclude.

2. INSULATING GLASS UNITS

In this paper, durability of an IGU is considered. Several design solutions for the IGUs are currently available on the market. Most of the design solutions can be classified according to few design classes. In this paper we consider two general classes of IGUs: Homogenous Spacer System and Box Spacer System. The Homogenous Spacer System typically consists of two homogenous layers (spacer materials), representing structural and vapor diffusion layers. Typical spacers in this design class are Thermoplastic spacers, foam spacers (e.g., Super Spacer®, Edgetech), etc. The schematic representation of this spacer system design class is shown on Fig.1. The two glass panes are separated by two separate sealant layers. Both sealants are of polymer origin. The Inner Sealant (also called Primary Sealant) has favorable water vapor barrier properties, but usually very poor structural properties. In order to ensure structural stability of the IGU the spacer system is structurally enforced by the outer sealant (aka secondary sealant). The outer sealant has good structural strength properties, but usually is a very poor water vapor barrier. Together, both sealants provide moisture vapor transmission resistance (MVTR) and structural strength of the IGU. In addition to the sealant materials, a desiccant is embedded in the primary sealant. The role of the desiccant is to capture any water vapor that diffuses through the primary sealant.

The Box Spacer System is representative of hollow bar spacer designs, like metal (i.e. aluminum, stainless steel), plastic, etc. spacers. In addition to the primary and secondary sealant, this system includes a hollow bar, typically a metal. The desiccant in this case is placed inside the hollow bar. The primary role of the metallic box is to provide the stiffness for separation of the panes.

3. PHYSICAL MODELS AND FAILURE MODES OF THE INSULATING GLASS UNITS

The physical model of the IGU in **SealSim** is given by a set of three sub-models: Permeation sub-model, Thermal sub-model and Structural sub-model.

3.1 Permeation model

One of the most significant modes of failure of the IGU is condensation of water on the inner sides of the glass panes. In that case the unit fails both due to limited visibility through the unit and a failure of the isolation properties of the IGU. Condensation of water will occur due to failure of the sealant.

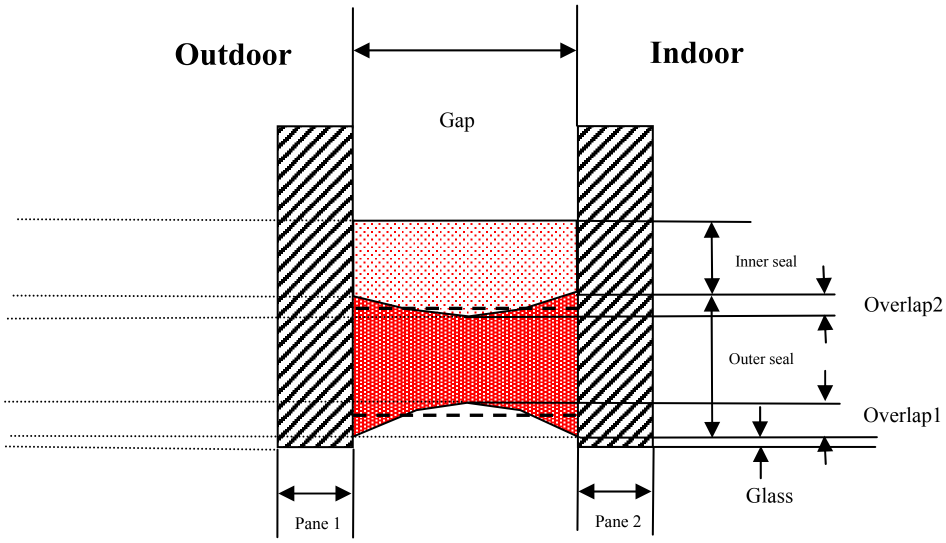


Figure 1. Thermoplastic Spacer System IGU.

Given the fact that the secondary sealant is typically a poor water vapor barrier, the most significant mode of failure is due to water permeation in the primary sealant and water overloading of the desiccant that is mixed with the primary sealant

The main channels of water vapor transport through the sealant into the gas space of the IGU are:

- Permeation of water vapor through the secondary sealant
- Permeation of water vapor through the primary sealant combined with desiccant absorption/desorption.
- Transport and absorption/desorption of water vapor in the (porous) desiccant beads present in the spacer bar.

Simultaneous to water transport, transport and absorption/desorption of other gases takes place, i.e. loss of fill gases such as argon. These are governed by similar equations, but with different transport coefficients. The general equation for one-dimensional spatial, time dependent diffusion of a gas through a polymer slab, mixed with desiccant is modeled by the diffusion equation [Velthuis *et al.* 2004], [Toshima, 1992]:

$$v_p \frac{d\,c_{p,i}}{dt} + (1-v_p)\cdot\frac{d\,c_{d,i}}{dt} = \frac{v_p}{\tau}\cdot\frac{d}{dx}\left(D_i\cdot\frac{dc_{p,i}}{dx}\right)$$

where ‘t’ denotes time, ‘x’ the spatial coordinate, ‘i’ the specific gas involved, ‘v_p’ the volumetric polymer fraction. In regions where no desiccant is present, as in case of the secondary sealant, v_p=1. The concentration of gas ‘i’ soluted in the polymer material is assumed to be proportional to the (partial) gas pressure ‘p_i ’ of gas ‘i’ [Pa] according to Henry’s law: $c_{p,i} = S_i \cdot p_i \quad [kg\ gas_i/m^3\ polymer]$

where $S_i \quad [kg\ gas_i/m^3\ polymer/Pa]$ is the solubility of gas ‘i’ in the polymer material. The permeation coefficient ‘P_i’of gas ‘i’ in the polymer is defined by:

$$P_i = D_i \cdot S_i \quad [\frac{m^2}{s}\,\frac{kg\ gas_i}{m^3\ polymer}\cdot\frac{1}{Pa}]$$

where $D_i \quad [m^2/s]$ is the diffusion coefficient of gas ‘i’ in the polymer.

For simplicity it is assumed that the solubilities, diffusion coefficients and permeation constants of the gases are independent of each other, but are exponential functions of temperature that are different for the primary and secondary sealant. The absorbtion of multiple gases by the dessicant is assumed to be governed by the (LRC) Loading Ratio Correlation, an extension of the Langmuir isotherm for a single gases acording to:

$$c_{a,i} = c_{\max,i} \frac{b_i \cdot p_i}{1 + \sum_{all\ gases} b_i \cdot p_i} \quad [kg\ gas_i/m^3\ desiccant]$$

The factor 'b' [1/Pa] determines the shape of the Langmuir soption isotherm. The dessicant in the polymer matrix not only acts as an immobilising agent, but also increases the distance over which diffusion takes in the polymer, which is described by the tortuosity factor τ.

For the description of the diffusion and absorption/desorption of gases in the porous desiccant beads present in the spacer bar, see [Velthuis *et al.* 2004]. The model is one dimensional, and the transport of gases through the secondary sealant is in series with the transport of gases through the primary sealant. The simultaneously coupled equations are solved by considering a finite grid of points in the x direction only. Although an oversimplification, a one dimensional model is expected to give satisfactory results for the simulations of an IGU. A more sophisticated model should incorporate all three or at least two dimensions and take into consideration the effects of the presence of edges and corners in the IGU.

3.2 Thermal model

The thermal model [Curcija 2004] includes three mechanisms for heat transfer through the IGU: conductive, convective and radiative heat transfer. The IGU has two layers of glazing, each characterized by three infra-red (IR) optical properties – the front and back surface emissivities, $\varepsilon_{f,i}$ and $\varepsilon_{b,i}$, and the transmittance τ_i where the index i represents the two layers. The variables considered in the model are the temperatures of the external (front) and internal (back) facing surfaces, $T_{f,i}$ and $T_{b,i}$, plus the radiant heat fluxes leaving the front and back facing surfaces (i.e. the radiosities), $J_{f,i}$ and $J_{b,i}$. In terms of these variables the heat flux across the gap is:

$$q_2 = h_{c,2}[T_{f,2} - T_{b,1}] + J_{f,2} - J_{b,1}$$

where $h_{c,2}$ is the convective heat transfer coefficient in the gap. The thermal conductivity for each glazing layer is governed by the equation:

$$T_{b,i} - T_{f,i} = \frac{t_{g,i}}{2k_{g,i}}[q_{i+1} + q_i]$$

where $k_{g,i}$ is the conductivity coefficient and $t_{g,i}$ is the thickness of the glazing layer. The solution is obtained by solving the following system of equations:

$$q_1 = S_1 + q_2 \ ; \quad J_{f1} = \varepsilon_{f,1}\sigma T_{f1}^4 + \tau_1 J_{f,2};$$

$$J_{b,1} = \varepsilon_{b,1}\sigma T_{b,1}^4 + \rho_{b,1} J_{f,2}; \qquad T_{b,1} - T_{f1} = \frac{t_{g,1}}{2k_{g,1}}[2q_2 + S_1];$$

for the first glazing layer and:

$$q_2 = S_2 + q_3; \quad J_{f,2} = \varepsilon_{f,2}\sigma T_{f,2}^4 + \rho_{f,2} J_{b,1};$$

$$J_{b,2} = \varepsilon_{b,2}\sigma T_{b,2}^4 + \tau_2 J_{b,1}; \quad T_{b,2} - T_{f,2} = \frac{t_{g,2}}{2k_{g,2}}[2q_3 + S_2];$$

for the second glazing layer. The expression σT^4 is the black emissive power. The system of equations is not linear due to the presence of the σT^4 factor but they can be transformed into a linear system by introducing the black emissive power as a variable instead of the temperature.

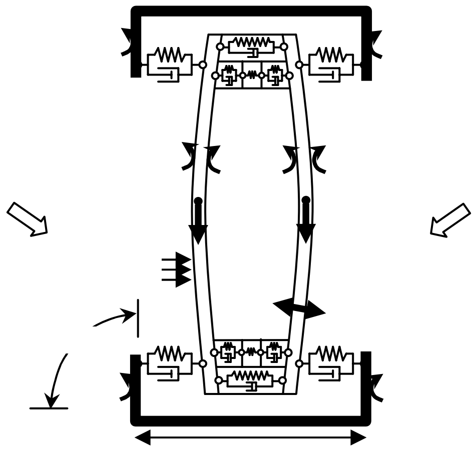


Fig 2. Structural model of the IGU, in this case Box Spacer.

3.3 Structural model

The structural model [Velthuis *et al*. 2004] of the IGU system is shown on Fig. 3. Linear elastic deformation of the glass panes is assumed. The visco-elastic behavior of each polymer element of the system is modeled as a damped oscillator, governed by an equation of the form:

$$B\dot{u} + Ku = f$$

where u is the displacement vectors, B and K are the damping and stiffness matrices and f is the load vector. The polymer material properties are highly dependent functions of temperature. The displacements are related to volume change, which in turn relates to the pressure in the IGU. Note that the volume of the gas space also changes due to temperature effects, that is gas expansion/contraction, and due to gas permeation effects, which are incorporated into the model.

3.4 Model integration and failure modes

The three sub-models are solved simultaneously during the simulation of the IGU life-time. The models are coupled through the common variables: The temperature is common variable in both the permeation and the thermal model, while the displacement in the structural model is related to the volume inside the IGU and therefore the pressure which in turn is a variable in the permeation model. The boundary conditions for the model are given by a database of Typical Meteorological Year on an hourly base. The boundary conditions of the simulation are therefore stochastic model of the real climatic conditions in the field. A single simulation run lasts until a failure of the IGU occurs or until the run is stopped by the user. A set of failure modes is defined. Each failure mode can be redefined by the user by setting the limits of tolerance. The set of failure modes include: condensation of water in the IGU, U-factor exceeds user's limit, average distance between panes drops below the specified limit, cohesive stress exceeds set limit, etc.

4. INTEGRATION OF THE PHYSICAL METHODS (REAL-TIME SIMULATIONS) AND THE ACTUARIAL APPROACHES (FMEA)

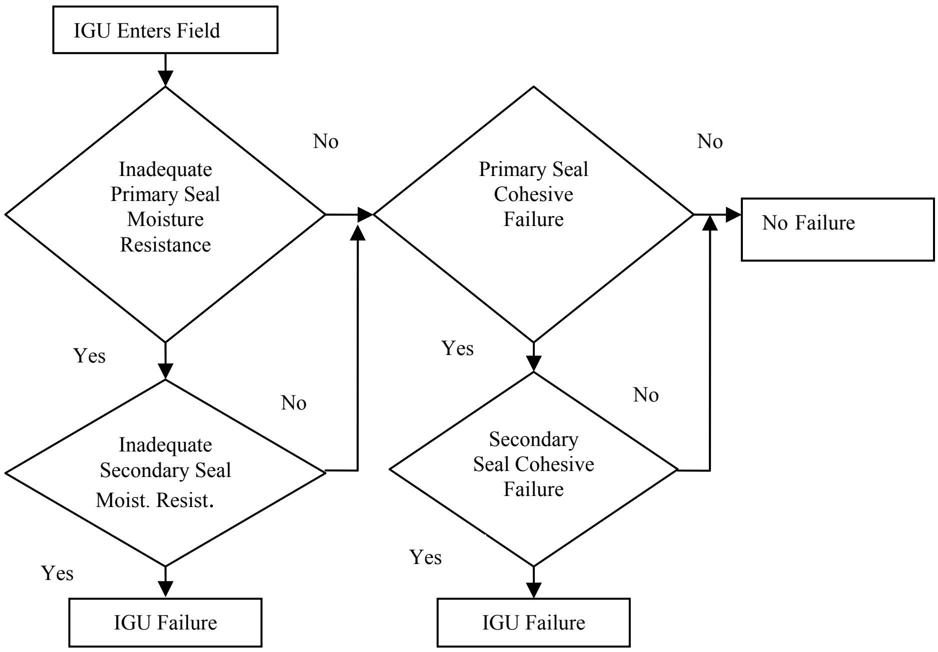
4.1 FMEA

So far we have described a physical method for prediction of IGU durability. Although the physical method provides an accurate and realistic estimation of the life-time of the unit, it does very little in terms of identifying the possible ways of improvement of the IGU design with the purpose of producing more durable and reliable units. Traditionally, during the design of the system several actuarial approaches are used for reliability analysis of the particular design. The most commonly used approach, Failure Modes and Effects Analysis (FMEA) has the purpose of identification of design changes that can lead to the greatest durability enhancement. Its downside, however, is the fact that it is not a quantitative analysis of the system's durability. Once the failure modes are identified by the FMEA, they are organized in a block diagram that reveals the cause and effect relationships between the failures of the parts of the system. In this paper we consider the Event Tree diagrams, although other methods can be used. The Event Tree diagrams provide a simple but efficient way of integration of the qualitative FMEA and the real-time simulations methods.

Figure 3. Example of an Event Tree diagram for an IGU.

4.2 Event Tree Analysis

An example of a typical actuarial method is the Event Tree Analysis. This method provides insight



into all of the possible sequences of events that may lead to the failure of the system. An example of an Event Tree for the Box Spacer IGU is shown on Fig. 3. It is a network of failure events resulting in failure or non-failure of the entire system. Each failure event is represented by a decision block. The decision in each block is done according to a probability of failure assigned to each decision block. A series of events leads to the final failure mode. The main purpose of the Event Tree diagrams is to provide the designer of the system a detailed insight in the all possible scenarios for failure. The designer can therefore easily identify the "weak spots" in the design and concentrate on their improvement. In order to perform quantitative event tree analysis one must have a full knowledge of the probabilities of occurrence for each of the events. These probabilities can be obtained from statistical analysis of field data. This approach however has little value in the design of a novel IGU when the relevant field data does not exist. Also, the field data usually records failure of the entire system with very little knowledge about the series of events that leads to the failure of the entire system.

Instead of field measurements, a realistic physical model of a novel design, such as the one described above and implemented in the **SealSim** package, can be used to simulate its behavior under field conditions. Real-time simulations can be used selectively on separate units of the system. This way, the probabilities of failure of separate units ("unitized simulations") can be estimated and incorporated into the Event Tree analysis. The software package **SealSim** is organized in a way that the user can set the failure criteria and tolerance for each failure mode. In the language of Event Tree diagrams the user can fix the decision outcome in some of the decision blocks along a branch in the Event Tree. The real-time simulation is therefore forced along a certain branch in the event tree with only one of the decision blocks being really active. The relevant probability of failure for the isolated active block in that case equals the probability for failure of the entire system in this particular real-time simulation. I the next section we will describe a general methodology for extraction of failure probabilities from the analysis of the simulated life data.

4.3 Simulated Life Data Analysis

From a point of view of life data analysis there is no crucial difference between field observations and computational simulations of the system. The field observations are typically done on a sample of (more or less) identical systems. The life time of each system is recorded and life data analysis is performed on the set of life times. In a similar manner, real time simulations should be repeated several times until sufficient statistics on the life time of the system is collected. The advantage of real time simulations over field observations is that each simulation starts with known initial conditions so it is safe to assume that each simulation starts at the same point. In the language of life data analysis, simulation data can always be prepared to be right censored.

In this paper we will consider both cases when in the course of a series of simulations each run ends by a failure of the system (complete set of life data) and the case when a number of runs where terminated before the system failure occurred (incomplete set of life data). Our goal is to estimate the life distribution $F(t)$ of the IGU based on a limited number of real time simulations of the system. Furthermore, the goal is to estimate the probabilities of failure for each failure mode in the Event-Tree diagram through the failure rate function $z(t)$ as defined below. The life distribution $F(t)$ of a system [Rausand & Hoyland 2004] is defined as the probability that the system fails within the time interval $(0,t]$. The survivor function is defined as: $R(t)=1-F(t)$. The life distribution is defined in a probabilistic way. In other words, in order to obtain an accurate measurement of life distribution one would need to perform an infinite number of simulations over the period of time $(0,t]$. In reality one can perform only a very limited number of simulations. The goal of life data analysis is to provide an empirical estimate of $F(t)$ from a highly limited set of life data. Let's assume that an IGU has been simulated n times and each simulation was terminated after failure occurred. This way we obtain a complete set of simulated life times: $T_1 \le T_2 \le \ldots \le T_n$.The empirical estimation of the life distribution for this set of n life times can be obtained by: $F_n(t)=i/n$ where i is the number of life-times smaller than t: $T_i \le t \le T_{i+1}$. The survivor function is accordingly estimated as: $R_n(t)=1-F_n(t)$. The empirical life distribution and survivor functions are step-like functions with steps *1/n* at the times of failure. In the case when the set of simulated life times is not complete, i.e. some of the simulations were aborted before a failure occurs, we use the Kaplan-Meier empirical estimator of the survival function: $R_{KM}(t)=\prod_j \frac{n_j-1}{n_j}$ where n_j is the number of surviving systems immediately before time $t=T_j$. In the case when the data set is complete the Kaplan-Meier estimator is identical to $R_n(t)$. In the case of right censored data set, the Kaplan-Meier estimator takes into account the fact that some of the simulations were aborted in the period between two failures and therefore the probability of survival must be modified accordingly. Once the survival function and the life distribution are estimated, a probability measure can be estimated for the observed failure mode. A quantity of interest is the failure rate function $z(t)$ defined with $z(t)\cdot\Delta t$ being the probability that the system will fail in the interval $(t,t+\Delta t]$ if it was functioning at time t. Its relation to the life distribution and the survivor function is: $z(t) = \frac{dF(t)}{dt} \cdot \frac{1}{R(t)}$. The significance of the failure rate function for the Event Tree analysis is in the fact that it quantifies the probability of the system to fail at a given moment. This is exactly the probability according to which a decision will be made in a decision block of the Event Tree diagram. This way, the life data analysis provides a quantitative connection between the Event Three analysis and the results of the real-time simulations.

5. CONCLUSIONS

We presented a physical model for realistic simulations of an Insulated Glass Units. Two types of IGU were considered, each modeled by a set of three sub-models. The permeation, the thermal and the structural physical sub-models were explained. The life-data analysis of the simulations outcomes provides estimates for the durability of the system under given climate conditions. The real-time physical model simulations are necessarily done on the entire system and do not provide insight into the weak points of a given design. As applied to product development, FMEA methodology is traditionally used during the design process in order to eliminate problematic design solutions and ensure robust product attributes including the durability of the final product. The FMEA is however highly qualitative and not as accurate as real-time simulations. We proposed a general methodology for combining the actuarial and physical approaches to durability analysis into an integral methodology. This novel approach will benefit not only the consumer by providing the information for choosing the optimal IGU for his/her climate region but also it will assist the design engineer to find and eliminate potential weak points in the design and provide highly durable and reliable IGU.

6. ACKNOWLEDGMENTS

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7. REFERENCES

Hage, R., Anderson, J., Bender, T., Eastep, M., Fairman, J., Hendrickson, G., Lagos, G., Russell, P., Rozynov, B., Scripter, C., Slough, W. & Wilken, D., 2002, *An Insulating Glass Knowledge Base: Phase I Final Report.* Aspen Research Corporation, USA.

Curcija, C., 2004, *Window 6: Mathematical models and computer algorithms for calculations of thermal performance of glazing systems*. Carli, Inc. Amherst, MA, USA.

LBNL 2003, *WINDOW5 - A PC Program for Analyzing Window Thermal Performance of Fenestration Products*, Berkeley CA: Windows and Daylighting Group, Lawrence Berkeley Laboratory, 2003.

Rausand, M. & Hoyland, A., 2004, *System Reliability Theory: Models, Statistical Methods and Applications*. Wiley-Interscence, Hoboken, New Jersey.

Velthuis, H., Hegen, D & Riemersma, T., 2004, *SealSim 1.1 User Manual*. TNO Institute of Applied Physics, Delft, The Netherlands.

Toshima, N. 1992, “Polymers for Gas Separation”, VCH Publishers, Inc, New York.

An Assessment of Factors Affecting the Service Life of External Paint Finish on Plastered Facades

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ABSTRACT

An increasing number of cases of poor maintenance of plastered and painted facades characterized by early and unexpected onset of defects and damage to the surface have been recently reported in Singapore. With large amounts of money being spent for such maintenance activities, a growing industry demand has been to provide for suitable quantitative tools to predict the onset and occurrence of defects and the ensuing damage to the external paint finish. The prediction of service life of the paint finish is the cornerstone of any such quantitative tool as it helps to establish the planning horizon over which the various costs that arise during the intended lifetime of the paintwork are incurred. This paper presents a comprehensive qualitative assessment of the factors that affect the service life of external paint finish on plastered facades of public housing in Singapore. Visual surveys of 1754 housing blocks around the island were conducted and followed by detailed interview with major paint manufacturers. The influence of major factors identified and grouped under four categories – weather, material composition of paint used, degree of workmanship and building characteristics/attributes, was evaluated and discussed.

KEYWORDS

Service life prediction, defects, paint finish, maintainability.

1 INTRODUCTION

The global construction industry has been witnessing significant increases in building maintenance costs over the years. A significant amount of money, time and resources is being spent on maintenance and repair activities. As an example, the annual maintenance expenditure for residential buildings in Singapore has risen rapidly from about S$11/m^2 to S$38/m^2 over a period of about 10 years [BCA Pilot Study Report 2000]. An important area that has been identified by Construction 21 (a comprehensive review programme of Singapore construction industry) to focus on is to improve maintainability of buildings. In order to achieve overall cost optimization for the building maintenance sector, study and investigation of best practices in this area is currently being undertaken so as to pave the way for their subsequent introduction in the Singapore context [Construction 21, 1999].

Out of the total maintenance cost incurred for a building, a significant proportion is spent on maintenance of facades. The maintenance and upkeep of the façade or external finish of the building is seen as an essential component of any building maintenance programme; being the exterior fabric of the building, it fulfils i) decorative/ aesthetic functions thus epitomizing the image of the building and ii) provides protective functions to the underlying layers of the building [Boussabaine & Kirkham 2004; Hajj & Horner 1998]. However with the advent of time, external finishes like any other building elements undergo deterioration and suffer loss in their decorative and protective functionalities. With age, deteriorating effects gradually and increasingly begin to show on the colour, texture and condition of these external finishes.

Paint is used extensively as an external finish on cement plastered facades and concrete surfaces. Paint finish may not perform in the manner intended and fails to provide the desired functionality for the intended time period possibly due to i) exposure to adverse environmental conditions ii) poor workmanship during application, iii) inadequate quality of the finish material and/or substrate. There is hence a clear need to identify the different nature and influence of these factors in order to arrive at cost effective ways of preventing and dealing with such defects.

This paper examines and assesses the influence of the major factors on the service life of paint finishes. Possible solutions to either eliminate or mitigate their influence in order to ensure that the paint finish lasts for its desired lifespan were evaluated. The paper is part of a research project funded by key industry and institutional players in the building maintenance sector in Singapore. The major objective of this research project is to study and investigate the aspect of façade maintenance in the public housing sector in Singapore being built by the Housing and Development Board (HDB) and develop a quantitative life cycle cost based model for the optimal and reliable prediction, monitoring and maintenance of the costs incurred for efficient façade maintenance. The importance of this sector can be valued in terms of the high percentage (more than 80%) of resident population living in HDB flats in Singapore since the early 1980s [Housing and Development Board, 2002]. An enormous majority of these HDB buildings uses plaster and paint as the external finish; hence the focus of this research initiative is to develop cost effective maintenance strategies for plastered and painted facades. By and large for such maintenance work, the costs incurred are towards cleaning, plastering, painting, minor improvement and repair work to the façade surface.

This paper discusses the significance of the various important factors on the service life of paint finishes derived from the study.

2 RESEARCH METHODOLOGY

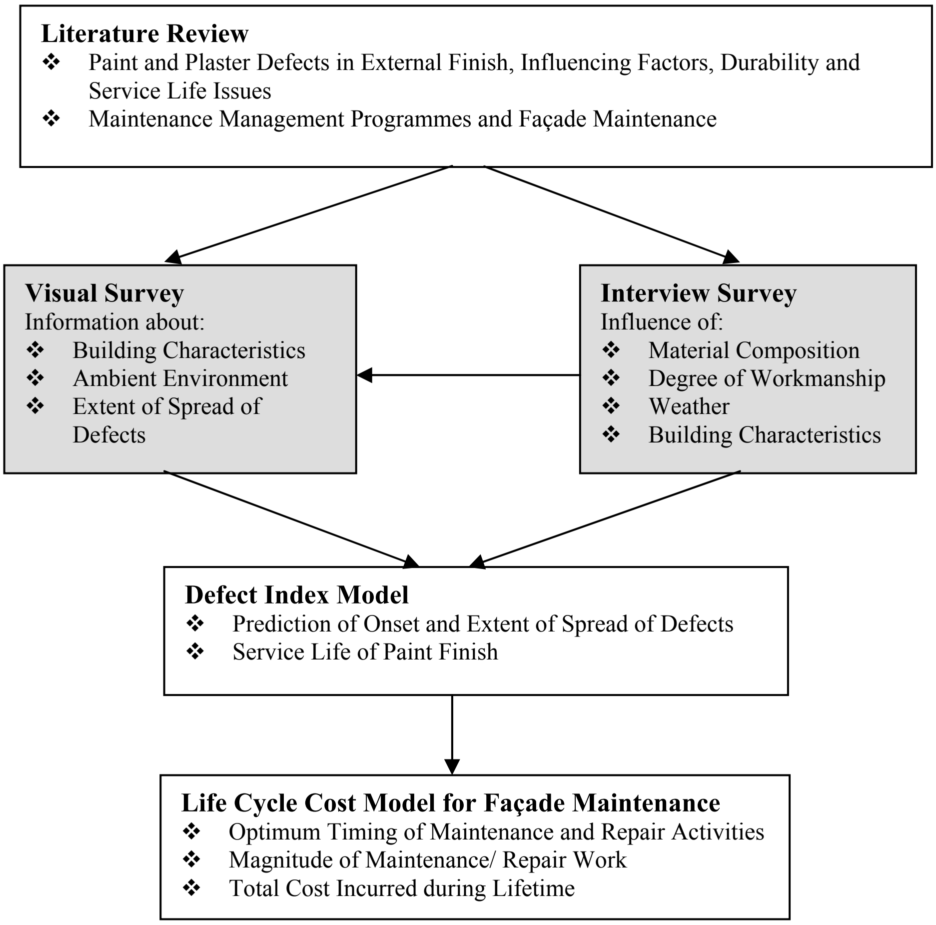

Figure 1 Flow diagram showing research methodology

The research involved several steps which are shown in Figure 1. It started with a literature review focusing on defects occurring in the external paint finish, the factors influencing the occurrence of these defects and related durability and service life issues. The performance requirements stipulated by the local Singapore Standards for external paint finish were studied and their effectiveness evaluated. Different types of maintenance programmes and locally followed maintenance practices were examined with particular focus on façade maintenance programmes and activities. From this review, the various factors affecting and influencing the durability and service life of external paint finish were grouped under four categories: weather, material composition of paint used, degree of workmanship, and building characteristics/attributes. Data collection was done through a two pronged sample survey – a visual survey was used as the primary data collection tool and was supplemented by secondary data collection through interview surveys.

The collected data was used in the development of 'defect index models' for the various defects occurring on the external finish. These models help to establish the timing and extent of spread of these defects and therefore serve as the basis for the development of the life cycle cost based model for façade maintenance. Finally, the developed life cycle cost model for façade maintenance provides i) the optimum interval and hence the timings at which maintenance and repair work would have to be undertaken during the lifetime of the paint and plaster system and ii) the magnitude of maintenance/repair work required at each of these times and the costs incurred for the required work. Such a quantitative model can hence be used for the effective evaluation, monitoring and management of the costs incurred for the efficient maintenance of facades.

2.1 Sampling Characteristics and Methods

2.1.1 Visual Survey

Visual surveys were conducted from a sampling frame comprising of all the apartment blocks under the Singapore public housing sector. These surveys recorded information about:

i) building characteristics pertaining to location of the building, age, time of last repaint, number of storeys, shape of plan view and colour of the external paintwork.
ii) ambient environment of the building which included proximity to vegetation, traffic, industrial area or seaside.
iii) the occurrence of the various defects on the external finish in terms of extent of spread or area of façade affected.

The population size as well as the sampling frame consisted of all the public sector housing (HDB) apartment blocks in Singapore. For purposes of estate maintenance, these apartment blocks have been placed under the control of 16 town councils – these town councils are responsible for the overall maintenance and upkeep of their respective housing estates. Information about the age range to which these blocks belong to was obtained and the blocks within each town council were grouped in age clusters. Due to a large variation in the number of blocks in each age group, disproportionate stratified sampling was adopted to cover all the possible ranges of age groups so as to ensure the data collected in the surveys are accurate.

The visual surveys collected data from all the 16 town councils in order to provide a true representation of the entire population. A total of 1754 apartment blocks were thus surveyed. The property maintenance managers of these town councils as well as the HDB recommended paint manufacturers were interviewed. For reference, a map of Singapore divided into 5 regions based on the locally established Community Development Councils – South West, North West, Central, North East and South East regions is shown in Figure 2. Each town council falls under one of these Community Development Councils. The number of blocks surveyed under each town council is shown in Table 1.

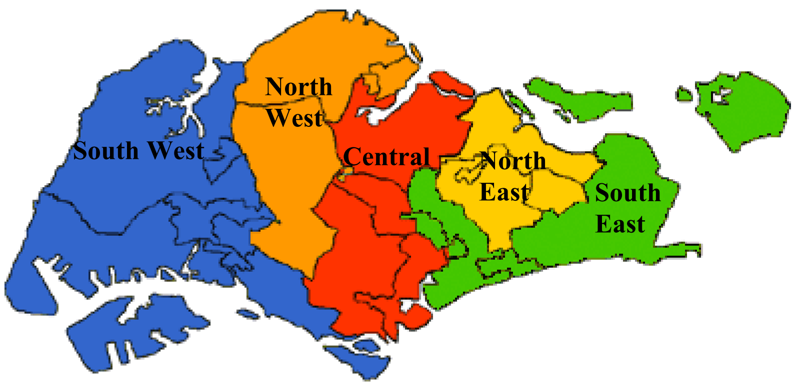

Figure 2 Map of Singapore divided into 5 regions or Community Development Councils (Source: http://www.cdc.org.sg/index.html)

Region / Community Development Council	Town Council	Number of Blocks Surveyed
South West	Hong Kah	212
	Jurong	64
	West Coast – Ayer Rajah	72
North West	Holland – Bukit Panjang	167
	Sembawang	117
Central	Ang Mo Kio	179
	Bishan – Toa Payoh	84
	Jalan Besar	162
	Tanjong Pagar	217
North East	Aljunied	36
	Hougang	25
	Pasir Ris – Punggol	160
	Tampines	156
South East	East Coast	32
	Marine Parade	45
	Potong Pasir	26
TOTAL		**1754**

Table 1 Number of blocks surveyed under town council

2.1.2 Interview Survey

Since the visual surveys are based on visual observation and judgment, it is not possible for them to capture information on the influence of the material composition of the paint that was used and its quality or the expected degree of workmanship during the last painting/ repainting work on the existing condition of the paint finish. Hence in order to account for all the factors falling under the four categories that were identified in the review as influencing the service life of the external finish, interviews with leading paint manufacturers from the local industry were conducted. These interviews attempt to provide information about the influence of all the four factors – something that was not completely captured by the visual surveys and hence play a crucial role in the conceptual development of the 'defect index models' discussed below. Further they also helped to specifically identify and select the variables that needed to be recorded in the visual survey data collection process by providing information about their possible influence and effect on the occurrence of defects on the paint finish.

Interviews were conducted with representatives from the leading paint manufacturers in the local industry in order to gain a better understanding of the various factors affecting the durability and service life of the external paint finish. The idea was to make use of their practical knowledge and experience in obtaining information and gain a better qualitative understanding about the importance and influence of the different factors on the service life of the paint finish. The four categories identified in the literature review are rather broad and encompass a number of factors. Hence through the interviews conducted, the respondents were asked to identify the important considerations or factors under each of the four categories and then report on their influence or role in the occurrence and propagation of defects on the paint finish.

The current practice followed in the public housing sector in Singapore requires paint manufacturers/contractors who are awarded repair and painting/repainting contracts to offer a five to seven year warranty (depending on the type of paint) against common paint defects such as discoloration, peeling, fungus or mould growth, and uneven fading for the paints they apply externally. Hence the sampling frame for the interview surveys was taken to be the list of all paint suppliers executing repairs and painting/repainting works (either by themselves or by engaging the services of suitable contractors) for public housing in Singapore. The choice of paint suppliers for the interviews was based on the contractors registry maintained by the Building and Construction Authority (BCA) – the regulatory body for Singapore's construction industry. This registry is maintained for different building and construction related trades and classifies contractors and suppliers into different levels with each level characterized by a cap on the maximum amount that can be tendered by the contractor. Based on this registry for Construction Related – Repairs and Redecoration (R & R) Works, there are no paint suppliers or contractors eligible for tendering at the highest Level 6 (eligible to tender unlimited); there are five paint suppliers eligible for tendering at Level 5 (eligible to tender upto S$ 10 million) and four paint suppliers eligible for tendering at Level 4 (eligible to tender upto S$ 5 million). Since the above nine suppliers are the "big players" in the painting/repainting market in Singapore, interviews were requested with representatives from each of these companies. The number of respondents interviewed and the response rate are shown in Table 2.

Classification of Contractor	Population Size	Number of Respondents	Response Rate
Level 5 (S$ 10 million)	5	4	80%
Level 4 (S$ 5 million)	4	2	50%

Table 2 Number of respondents and response rate for interview survey

This paper presents a comprehensive qualitative assessment on the influence of the various factors on the service life of the external paint finish based on the information obtained from the surveys in the subsequent sections. The analysis of the quantitative data and the subsequent development of the defect index and life cycle cost models are outside the scope of this paper.

3 HOW CAN SERVICE LIFE OF PAINT FINISH BE DEFINED?

The framework for this study to determine the service life of paint finish is derived based on the major functions of the paint finish to (1) provide an aesthetic and pleasing appearance to the external surface and (2) protect the inner layers/substrates of the building from the deterioration due to different factors sometimes in conjunction with the underlying plaster substrate.

Thus the loss of functionality provided by the paint finish manifests itself in the form of defects on the finish surface and therefore has implications for the service life of the paint finish. These defects can be classified based on which functionality of the paint finish they compromise. For instance, the occurrence of defects such as uneven fading, discolouration implies a loss in appearance and aesthetic functions provided by the finish whereas instances of flaking/peeling of the paint finish indicate a possible loss of protective capability over the affected area, thereby exposing the underlying layers to possible deterioration. Hence any definition of service life needs to look at reduction in protective capabilities as well as possible loss of appearance.

In general, the occurrences of such defects need to be related to i) the extent/coverage of the defect and ii) the degree of seriousness of the defect under question. In the case of the paint finish, the extent/coverage of the defect can be expressed in terms of the percentage of façade area affected; this information can be ascertained through a condition survey of the façade. The quantification of the degree of seriousness is more relevant when the safety and stability of the building is called into question. In the case of the paint finish, even though it performs protective functions, the loss of the protective functionality for the paint finish alone does not have implications for the safety and stability of the building; it only provides possible gateways for deterioration to take place in the underlying layers of the building. Hence in order to quantify the effects of these defects, it is reasonable to assign the same weightage to both types of defects.

From the visual surveys, the findings indicated that for the nine defects viz. crazing, delamination, cracklines, efflorescence, algae growth, chalking, flaking/peeling, uneven discolouration and water seepage; are significantly important to the development of defect index models in establishing the timing and extent of spread of the defects (refer to Table 3; z value greater than critical value of 1.646).

Defect	N	Mean	Z value	Significance	Standard Deviation	Std. Error Mean
Crazing	1754	2.287	68.927	.000	.782	0.019
Chalking	1754	2.056	50.486	.000	.876	0.021
Efflorescence	1754	2.021	54.962	.000	.778	0.019
Cracklines	1754	1.944	53.863	.000	.734	0.018
Uneven discolouration	1754	1.825	45.166	.000	.765	0.018
Algae growth	1754	1.819	45.612	.000	.752	0.018
Faking/peeling	1754	1.663	42.784	.000	.649	0.015
Water seepage	1754	1.425	21.240	.000	.838	0.020
Delamination	1754	1.313	23.119	.000	.567	0.014
Substrate problems	1754	1.012	1.542	.062	.326	0.008
Dirt	1754	1.023	1.473	.071	.654	0.016

Table 3 Z-Test Results for Significance Level of Defects

Using this approach, the service life of paint finish can be related to the percentage of façade area affected by the presence of defects. This value can be compared as a benchmark expressed as the "minimum limit of repair area" which refers to the area that would normally merit attention and activate action in the form of repairs and repainting to the painted façade. This reference value would vary depending on the subjective preferences of different individuals and organizations; since it involves issues related to appearance and aesthetics, it is not possible to fix this limit to a single absolute value. Hence the time period during which the percentage area of the faced affected by the occurrence of defects is less than the benchmark value of the "minimum limit of repair area" can be considered to be the service life of the paint finish.

In the context of the paint finish, it is important that the service life of the paint finish is defined within the scope of defects that occur due to "failure" of the paint. Such failure of the paint is due to inadequacies in material formulation (which result, for instance, in poor resistance to prevailing environmental conditions) or poor degree of workmanship during application. However defects of the same nature could also occur due to "extraneous" factors such as internal water seepage, cracking in substrate layers, etc.; the cause of these defects cannot be blamed on inadequacies of the paint finish itself and hence such defects and the ensuing damage fall outside the scope of the definition of service life of the paint finish.

4 ANALYSIS OF SURVEY RESULTS : FACTORS AFFECTING SERVICE LIFE OF EXTERNAL PAINT FINISH

4.1 Overview

The literature reviews that (a) weather, (b) material composition of paint used, (c) degree of workmanship and (d) building characteristics/attributes; can be identified as influencing the occurrence and propagation of defects on the external paint finish. The occurrence of defects renders the paintwork unsuitable in performing its stipulated functions and consequently affects and influences the durability and service life of paintwork.

This section presents the results of the interview survey with the leading paint manufacturers from the local industry. These interviews attempt to provide information about the influence of the important factors under each of the four categories identified in the review (something that was not completely captured by the visual surveys) and their possible influence and effect on the occurrence of defects on the paint finish.

4.2 Weather

Under the realm of weather conditions, three main factors namely temperature, ultraviolet radiation from sunlight, and moisture are mainly responsible in causing material degradation and hence occurrence of defects and damage to the paint finish. These factors not only act individually to cause degradation but also have a synergistic effect meaning that their combined effects contribute to cause greater degradation of the paint finish. Table 4 presents an overview of these factors and the major considerations and defects associated with these factors.

Climatic Conditions in Singapore
The climate in Singapore places strenuous demands on the performance and durability of all building materials and in particular, paints. Because of its geographical location and maritime exposure, the tropical climate in Singapore is characterized by uniform temperature and pressure, high humidity and abundant rainfall throughout the year. Having a tropical climate with a relatively high amount of rainfall also provides ideal conditions for moisture, the deleterious effects of which have been discussed earlier. Also such conditions provide the ideal conditions for algae growth on external walls.

Factor	Characteristic features/ Considerations	Associated Defects
Temperature	❖ Temperature and temperature fluctuations influence the rate of material weathering ❖ Amount of absorbed radiation depends on colour of paint finish	❖ Physical weathering and cracking
Ultra Violet (UV) Radiation	❖ Amount of UV radiation influenced by latitude, hours exposed to sunlight and angle of exposure of facade ❖ Major concern in the tropics	❖ Discolouration ❖ Chalking
Moisture	❖ Tendency of moisture to impregnate paint systems and affect substrate and underlying layers of building ❖ Hydration and dehydration of finish layer due to repeated wetting and drying leads to surface stress cracking	❖ Blistering ❖ Efflorescence ❖ Microbial growth ❖ Flaking
Wind	❖ Nature of wind affects dispersion of atmospheric pollutants ❖ Wind speed affects concentration of pollutants and hence rate of deterioration and extent of façade areas affected	❖ Deposition of dirt

Table 4 Overview of factors under weather, major considerations and associated defects

4.3 Material Composition of Paint Used

An understanding of the components of paint and their functions is necessary to appreciate the reason behind the occurrence of the defects and prevent such occurrence due to material failure. Paints consist of *pigment(s)*, resin(s)/*binder(s)*, *solvent(s)* and *additive(s)*. Pigments are dry coloring matter generally in the form of an insoluble powder to be mixed with a liquid to produce paint. Binders consist of oils and varnishes that hold the pigment particles together on the surface to form the paint film. Solvents are thinners added to paint or varnish to dilute it, reduce its opacity or viscosity, so as to allow the paint greater workability and for it to spread easily. Additives are different substances added in small quantities to the paint to achieve specified properties.

It is the properties and the makeup of the resin/binder and the pigment that affect the life span of the paint system. The various additives serve the purpose of enhancing the surface finish of the paint layer for different textures and have a minimal influence on the service life of the paint. The pigment used in paintwork can be of two types – organic and inorganic. Organic pigments generally cause fading faster. Bright colours such as red, yellow, orange, etc. are normally obtained from organic pigments. Inorganic pigments give longer durability and fade less easily; less bright colours like blue and grey are made from these pigments. While deciding on the choice of pigments, it is necessary to consider the tradeoff between durability on one hand and aesthetics and style on the other.

By following proper procedures for paint design and formulation, and quality control during manufacture, it is possible to prevent the occurrence of defects that occur due to the failure of the paint material itself. It is hence necessary to have proper standards or specifications which provide guidance and lay down performance requirements to be met by the paint finish during its intended lifetime. If these standards are followed judiciously and thoroughly, it should ensure the lasting of the paint for the desired service life by i) eliminating the occurrence of defects due to inadequacies in paint formulation and manufacture and ii) knowing the effect of deterioration due to factors like weather and the surrounding environment and accordingly providing for the same during the paint formulation process. Secondly in order to ensure full compliance to these standards, it is necessary to establish a systematic mechanism for conducting regular sample testing preferably by an independent quality control agency in order to strictly enforce the adherence to these standards during the process of manufacture of paints. Such measures can ensure that any occurrence of defects due to "paint material or quality failure" are generally less frequent and isolated.

4.4 Degree of Workmanship

A good degree of workmanship plays an important role in preventing or minimizing the occurrence of defects or in prolonging their onset. Paintwork is a labour intensive operation and is hence subject to the quality of the labour and work execution. Site conditions essential for good paint application are adequate protection from both weather and dirt. Also simple tasks like making sure the plaster is sufficiently dry before painting over would reduce the possible occurrence of defects in the future. Some important workmanship related considerations pertinent to paintwork are summarized in Table 5.

Factor	Characteristic features/ Considerations	Associated Defects
Poor surface preparation	❖ Inadequate surface preparation in the form of no treatment of existing loose paint films results in poor adhesion with the underlying layers	❖ Flaking/ Peeling
Insufficient drying time	❖ Application of a subsequent coat on a partially dry coat leads to mixing of the two coats together ❖ Can have implications for durability by not allowing the coats to perform their individual functions effectively	❖ Separation of layers
Surface discontinuities	❖ Disturbance or non-uniformity to be avoided during paint application	❖ Loss in appearance ❖ Peeling
Uneven paint spreading	❖ Coverage in specification provided in terms of consumption rate measured in kg. of paint used per sq. m of area per coat to avoid use of excess or insufficient amount of paint	❖ Loss in appearance and desired thickness of paint finish
Over dilution of paint	❖ Lead to reduction in protective properties and performance degradation of paint finish	❖ Greater risk of weathering ❖ Loss of adhesion with substrate
Poor substrate condition	❖ Unacceptable condition of substrate (plaster or brick) over which the paintwork is applied can be responsible for occurrence of defects on paint finish ❖ Common points to be borne in mind include ensuring sufficient curing time for the underlying cement plaster and moisture entrapped in the substrate layers	❖ Cracking ❖ Efflorescence

Table 5 Overview of factors under workmanship, major considerations and associated defects

4.5 Building Characteristics/ Attributes

The major factors under building characteristics/ attributes that affect the durability and service life of paintwork and their effects are summarized in Table 6.

Factor		Characteristic features/ Considerations	Associated Defects
Orientation		❖ Façades directly exposed to sunlight undergo greater physical weathering leading to chalking of paint. ❖ Facades facing away from direct radiation are comparatively colder and damper providing ideal conditions for algae and other microbial growth ❖ The intensity and duration of sunlight that a surface receives affect the limit of runoff flow, the type of biological stains and hence the pattern of staining. [Chew & Tan 2003]	❖ Chalking ❖ Algae Growth ❖ Staining
Shape of Plan view		❖ Regularity or irregularity of the shape of plan view affects ease of access to the affected areas and hence identification of defects through visual observation or other means, thereby influencing the timing of maintenance and repair and hence the service life. ❖ Greater incidence of cracking in the case of irregular profile possibly due greater difficulty in detecting and repairing crack mapping patterns and their sources	❖ Cracking
Height		❖ Tall buildings are at greater risk to deterioration due to their direct exposure to impacting rain and ultraviolet radiation [Choi 1994] ❖ Wind speed varies with height due to the level of openness as well as the instability of air at higher levels ❖ Costs of maintenance and repair of defects to the façade higher for higher storeys due to additional costs in the form of scaffolding ❖ Safety during work in higher storeys is another concern	❖ Greater rate of weathering and discolouration at higher storeys
Surrounding Environment	Industrial	❖ Concern on account of acid and/or alkali exposure ❖ Downwind location of building from industrial source poses greater risk ❖ Gaseous and particulate exhaust emissions can have a serious effect on paint systems by corroding the surface.	❖ Chemical weathering and etching of paint finish
	Seaside	❖ Possibility of chloride exposure in the form of sea spray or migration and transportation by winds ❖ Harmful effect of salts on hydration products of Portland cement of highly permeable concrete	❖ Chemical weathering and etching of paint finish
	Vegetation	❖ Ambient moisture levels and humidity fluctuations provide good conditions for algae and other microbial growth on paint finish	❖ Microbial growth
	Surrounded by other buildings	❖ Presence of adjacent buildings provides a sheltering effect ❖ May result in slower drying period for façade after it has been wetted, leaving it damp for longer periods and therefore promoting biological staining	❖ Biological Staining ❖ Algae growth
Age		❖ Natural tendency of material to undergo deterioration with time ❖ Condition and serviceability period of the underlying substrate layers of the building has an effect on the exterior paint finish ❖ Gradual loss of protective and other properties of the paint finish itself with age	❖ Cracking ❖ Chalking ❖ Flaking/ Peeling

Table 6 Overview of factors under building characteristics/ attributes, major considerations and associated defects

Building Design and Detailing

The poor performance of the paint finish can also stem from defective building design and detailing and inappropriate construction methods. It is important to give proper attention during the building design phase so that defects arising from such factors can be minimized. Some examples of situations that arise during design and the corresponding considerations that need to be taken to note are presented in Table 7.

Factor	Characteristic features/ Considerations	Associated Defects
Profile of Substrate	❖ Substrate cement plaster surfaces have varying degrees of roughness, texture and porosity that can be obtained depending on composition of mixture, plastering method and skill used ❖ The greater the roughness and porosity of the substrate, the more likely the occurrence of paint defects due to the higher tendency to trap dirt and water	❖ Blistering ❖ Efflorescence ❖ Discolouration
Condensation on Walls	❖ Continuous use of air-conditioning with poor insulation can cause condensation on the external walls ❖ Humidity and differences in external and internal temperatures affect occurrences of such condensation	❖ Algae growth ❖ Loss in appearance
Concrete Gutters	❖ Common for moisture to penetrate through concrete gutters and stain the underside ❖ Good design and workmanship to ensure proper water run-off and drainage and application of proper waterproofing help in curbing future occurrences of this problem	❖ Peeling ❖ Staining
Ledges	❖ Ledges on building facade can cause back splashes by rainwater ❖ If the rainwater is slow to run down or dry ,this leads to creation of a moist surface	❖ Presence of moisture and algae growth
Copings and Cappings	❖ Use of copings prevent occurrences of dirt streaking ❖ Cappings prevent water from retaining or seeping into the top parapet edge	❖ Accumulation of moisture ❖ Algae growth

Table 7 Overview of factors under building design and detailing, major considerations and associated defects

5 SIGNIFICANCE OF FINDINGS

The identification and assessment of the various factors influencing the occurrence of defects on the paint finish provides the crucial "durability" input to the paint formulation and design process; this provides a systematic way of incorporating durability and service life considerations in the design stage itself. In order to keep maintenance costs to a minimum, it is necessary to strive for prevention of any possible occurrence of defects in the first place by eliminating or minimizing the influence of these factors. Whereas it is desirable to achieve such complete prevention of occurrence of defects, it is not possible to completely eliminate their occurrence during the time period the paint and plaster is required to last. This is because all the influencing factors are not controllable. As seen above, the factors falling under the categories pertaining to weather or the surrounding environment of a building can never be under our control and hence can be characterized as "uncontrollable". On the other hand, some form of control can be exerted over factors coming under material composition of paint, degree of workmanship and building design and detailing. Hence a two pronged approach needs to be adopted wherein:

a) the influence of controllable factors under workmanship and composition of the material(s) used on the occurrence of defects is eliminated as far as possible through the adoption of appropriate design practices and quality control during construction and

b) the onset and rate of propagation of the various possible defects due to the influence of uncontrollable factors pertaining to weather and the surrounding environment that might still occur is predicted so that their effect is accorded due attention in the paint formulation and design process and/or appropriate remedial action is taken at the right time during the lifetime of the finish.

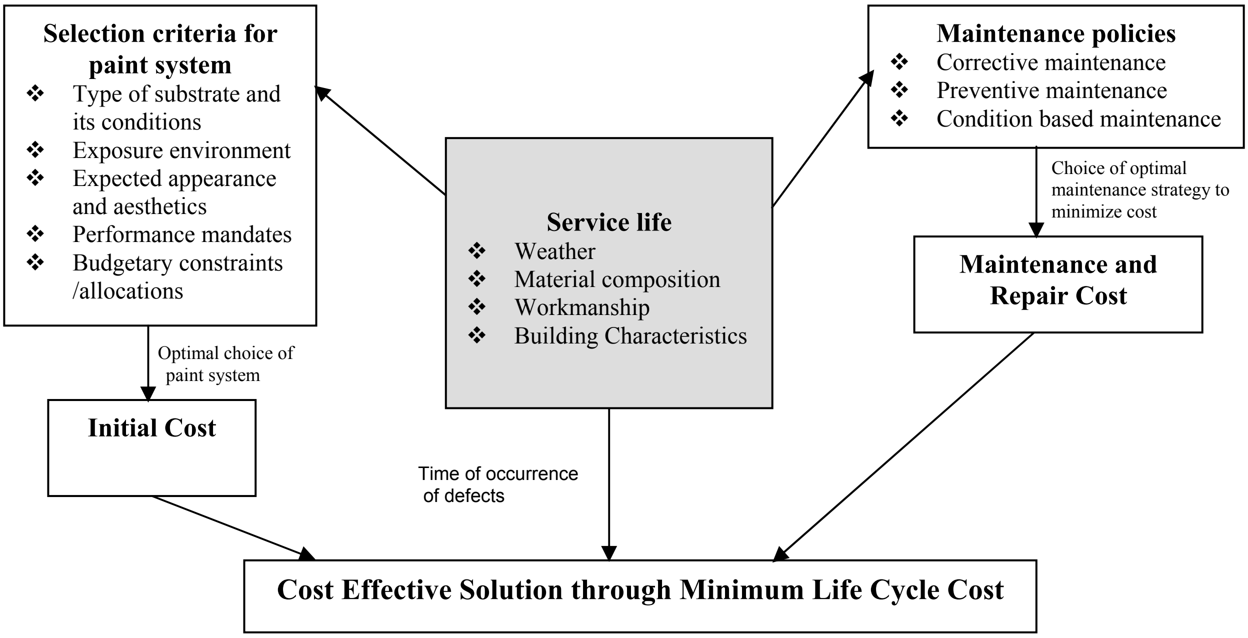

Figure 3 Framework of Life Cycle Cost Model for Façade Maintenance

The identification and assessment of the different factors affecting the service life of the paint finish is an important step in its prediction as it helps to provide for suitable measures to eliminate, minimize or resist their influence. The prediction of service life is the cornerstone of any predictive life cycle cost based model for façade maintenance as it helps to establish the planning horizon over which the various costs that arise during the intended lifetime of the paintwork are incurred. The service life influences both the initial costs as well as the future maintenance and repair costs incurred at different times during the lifetime of the paint finish. This can be clearly seen from Figure 3 which shows the framework of a life cycle cost based model for façade maintenance.

6 CONCLUSION

A comprehensive assessment of the various factors that the service life of external paintwork on plastered facades has been presented in this paper. The major factors that influence the service life of the paint system have been identified and grouped under four categories– weather, material composition of paint used, degree of workmanship and building characteristics/attributes. It is important to understand the distinction between the “controllable” and “uncontrollable” factors and accordingly cater to their effects during the paint formulation and design process as well as during the application process; this would ensure that the desired service life for the paint finish is attained. This is crucial for the subsequent development of any quantitative model that would serve to establish the timing of the various maintenance and repair costs that would need to be incurred to rectify the defects and damage to the paint finish during its intended lifetime.

7 REFERENCES

BCA Pilot Study Report 2000, *Enhancing Maintainability of Buildings,* Building and Construction Authority, Singapore.

Boussabaine, A. & Kirkham, R. 2004, *Whole Life Cycle Costing: Risk and Risk Responses*, Blackwell Publishing, UK.

Chew, M.Y.L., & Tan, P.P. 2003, *Staining of facades,* World Scientific, Singapore.

Choi, E.C.C. 1994, 'Determination of wind-driven rain intensity on building faces', *Journal of Wind Engineering and Industrial Aerodynamics*, **51**, 55-59.

Construction 21 1999, *Reinventing Construction,* Ministry of Manpower and Ministry of National Development, Government of Singapore, Singapore.

Hajj, A. Al & Horner, M.W. 1998, 'Modelling the running costs of buildings', *Construction Management and Economics*, **16**, 459-470.

Housing and Development Board 2002, *HDB Annual Report 2001/2002,* Housing and Development Board, Singapore.

Roy, S.K., Thye, L.B. & Northwood, D.O. 1996, 'The evaluation of paint performance for exterior applications in Singapore's tropical environment', *Building and Environment*, **31**[5], 477-486.

Singapore Standard SS 150 1998, *Specification for Emulsion Paints for Decorative Purposes*, Singapore Productivity and Standards Board, Singapore.

Singapore Standard SS 345 1990, *Specification for Algae Resistant Emulsion Paints for Decorative Purposes*, Singapore Productivity and Standards Board, Singapore.

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TT4-155

ABSTRACT

The endeavour to obtain estimates of durability of components for use in lifecycle assessment or costing and infrastructure and maintenance planning systems is large. The factor method and the reference service life concept provide a very valuable structure, but do not resolve the central dilemma of the need to derive an extensive database of service life. Traditional methods of estimating service life, such as dose functions or degradation models, can play a role in developing this database, however the scale of the problem clearly indicates that individual dose functions cannot be derived for each component in each different local and geographic setting. Thus, a wider range of techniques is required in order to devise reference service life. This paper outlines the approaches being taken in the Cooperative Research Centre for Construction Innovation project to predict reference service life. Approaches include the development of fundamental degradation and microclimate models, the development of a situation-based reasoning 'engine' to vary the 'estimator' of service life, and the development of a database on expert performance (Delphi study). These methods should be viewed as complementary rather than as discrete alternatives. As discussed in the paper, the situation-based reasoning approach in fact has the possibility of encompassing all other methods.

KEYWORDS

corrosion prediction, service life, case-based reasoning, durability, holistic model

1 INTRODUCTION

Service life planning is a design process that seeks to ensure that the service life of a building will equal or exceed its design life. Accurate estimates of the service life of a building are required to address and possibly optimise the lifecycle costs. It may also provide a means of comparing different building options.

ISO 15686 [2000] provides a methodology for forecasting the service life and estimating the timing of necessary maintenance and replacement of components. A forecast of service life should seek to use available data of known quality, account for variability and reduce uncertainty. Relevant data for forecasting includes:

- Data recorded over time.
- Comparison between exposure data and other evidence.
- Experience, feedback from practice and estimates of experts.

2 REFERENCE SERVICE LIFE

ISO 15686 (Clause 9) suggested the factor method as a means of estimating the service life of a particular component or assembly in a specific set of conditions. The factor method is based on a reference service life (RSL), which is defined as the expected service life of a component or assembly situated in a well-defined set of conditions. The factor method also incorporates a series of modifying factors that relate to the specific conditions of the case.

However, the quantity of data required in defining the RSL of dwellings in any given country is extremely large. The possible number of metal components in a simple detached Australian domestic dwelling indicates the extent of this problem. In such a dwelling, there are over 300 metal components, however each component may be commonly fabricated from 2 or 3 materials and coated with 2–3 different coatings, and thus RSL values would be required for in excess of 2000 distinguishable components. Further, these components may be placed in environments which may differ significantly, firstly due to local conditions or arrangements in the dwelling, and secondly due to the external environment.

Traditional methods of estimating service life, such as dose functions or degradation models, can play a role in developing this database, however the scale of the problem clearly indicates that individual dose functions cannot be derived for each component in each different local and geographic setting. Thus, new methods need to be derived to estimate service life and then these methods need to be combined in a rigorous manner.

3 OPEN INNOVATION APPROACH

The question is: how can one build a system that implements the factor method, while taking into account various sources of information such as measured data (both field and laboratory), results of mathematical models, anecdotal evidence from practice and estimates of experts and practitioners? The required system must be able to store, manipulate and compare numerous use–case scenarios.

In this paper, a business model for an R&D organisation proposed by Henry Chesbrough [2004] is adapted. In essence, the model proposes that rather than relying entirely on internal ideas to address an issue, an 'open' approach to innovation leverages internal and external sources of ideas. Note that the Chesbrough model is more than that, but for the purpose of this paper, the above concept should suffice.

How does one go about applying the open innovation approach? The guiding principle is to look at the current problem and try to identify solutions for each component of the problem. These solutions are normally mature or tested technologies and hence no longer considered an innovation.

For instance, the problem of defining reference service life and estimating the service life of a building component is about:

- Operating on use–cases or scenarios.
- Integrating data from different sources.
- Spatial references and operations.
- Rules and representations of expert knowledge and anecdotal evidence.

It is evident from the above list that several mature technologies can be used and integrated to address the issues of defining an RSL and service life estimates. These technologies include:

- Situated case-based reasoning [Liew & Maher 2004].
- Heterogenous databases [Widom 1996].
- Geographic information systems [Cole *et al.*, in press].
- Logic programming [Sterling & Shapiro 1994].

4 SITUATED CASE-BASED REASONING

The principles of case-based reasoning in building design have been developed [Maher 1998]. For example the work of Maher *et al.*]1995] and others provides a model for design reasoning based on the use of a set of previous design experiences represented as design cases. These cases are indexed and retrieved using information about a current design problem and then, through analogical reasoning, a selected case (or set of cases) is adapted until it satisfies the current design specifications and constraints. One aspect of design reasoning that is not addressed by traditional models for case-based reasoning is that design problems are situated. To accommodate the notion of 'situatedness' in design, the basic idea of case-based reasoning is extended to create a model of situated case-based reasoning (situated CBR) as shown in Fig. 1.

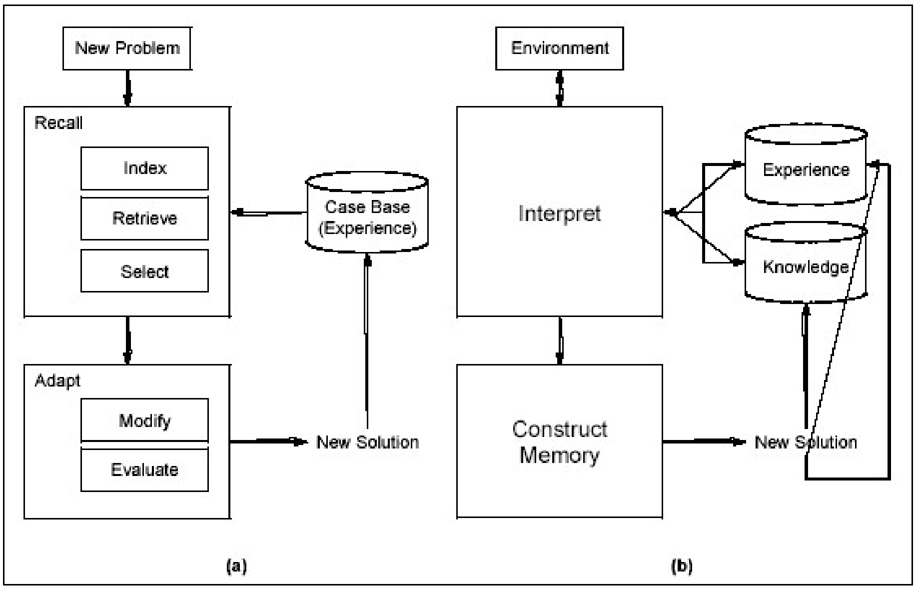

Figure 1. A conventional case-based reasoning (a), and a situated CBR model (b).

In the situated CBR model, instead of focussing on just the design problem and finding a solution to it, emphasis is also given to the environment within which the problem is framed. The model interprets the environment according to the current situation and the problem is framed accordingly. This interpretation is dependent on the current environment, the internal state of the situated CBR system and the interactions between the system and the environment. The internal state of a situated CBR system is defined by its content. This content is made up of individual entities that are classified either as experience or knowledge. Interactions between the system and the environment define different interpretations of the environment according to different interpretations of the selected entities used for memory construction.

A distinctive characteristic of situated CBR is the way its knowledge and experience are understood and used. In CBR, retrieved cases provide a solution or a starting point for case adaptation. In situated CBR, the memory of an experience and/or knowledge (entities) is constructed according to an interpretation of the environment and an interpretation of the selected entities relevant to the problem at hand. Rather than adapt a selected case to new design specifications, the selected entities are interpreted according to the interactions between the system and the environment. These interactions provide a specific view (interpretation) of the relationship between the design specifications and the environment. This view dictates another interpretation of the environment that can introduce new specifications. This 'feedback' loop causes the interpretations of the environment and the selection of experiences and knowledge to occur recursively until a common interpretation is reached. The recursive interpretations of the environment and the selected entities result in new memories as well as new indices to the selected experiences and knowledge to be created. Memories are constructed by:

- Instantiating the parameter values of the selected entities according to the current situation.
- Mapping existing parameters in the selected entities to new ones through an analogical process.
- Restructuring the selected entities according to the current situation.

5 HETEROGENEOUS DATABASE

The required database for the proposed system is actually a federation of three heterogenous databases, as shown in Fig. 2. The first component database is spatially referenced historical data on corrosion derived from CSIRO GIS data on atmospheric corrosion [Trinidad & Cole 2002]. The second component database consists of the synthesised results of a Delphi-based survey study on service life and maintenance of metal components in building assemblies (the Delphi study will be discussed in the next section). The third and final component consists of the database realisation of the 'holistic' corrosion model [Cole *et al.* 2004].

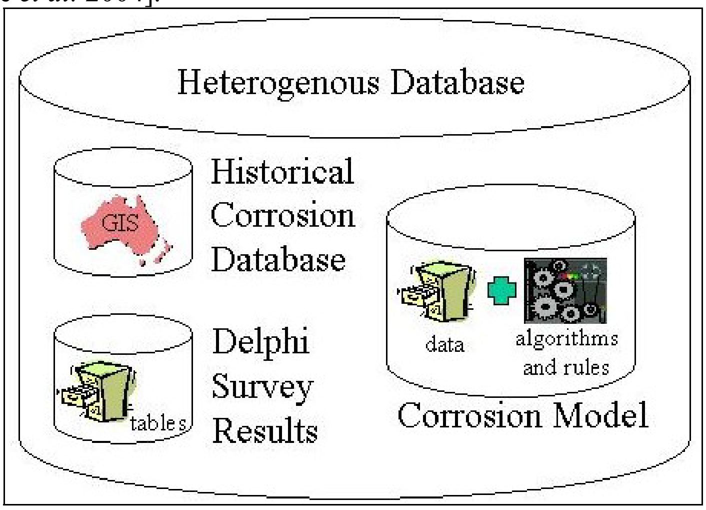

Figure 2. A federation of heterogenous databases; GIS, survey data and model base.

6 DELPHI STUDY

A Delphi survey has been conducted to provide expert opinion on the life of components in buildings. Thirty different components were surveyed, with a range of materials, coatings, environments and failure modes considered. These components were chosen to be representative of a wider range of components in the same building microclimate. The survey included both service life (with and without maintenance) and aesthetic life, and time to first maintenance. It included marine, industrial and benign environments, and covered both commercial and residential buildings. In order to obtain answers to this wide range of question, but still have a survey that could be completed in a reasonable time, the survey was broken into five sections:

- External metal components—residential buildings.
- Internal metal components—residential buildings.
- External metal components—commercial buildings.
- Internal metal components—commercial buildings.
- Metal connectors in buildings.

The survey was conducted in two stages. In the first stage, there were a total of 66 responses, with the number of the responses to each of the survey parts ranging from 9 to 18. The questions were placed in four classes depending on the degree of consensus in responses to the particular question. After the first stage, approximately 80% of questions had a consistent answer from the survey group. In Stage 2, 10% of questions were further investigated, with 75% of these remaining questions then having a consistent answer. The responses to each question were analysed to give a mode (most frequent interval), a mean value and a standard deviation of the mean.

The final database was examined in three ways to determine its accuracy and reliability. These were analyses for internal consistency of the data, for consistency with expected trends based on knowledge of materials performance and environmental severity, and for correlation with existing databases on component performance. In all cases, the Delphi survey data appears reliable.

The possible extensions of the approach are discussed in line with the technical 'success' of the method. However, the study was difficult to carry out owing to difficulties in obtaining answers from possible respondents. Thus, if a larger survey is to be undertaken including all building components, it is recommended that committed respondents be obtained before devising a survey.

7 CBR SYSTEM ARCHITECTURE

The system architecture of the situated CBR for service life prediction is shown in Fig. 3.

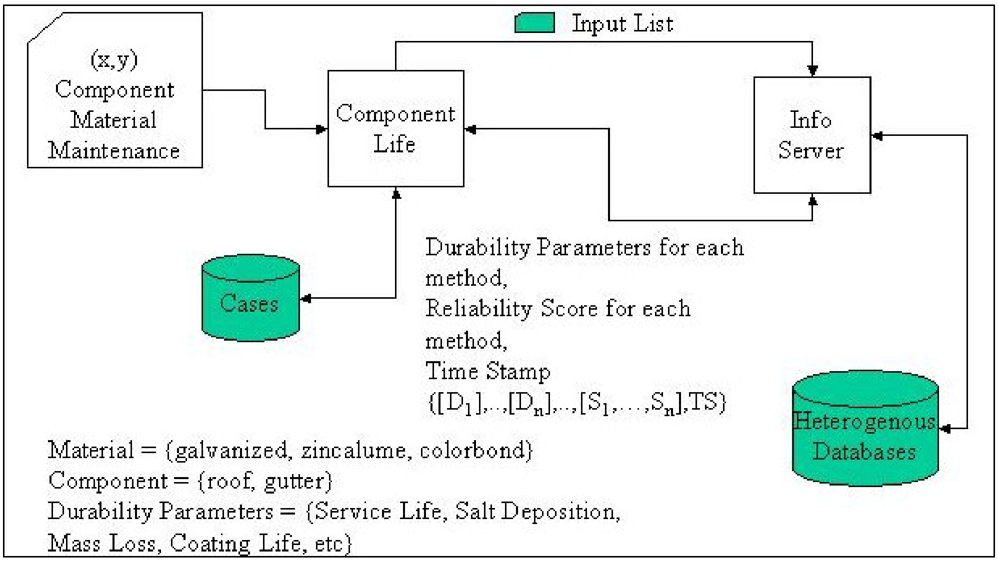

Figure 3. System architecture for a situated CBR on the prediction of service life.

The module 'component life' is the main situated CBR engine. It calculates service life estimates based on previous cases and information from the combined databases. An information server (Prolog application) module serves as an interface between the CBR engine and the combined databases. The information server provides the CBR engine access to the databases and corrosion model through five methods, as shown in Fig. 4. The five methods are:

- *getSalt*—provides access to an historical salt deposition database, and salt transport and deposition model.
- *getToW*—provides access to a 'time of wetness' database derived from climate databases.
- *getModel*—provides access to the holistic corrosion model.
- *getData*—provides access to historical corrosion data.
- *getSurvey*—provides access to the results of the Delphi study.

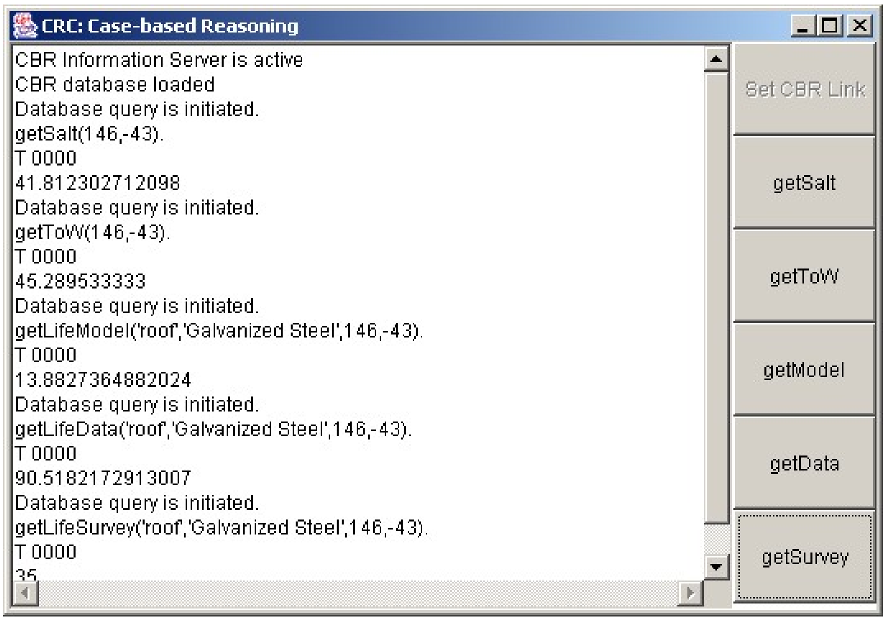

Figure 4. Java client illustrating the five interfaces to the information server.

Both the historical corrosion and holistic corrosion model databases are spatially referenced. On the other hand, the Delphi survey results are not referenced spatially. Hence, an environment classification module is needed, as shown in Fig. 5. This module is used to classify a given location (e.g. longitude–latitude coordinates or official place names of a geography gazette).

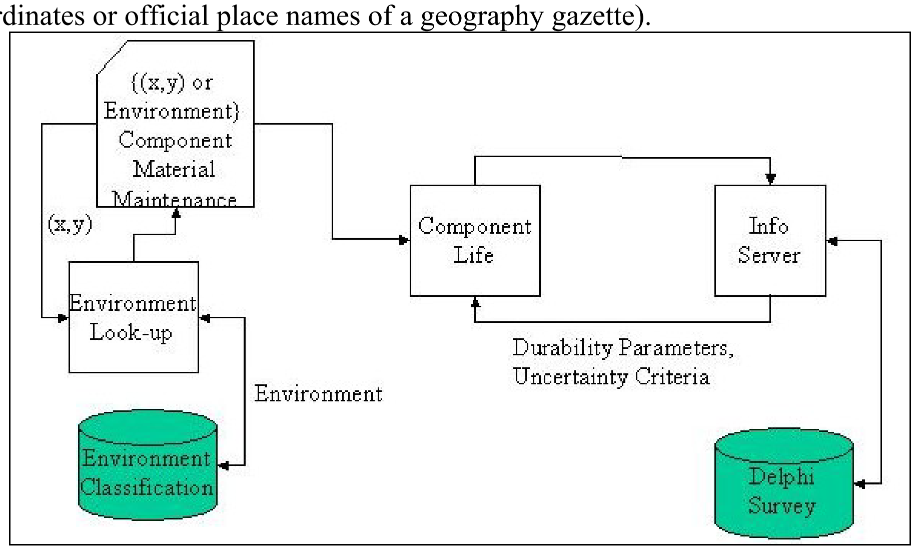

Figure 5. Using the results of a Delphi survey in a situated CBR context.

The CBR engine, which has access to the corrosion information server, is able to estimate the service life of a given building component in a particular use–case scenario, by combining the information from previous similar cases together with the information provided by the historical database, survey result and the holistic model (see Fig. 6).

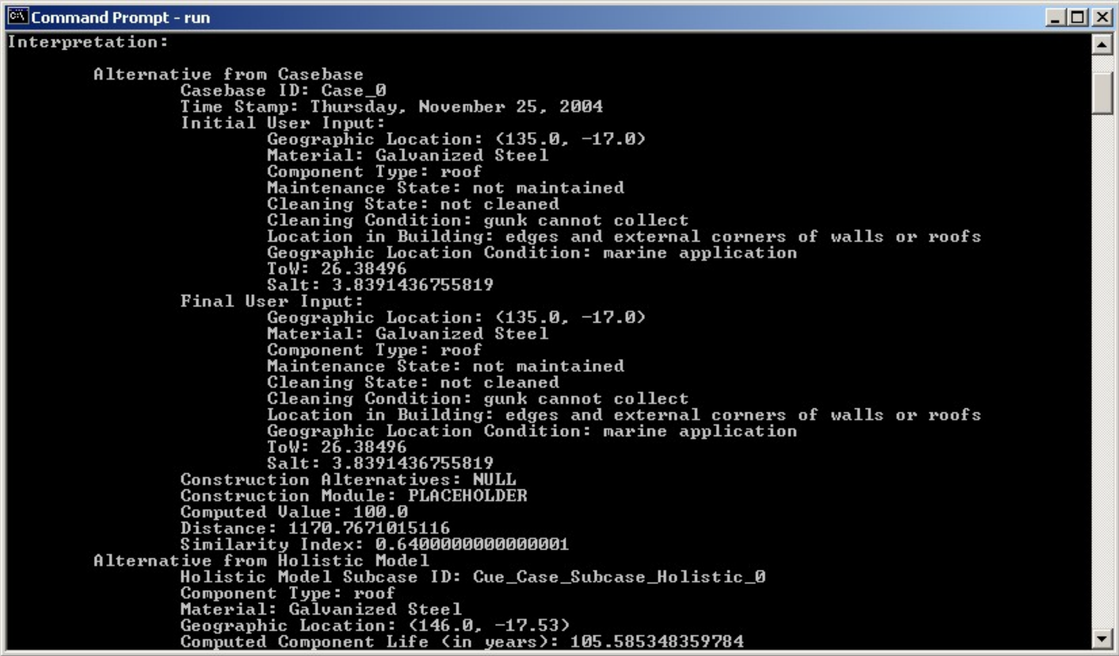

Figure 6. A screen showing output from the situated CBR engine in action.

8 SAMPLE APPLICATION WITH A GIS FRONT-END

A Cooperative Research Centre (CRC) for Construction Innovation [http://www.construction-innovation.info/] research program incoporates two case studies applying the concept discussed in this paper. The CRC for Construction Innovation is a national research, development and implementation centre focussed on the needs of the property, design, construction and facility management sectors.

The first test case involves implementing software that estimates the service life of roofs and gutters of school buildings in Queensland (see Fig. 7). The second test case is concerned with the prediction of the exposure environment of bridges in Queensland.

9 APPLICATION AND RELIABILITY OF MODEL

The integrated model will be applicable to the prediction of life of components of buildings throughout Australia while the general system is applicable for predicting component life wherever appropriate sources of data exist. The first application of the model is in the prediction of the life of gutters, as indicated above, however as this workis still in development it is not possible to define the accuracy of the model. However the accuracy of two components of the model – the Holistic Model of Corrosion and the Delphi Survey of Component life can be assessed. The predictions of the holistic model on mass loss of galvanised steel samples have been compared with data from over 40 field locations throughout Australia with the average error in prediction being 13% of the measured mass loss. Assessment of the reliability of the Delphi survey is more complex with a number of criteria being used including:

1. Internal consistency of the survey data.
2. Comparison against expected trends bases on material properties and environmental severity.
3. Comparison against other databases of component life.

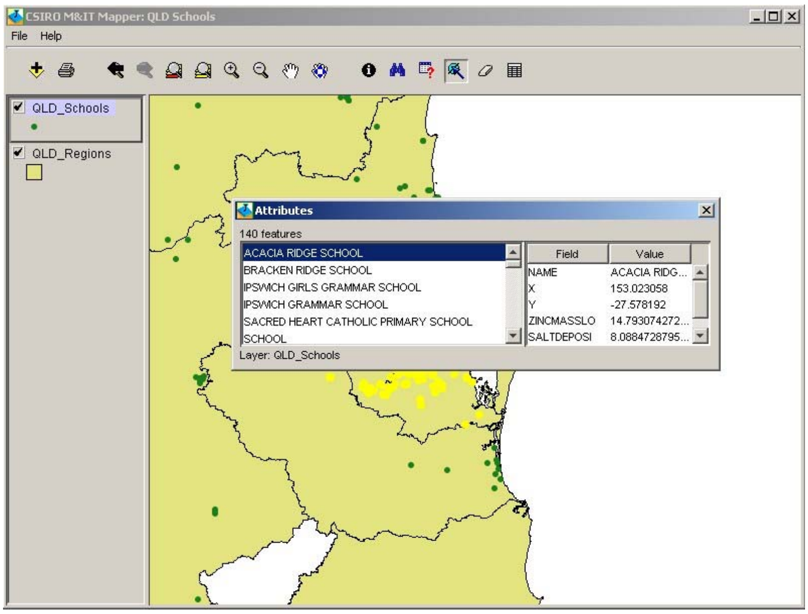

Figure 7. Early prototype software for estimating service life of roofs and gutters of schools in Queensland.

For many classes of components, data is not available in order to apply criterion 3. One class of components where data is available is roof sheeting with such a comparison being presented in Table 1. The table indicates that the predictions of the Delphi survey are close to those of other data sources.

Data	Environment	Mode (Years)	Mean (Years)	SD (Years)
Survey	Marine	10-20	12	6
Exp	Marine	5-10	14	12
Holistic	Marine	5-10	9	5
Maintenance	Marine		16	
Survey	Benign	30-50	35	13
Exp	Benign	>50	>50	
Holistic	Benign	>50	>50	
Maintenance	Benign		41	4

Table 1. Comparison of database and survey predictions for Service Life of roof sheeting. Survey refers to the Delphi survey, Exp to data from field exposures, Holistic to the holistic model, and Maintenance to maintenance data on roof replacement obtained from the Queensland Department of Public Housing

10 CONCLUDING REMARKS

This paper begins by outlining the difficulty of defining the reference service life for all the distinguishable components in a building. It argues that different types of information (e.g. historical data, process modelling and expert opinion) may play a role in developing these extensive databases. To integrate this information in a rigorous and efficient manner, a situated case-based reasoning engine has been developed. The situated CBR engine selects the appropriate information from diverse sources, including solutions in previous cases.

11 REFERENCES

TT4-155, Defining reference service life: an open innovation approach, by I. Cole *et al.*

Chesbrough, H. 2004, *Open Innovation: The New Imperative for Creating and Profiting From Technology*, Harvard Business School Press, ISBN 1-57851-837-7.

Cole, I., Chan, W.Y., Trinidad, G.S. & Paterson D.A., in press, 'Holistic model for atmospheric corrosion: Part 4—Geographic information for predicting airborne salinity', *Corrosion Engineering, Science & Technology*.

Cole, I.S., Muster, T., Paterson, D., Furman, S., Trinidad, G.S. & Wright, N. 2004, 'A holistic model of atmospheric corrosion: extending a microclimate model into a true corrosion model', presented to Eurocorr 2004: Long Term Prediction and Modelling of Corrosion, Nice, France, 12–16 September 2004.

International Standard Organization 2000, *Buildings and Constructed Assets—Service Life Planning—Part 1: General Principles*, ISO 15686–1:2000, ISO, Geneva.

Liew, P.–S. & Maher, M.L. 2004, 'Situated case-based reasoning as a constructive memory model for design reasoning', *Proc. CAADRIA 2004: 9th Conf. of the Association for Computer-Aided Architectural Design Research in Asia, Korea, 28–30 April 2004*, pp. 199–208.

Maher, M.L. 1998, 'CBR as a framework for design', presented to AAAI Workshop on Case-Based Reasoning Integrations, Madison, Wisconsin, USA.

Maher, M.L., Balachandran, B. & Zhang, D.M. 1995, *Case-Based Reasoning in Design*, Lawrence Erlbaum Associates, Inc., Mahwah, New Jersey, USA.

Sterling, L. & Shapiro, E. 1994, *The Art of Prolog, 2nd Edition*, MIT Press, ISBN 0-262-19338-9.

Trinidad, G.S. & Cole, I.S. 2002, 'Corrosion mapping and modelling', *Proc. 9th Int. Conf. on the Durability of Building Materials & Components, Brisbane, Australia, 17–20 March 2002*, vol. 3, paper 181, CSIRO Publishing, Melbourne, Australia.

Widom, J. 1996, 'Integrating heterogeneous databases: lazy or eager?', *ACM Computing Surveys*, **28A**(4), http://www-db.stanford.edu/~widom/integrating.html.

Calibrating and Benchmarking a Probabilistic Bottom-up Model for Predicting Failures in Cast Iron Water Pipes

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ABSTRACT

Physical probabilistic bottom up models have been developed for Cast Iron pipe failures to support improved pipeline asset management for water utilities. These models can be used to forecast future performance of critical assets that do not exhibit significant historical failure data. The model applied is a limit-state representation of the pipe failure process and this limit-state representation is used within a Monte-Carlo simulation framework. The limitation of this approach is that the model requires a range of input variables, such as material strength, defect growth rate, pipe wall thickness and internal pressure. To account for uncertainty and variation in these parameters, some of these are represented by stochastic variables.

In this paper, corrosion rates are paper described by a stochastic Weibull distributed variable. Unfortunately, due to prohibitive costs of extensive condition assessment for CI pipes, only limited experimental data is available to quantify stochastic corrosion rate variables. However, an alternative approach is to calibrate the physical probabilistic model against observed failure rate data in non-critical CI reticulation mains; data that is more commonly available. This paper describes such a calibration method, using a Least Squares approach.

Some of the results are reassuring, with a close match between experimental and fitted corrosion rate results. However, there is also a concern over the shape of the failure rate prediction curve. This leads on to future work, such as adding stochasticity in other variables and incoporating more realistic repair and renewal strategies. In conclusion, the exercise was useful in increasing the understanding of how physical probabilistic models can be used, and where future work should be focussed.

KEYWORDS

Cast Iron Pipelines, Pipeline Reliability, Monte-Carlo Simulation, Limit-State Modelling, Asset Management

1 INTRODUCTION

In Australian water utilities, CI pipes account for the majority of buried assets. Smaller diameters (between 40 mm and 300 mm) are used for reticulation and are classified as 'reactive'. These pipes are often left to operate until they fail, since the economic consequences incurred failure are relatively low. Since there are usually large volumes of historical failure data for these assets, the future failure rates in these mains are often forecast using statistical-based modelling [Jarrett *et al.* 2001]. In contrast, larger diameter trunk mains (between 300 and 600 mm) are usually classified as 'proactive' since relatively severe economic/social/environmental consequences are incurred upon failure and therefore proactive measures are often used. For instance, consider the failure of a large diameter supply main under a road in the Central Business District. Although they are comparable in age to reactive assets (up to 130 years old), proactive mains do not have significant failure histories. Consequently, water utilities are beginning to use non-destructive testing to quantify the condition of these assets. Examples are the use of electromagnetic tools to measure remaining un-corroded wall thickness, or the measurement of soil environment properties to identify areas of high corrosivity within a pipe network. In the absence of historical failure data, physical probabilistic failure models must be adopted to forecast future condition and failure probabilities of critical mains. Recent research in Australia has combined results from condition assessment, soil environment mapping and physical probabilistic modelling to provide asset management strategies for critical mains [Davis *et al.* 2004]. These models must be calibrated if they are to be used for asset management purposes.

2 PHYSICAL PROBABILISTIC FAILURE MODEL

Although external surface corrosion can occur in aggressive soil environments, failure of CI pipes in the field also depends on in-service loading conditions. Following Olliff and Rolfe [2001] the failure criterion for a buried CI pipe under combined pressure and external loads is given by

$$\frac{p}{p_c} \geq 1 - \left(\frac{W}{W_c}\right)^2 \tag{1}$$

where p and W are the applied internal pressure (in MPa) and external load (in kN/m) respectively. p_c is the critical internal pressure required for failure in the absence of an external load and W_c is the critical external load required for failure with no internal pressure. p_c (in MPa) and W_c (in kN/m) can be written in terms of the nominal tensile strength of the pipe wall σ_f [Olliff and Rolfe, 2001]:

$$p_c = \frac{2\sigma_f b_0}{D} \quad , \qquad W_c = \frac{1048\sigma_f b_0^{\,2}}{D} \tag{2}$$

The applied external load W in eq. (3) is comprised of separate components from soil dead loads and surface loads [Oliff and Rolfe, 2001] b_0 is the original pipe wall thickness (in metres) and D is the pipe mean diameter (in metres). As corrosion proceeds, the resistance of a pipe to service loads is reduced. σ_f can be related to the extent of corrosion damage in the pipe wall using fracture mechanics theory or loss of section analysis. As reported by Atkinson *et al.* [2002], the loss of section analysis (assuming that the corrosion simply reduces the pipe wall thickness) provides a greater correlation with experimental data. Based on experimental data from Atkinson *et al.* [2002], the dependency of nominal tensile strength (σ_f) on maximum corrosion rate $\dot{\delta}$ (in metres per year) in CI pipes can be written as

$$\sigma_f = \sigma_0 - 120\left(\frac{\dot{\delta}}{b_0}\right) \tag{3}$$

σ_0 is the inherent tensile strength of the pipe material (in MPa) in the absence of any corrosion damage, b_0 is the original pipe wall thickness (in metres) and t is the pipe age (in years). Whilst the simple model above can be used to predict failure times, it should be restricted to laboratory tests under precisely controlled corrosion rates and loading conditions. In practice however, buried CI pipes will be subjected to a range of operating conditions and corrosive environments. This uncertainty must be incorporated into the failure model. To develop a *physical probabilistic* failure model, the maximum corrosion rate $\dot{\delta}$ can no longer be treated as a single-valued quantity, but must be accommodated as a random, stochastic variable which exhibits a mean value and a variance. To quantify uncertainty in CI corrosion, the Weibull probability distribution function has previously proved useful [Atkinson *et al.* 2002]. A Weibull variable is one which has a 'survivor function' $S(\dot{\delta})$ written as

$$S(\dot{\delta}) = \exp\left\{-\left(\dot{\delta}/\alpha\right)^{\eta}\right\} \tag{4}$$

$S(\dot{\delta})$ is the probability that the maximum corrosion rate $\dot{\delta}$ is greater than or equal to a particular value, and can be obtained empirically from recorded data [Crowder *et al.* 1991]. α is the scale parameter of the Weibull distribution and η is the shape parameter. A way to estimate failure probability is via Monte Carlo simulation in conjunction with the physical failure model for CI pipes described above. Pipe lifetimes are sampled by repeatedly generating random values of corrosion rate and using these in the equations above to determine the time to failure for a set of pipe segments. For a simulated hypothetical pipeline comprised of these segments, a number of failures are recorded over time, which allows the lifetime distribution to be estimated. The steps required for Monte Carlo simulation of the physical failure model are outlined below:

1. Set up a population of short pipe segments with known diameter and original wall thickness
2. Randomly assign initial maximum corrosion rates in each pipe segment based on the Weibull probability distribution function.
3. Calculate/assign operating loads for each pipe segment;
4. Step through in a time marching loop and calculate reduced nominal tensile strength, reduced critical pressure, load bearing capacity; then check for failures and record failures.

A drawback of this approach is that the model requires a range of input variables: fracture toughness, corrosion rate, pipe wall thickness, internal pressure and external loads. These are represented by stochastic variables in the Monte-Carlo simulation, typically by the Normal or Weibull distribution. The model parameters are then the mean and standard deviation or scale and shape of these stochastic variables and the model has varying sensitivity to them. An important stochastic variable to consider for CI pipes is maximum corrosion rate and it is imperative to select the model parameters to accurately represent reality and ensure usefulness of the model. Unfortunately, due to the prohibitive costs of extensive condition assessment for CI pipes, only limited experimental data is available to quantify stochastic corrosion rate variables [Davis *et al.,* 2004].

3 CALIBRATING THE PHYSICAL PROBABILISTIC MODEL AGAINST ACTUAL FAILURE RATES

An alternative approach is to calibrate the physical probabilistic model against actual observed failures in *non-critical* CI reticulation mains, as such data is more commonly available. In this study, failure data for a set of small diameter reticulation mains has been used, which contains all pipes in the network as recorded in mid-2002. The failure recording period was from mid-1995 until mid-2002. The pipe database contains data for 81595 pipe assets, and 21001 pipe failures. The matching rate, which is the proportion of failures that can be linked with an asset, was initially approximately 75% in 1995 and increased to 95% in 2002 as failure recording procedures improved. The systematic variation (e.g. trend + oscillation) in number of failures per year must also be accounted for when calculating the observed failure rates from actual data. A 'failure year effect' is estimated by dividing the number of failures in a year with the average number of failures per year.

3.1. Estimating actual failure rates from data

When estimating the failure rates (failures per km per year), it is important to group the pipe assets into homogeneous pipe groups/cohorts. This ensures that those external factors that can influence pipe failure are isolated. Different pipe groups can be identified based on the year in which a pipe was installed and its surrounding soil environment. Fortunately, a large proportion of pipes in the network were installed in a similar soil environment, consisting of duplex, heavy-textured yellow clay, coarse dendritic drainage and undulating terrain [Davis *et al.* 2004]. It should also be noted that different installation practices were adopted throughout the history of these pipes. Figure 1 shows an event chart of CI pipe manufacturing methods, installation practices and concrete lining programs for CI pipes in Melbourne.

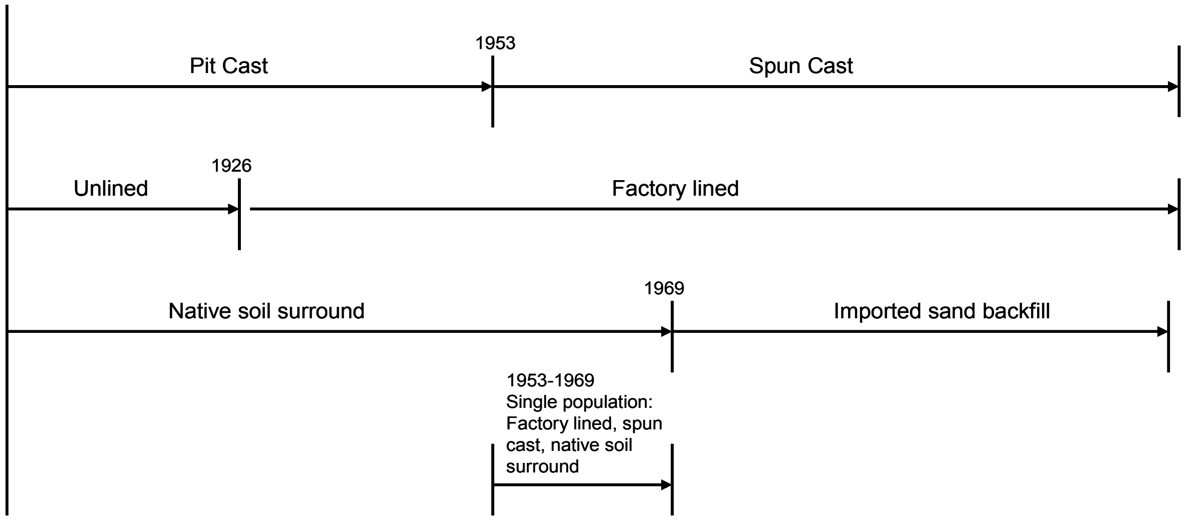

Figure 1. Event chart for manufacture, installation and cement lining of CI pipes in Melbourne.

A homogeneous group of pipes can be identified which were installed between 1953 and 1969. These pipes were spun cast only, with a factory-applied cement mortar lining, and installed in native soil backfill [Davis *et al.*, 2003]. Choosing installation dates either side of this interval would result in a mixed population of pipes that were manufactured using different methods and installed in both native soil and imported sand backfill. Therefore, this study focussed on the single population of pipes installed between 1953 and 1969. The total length of pipes in this cohort was 1709km which accounts for 40% of the network's length of Cast Iron pipes and 23% of the total network length. These pipes and their failures were extracted from a Microsoft Access database, under the conditions; pipes with diameters smaller than 40 mm were not included as there was uncertainty regarding the validity of the physical probabilistic model in this diameter region; only pipes installed between 1953 and 1969 were included; only Cast Iron pipes were included.

The following entries from the failure table were used in the analysis of this dataset: failure id, failure types according to recommendations to the Water Services Association of Australia, date of failure, as well as the pipe asset in which the failure occurred. Also, only those failure modes that occurred by corrosion and combined corrosion/fracture were included. These are 'Blown sections' (corresponding to removal of a corroded section of the pipe wall under applied stress), 'Longitudinal split' (fracture along the pipe axis) and 'Perforations' (pitting corrosion). Other failure modes, such as 'joint leak' and 'fitting failure' are beyond the scope of the physical probabilistic failure model and were omitted. In the queried data set, 3723 failures out of 8292 failures (45%), were categorised into one of the included failure modes. Hence, in this case, it is expected that the physical probabilistic model should predict only a little less than half of all failures. The average failure rate for a particular pipe age is defined as:

$$\text{Average failure rate at age A} = \frac{\text{Number of observed failures at age A}}{\text{Failure Exposure [km} \cdot \text{Years]}} \quad (5)$$

To calculate the failure rates (failures per km per year), the failure exposure (km-years) must first be calculated. The failure exposure is defined as a length of pipes of a specific type going through a year in which failures have been recorded. For this group of pipe assets, the failure exposure as a function of age is shown in Fig. 2. The failure exposure is calculated by using the following routine:

1. For the failure years, 1995→2002: Step through the pipe data set, pipe by pipe;
2. Calculate the pipe age at time of failure: 'Age = Failure year – Construction year';
3. Calculate the incremental exposure: E[Age] += F[fYear]*M[fYear]*R[fYear]*L.

L is the length of a pipe. R is a table containing the recording rates for different failure years. M is a table containing the matching rates for different failure years. F is a table containing the Failure year effects for different years. E is a table containing the failure exposure at different pipe ages. Figure 2 shows the calculated exposure at different pipe ages for the data set used.

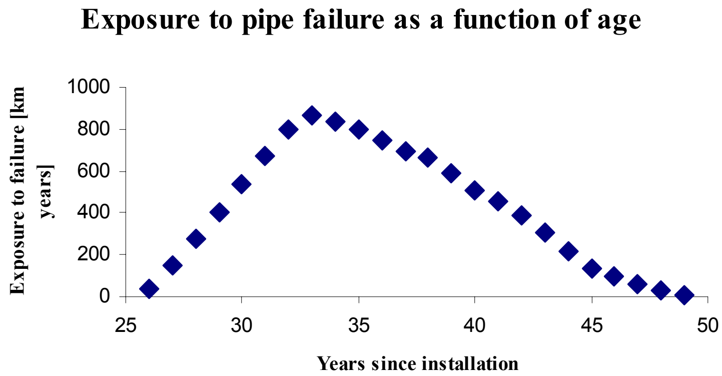

Figure 2. Failure exposure for a range of pipe ages. This exposure relates to recorded failures between 1995 and 2002 in Cast Iron Pipes installed between 1953 and 1969.

Failure rate and failure exposure calculations are based on the following Poisson model [Gut, 1995], which assumes that the frequency of failures is proportional to the exposure, and that the failure frequency varies with time, and that the variance has the same value as the expectation value:

$$E[Y_t] = \lambda(t) \cdot Exposure\ (t) \quad (6)$$

$$Var[Y_t] = \lambda(t) \cdot Exposure\ (t) \quad (7)$$

The estimator of the failure rate and its variance is given by:

$$\hat{\lambda}(t) = \frac{Y_t}{Exposure\ (t)} \quad (8)$$

$$Var[\hat{\lambda}(t)] = \frac{\lambda(t) \cdot Exposure\ (t)}{Exposure\ (t)^2} = \frac{\lambda(t)}{Exposure\ (t)} \quad (9)$$

$$\hat{\sigma}^2_{\hat{\lambda}(t)} = \frac{\hat{\lambda}(t)}{Exposure\ (t)} = \frac{y_t}{Exposure\ (t)^2} \quad (10)$$

Y_t is a stochastic variable for the number of failures at age t; and y_t is the actual number of failures at age t. $E[Y_t]$ is the expectation value of Y_t. Exposure(t) is the exposure to failure at age t as above.

$\lambda(t)$ is the failure rate at age t. Estimators are notated as exemplified by $\hat{\lambda}(t)$, which is the estimator of $\lambda(t)$. Please note that the estimator is in itself a stochastic variable. Using the Normal approximation, confidence limits are given by $1.96 \cdot \hat{\sigma}_{\hat{\lambda}(t)}$. Failure rates at different pipe ages as well as the 95% confidence limits are displayed in Fig. 3. The certainty in the estimates increases with the square root of the number of failures at a pipe age and decreases inversely proportionally to the exposure.

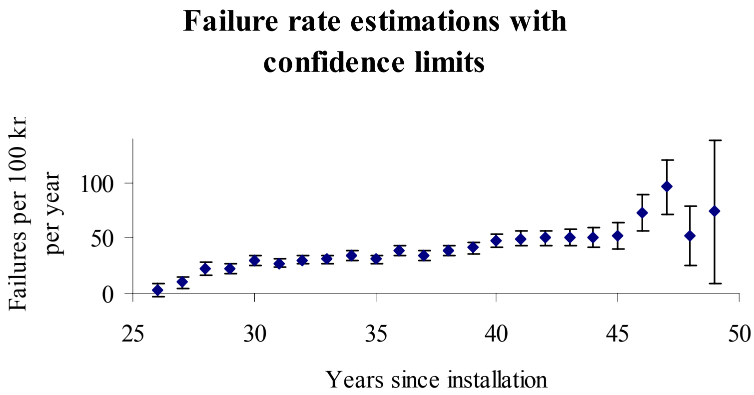

Figure 3. Estimated failure rates (per 100 km per year) for different pipe ages with related confidence limits on each failure rate estimate.

3.2 Fitting corrosion rate variables in physical probabilistic model by Least Squares estimation

In Eq. 4, maximum corrosion rate is a stochastic variable that can be reasonably well represented using a 2-parameter Weibull distribution, with scale and shape parameters α and η respectively. However, the cost of exposing buried CI pipes and measuring corrosion rate can be prohibitive, and relatively few previous studies have determine α and η from experimental data [Davis *et al.* 2004]. Having estimated actual failure rates from data, output from the physical probabilistic failure model can be matched against this data.

Since the observed failures occurred in a range of pipe sizes operating at different pressures, the Monte Carlo simulation is modified to generate an equivalent hypothetical data set for comparison. For all pipes in the actual data set the steps involved are:

1. Retrieve pipe size and operating pressure from the actual data set and create an equivalent pipe in the hypothetical data set.
2. Randomly assign a maximum corrosion rate to each pipe in the hypothetical data set.
3. For each year that failures where recorded in the actual dataset, evaluate whether each pipe in the hypothetical data set has failed according to the physical failure model above.
4. Record failures in a given year.

This generated hypothetical failure history can then be used to estimate failure rates just as failure rates can be calculated from the actual observed failure data. The Weibull scale and shape parameters for maximum corrosion rate (α and η) can be adjusted so that the predicted failure rates are as close as possible to the actual observations. Whilst Maximum Likelihood estimates of these parameters would perhaps be the most rigorous method of parameter estimation, a useful first approach is to minimise the least-squares deviations between observed and predicted failure rates. The task is to minimize a goal function $L(\alpha, \eta)$ by varying the Weibull shape and scale parameters for maximum corrosion rate. $L(\alpha, \eta)$ is defined as the sum of the squares of the deviations between actual observed and simulated failure rates. α and η are the two Weibull parameters for maximum corrosion rate (Eq. 4). In order to calculate the goal function $L(\alpha, \eta)$, the predicted failure rates must first be estimated through Monte-

Carlo simulation. By varying α and η in the failure rate predictions, graphs describing the variation in the goal function $L(\alpha,\eta)$ was produced (Fig. 4).

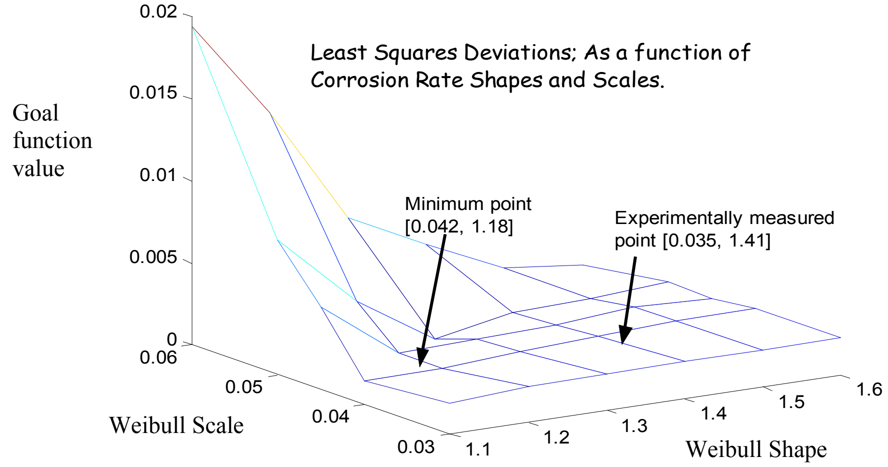

Figure 4. Variation in goal function L(α,η) with Weibull scale and shape parameters for maximum corrosion rate, α and η

Figure 4 plots the Weibull parameters for maximum corrosion rate (α and η) on the x and y axes and the goal function $L(\alpha,\eta)$ in the z-axis. The minimum point was identified by running a large number of simulations and identifying those values of α and η that give the lowest value of L(α,η). Despite some variation due the stochastic nature of the simulations and the plateau in the goal function, the minimum point was consistently identified as α = 0.042 and η = 1.18 (corrosion rate expressed in mm/year). This indicates that using these values in the physical probabilistic failure model gives the best agreement between predicted and observed failure rates. For comparison, experimentally determined values of α = 0.035 and η 1.41 are also included in Fig. 4. These values were obtained using raw data from a condition assessment program that was previously conducted on critical CI mains [Davis *et al.,* 2004]. It is encouraging to note that whilst the experimental values of α and η do not minimise the goal function, they are located on a relatively flat area of the $L(\alpha,\eta)$ surface with no significant difference in terms of least squares deviations. Using least squares estimation means that it is likely that the fit is slightly skewed. Figure 5 shows that the resulting Weibull probability density functions for maximum corrosion rate are very similar. The difference is that, compared to the fitted values, the experimentally determined values of α and η result in slightly less variation and lower corrosion rates. However, it should be noted that the experimentally determined values were obtained from corrosion rate measurements in a single soil environment [Davis *et al.* 2004]. Although this soil environment covers a large region of the network area, it is likely that actual failures in the data set occurred in more than one single soil environment.

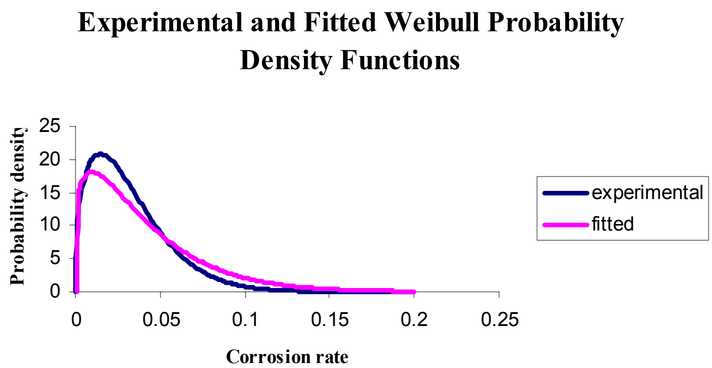

Figure 5. Probability density functions for maximum corrosion rate in the physical probabilistic model: Experimentally determined vs. numerically fitted

Figure 6 shows a comparison between observed failure rates from the data set and predicted failure rates form the physical probabilistic model.

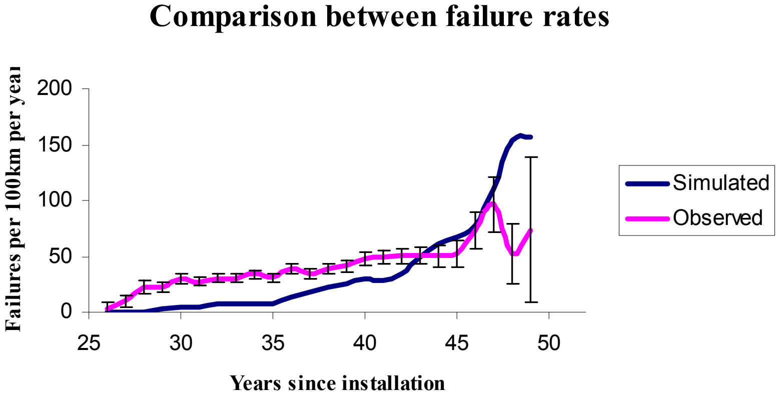

Figure 6. Predicted and observed failure rates observed in the time period

It can be noted in Figure 6 that whilst the observed failure rates follow a logarithmic type growth with age, the physical probabilistic model predicts an exponential growth, with particular deficiencies in the tail ends of the curve (young and old pipes). It can appear that because of this non-linear characteristic of the failure rate curves, it is questionable whether the model in its current simplistic form could accurately predict the number of failures outside the given age period. However the purpose of this simplified model was to estimate and investigate the impact of stochastic corrosion rate rather than estimating failure rates. Future work will aim at establishing what the shape really as, as well as investigating how the model could be extended in order to better match observed data. There are indications in other data sets that there is more of an exponential shape in the observed failure rate curves than was seen in this data set for pipes up to older ranges. Additionally, there is likely to be a an effect from the lack of stochasticity in internal and external loads, underestimating pressures, as well as the lack of consideration to the impact of repairs and renewals. The probability distribution tails of the pressures will push failures upwards for newer pipes, allowing for a smoother rise in failure rates to be fitted to observed failure rates. Improvements to be incorporated into the model relate to implementing:

- Stochastic pressure, accounting for poor or inadequate pressure data
- Stochastic external loads
- Renewal and repair strategies into simulations
- Logarithmic or exponential corrosion pit growth
- Stochastic pipe wall thickness

For instance, with stochasticity on loads, the probability of extreme loads (such as what is caused by pressure surges) will bring the failure rate up for newer pipes. This effect in combination with implementing repair and renewal strategies into the prediction model that is likely to filter out poor condition pipes will bring the failure rates down for older pipes. There is also a limited effect that the accumulated repairs will bring down the exposure as time goes by. Accounting for all such effects, there is considerable potential for adjusting the shape of the predicted failure rate curves to better fit observed failure rates.

4 CONCLUDING REMARKS

This investigation initiated from the problem of predicting failures in populations of CI pipes with limited (or no) failure history. Whilst a physical probabilistic failure model has been developed, it is 'data hungry' and the relevant parameters it requires must be estimated. There is also a need to justify and benchmark the model against existing failure rates. Therefore, this paper has focussed on using existing failure data in non-critical CI pipes for two purposes.

The first purpose was to gain better estimates of corrosion rate parameters and evaluate the extent by which experimental corrosion rate data can be used. For this purpose, the corrosion rate parameters were chosen so that the least-squares deviations between predicted and observed failure rates were minimised. Whilst this is less rigorous methodology to the Maximum Likelihood (M-L) method (which should be used when the Normal distribution assumption does not hold), it is well-suited to computer simulation models rather than closed form models. However, further work will aim at improving the estimation methodology. The estimated corrosion rate parameters were relatively close to those that were experimentally determined in a previous study [Davis *et al.* 2004]. This indicates that experimental corrosion rate estimations for condition assessment of critical CI pipes are valuable.

The second purpose was to benchmark the physical probabilistic model against existing failure data to see how closely the model could match actual failure rates, and whether it could be extrapolated to other ages and diameters. It was found that the model could predict failures that were in the "right ball-park" but the predictions were also flawed in the sense that it appeared as if the shape of the failure rate curve could not be replicated. However, it was noted that currently, only variation in maximum corrosion rate is accounted for and with further work, the fitting exercise should be extended to stochastic parameters for material properties and loading conditions. This may adjust the shape of the predicted failure rate curves to better suit observed failure rates.

In conclusion, the exercise was useful for increasing the understanding of how physical probabilistic models can be used to predict failures in Cast Iron pipelines. However, further work is required before the physical probabilistic model be applied with confidence for pipes with no failure data.

5 REFERENCES

Atkinson, K., Whiter, J.T., Smith, P.A., and Mulheron, M. (2002). Failure of Small Diameter Cast Iron Pipes, Urban Water, **4**, pp. 263-271

Crowder, M.J., Kimber, A.C., Smith, R.L., and Sweeting, T.J. (1991) Statistical Analysis of Reliability Data, Chapman and Hall, London.

Davis, P., Allan, I., Burn, S., van de Graaff, R. (2003). Identifying trends in Cast Iron pipe failures with GIS maps of soil environments. Proc. Pipes 2003-Back to Basics, Design and Innovation, , Wagga Wagga, Australia, 21-23 October, 2003.

Davis, P., M. Moglia, et al. (2004). Estimating failure probability from condition assessment of critical cast iron water mains. Proc. 6th National Australasian Society for Trenchless Technology Trenchless Conference, Melbourne Exhibition & Convention Centre, Melbourne, Australia, 27-29 September, 2004.

Gut, A. (1995). An Intermediate Course in Probability, Springer-Verlag.

Jarrett, R.. Hussain, O., Veevers, A. and Van der Touw, J., (2001). A Review of Asset Management Models Available for Water Pipeline Networks, Proc. Int. Conf. of Maintenance Societies, Melbourne, Australia

Olliff, J. and Rolfe, S. (2002). Condition Assessment: The Essential Basis for Best Rehabilitation Practice. Proc. 20th International NO-DIG conference, Copenhagen, Denmark, May 28-31 2002

Enhancing the Factor Method – Suggestions to Avoid Subjectivity

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ABSTRACT

ISO 15686 standard can be used as a guide for service life planning and service life prediction. The standard can guide user to find out the Reference Service Life (RSL) of components and materials from both laboratory aging tests and field exposures and provides some directions on methods for estimating the Estimated Service Life (ESL) of components or materials in real projects. The transition from experimental data to ESL can be done, according to ISO 15686 adopting the Factor Method.
In particular, Factor Method computes ESL of building components by multiplying their RSL by some factors. There are several examples in literature of service life estimation made using the Factor Method. Even if these examples are quite different from each other, all of them highlight advantages of the method and a few disadvantages. The worst one is the high degree of subjectivity of the method: different results can be obtained by several people estimating the ESL of a building component (material) with the same input data.
This paper presents results from a research developed at the BEST– Polytechnic of Milan aimed to give a rigorous technical approach to the development of the Factor Method in order to avoid the previous issue, that is to say the subjectivity in assessing the right value for each factor.
In the paper, after a brief description of the Factor Method, a new approach to assessing the value of the A-factor (concerning the quality of the component) is suggested.
The idea is to divide each factor in sub-factors and to create evaluation grids helpful for the users in assessing the value of each sub-factor. In such a way the assessment of a value involves in a process divided into two different steps: building and using the grids. The first step, characterized by high subjectivity, can be made once for all for different classes of components (materials) by experts or even by manufacturers. The second step can be made by the user himself, even if he is not very experienced in the service life prediction. Of course, grids have to be based on standards or on widely used designing procedures.
A case study shows the application of the suggested approach to the service life prediction of ceramic tiles focusing, mainly, on the A-factor. Starting from a failure mode analysis of the ceramic tile, grids based on (Italian) standards for the evaluation of the characteristics of building materials are built.
A sensitivity analysis on the grids is performed to highlight which of the sub-factor is the most influencing the final result and a risk analysis (using the Monte Carlo Tecnique) is carried out to assess the risk related to errors in building the grids.

KEYWORDS

Service life prediction, performance limits, degradation, engineering design methods.

1 INTRODUCTION

Goal of the paper is to show the outcomes of a research carried out by the authors within the BEST Department – Polytechnic of Milan, aiming at providing a rigorous technical approach for the development of the Factor Method, whose main disavantage can be the excessive subjectivity during the phase of attribution of values to the factors leading from RSL (Reference Service Life) to ESL (Estimated Service Life).
A case study is provided in the paper to better explain the different steps constituting the developed procedure.

2 FACTOR METHOD WITHIN THE ISO 15686

At date, the only standardized method to obtain ESL from experimental data is the Factor Method.
The Factor Method, defined within the ISO 15686-1, computes ESL of building components by multiplying their RSL by some factors, whose number and combination depends on the type of building component (material) under investigation. There are several examples in literature of service life estimation made using the Factor Method; each one of these examples develops the Factor Method with specific limits due to building component (material) under analysis. Even if these examples are quite different from each other, all of them highlight advantages of the method and a few disadvantages. The worst one is the high degree of subjectivity of the method: different results can be obtained by several people estimating the ESL of a building component (material) with the same input data.

Stated that the formula defining the Factor Merhod is the following [ISO 2000]

ESLC = RSLC * factor A * factor B * factor C * factor D * factor E * factor F * factor G

one can notice how the ESLC value is strictly influenced by the estimator judgement.
For example, some simple considerations of Cusmano *et al.* [2003] show how even when the differences between the factor values attributed by different estimators are numerically small, the difference resulting from the ESLC figures is much more significant.
In the quoted sample, differences lower than 10% in the attribution of values to the factors lead to a difference in the ESLC value even over 80%.
From here, therefore, is clear the need to define guidelines and tools allowing the user of the Factor Method to be accompanied in the least subjective way towards the attribution of the several values.

3 EVALUATION OF THE A – FACTOR

The first phase of the research has been finalized at defining tools for the evaluation of the A-factor ("quality of components"), result of detailed analysis of all the parts constituting the building component.
The developed procedure get through the functional analysis of technical elements and components of the work, the individuation of factors capable of influencing their service life and the comparison with similar components, whose performance are known, with the aim to provide a more objective evaluation.
The basic idea of the research is to divide each factor in sub-factors and to create evaluation grids helpful for the users in assessing the value of each sub-factor. In such a way the assessment of a value involves in a process divided into two different steps: building and using the grids. The first step, characterized by high subjectivity, can be made once for all for different classes of components (materials) by experts or even by manufacturers.
The second step can be made by the user himself, even if he is not very experienced in the service life prediction. Of course, grids have to be based on standards or on widely used designing procedures.
Below are briefly exposed the steps constituting the procedure to estimate the service life of building components, using the proposal of the evaluation's method for the A-factor.

3.1 Rational organization of preliminary information

First, the building work has to be disassambled with a tree classification system, in systems and sub-systems, to individuate any time (and in univocal and rational way) all the elements constituting the building itself. To reach this, in the case study has been chosen the Italian standard UNI 8290, which divides the work into more levels, for homogeneous elements. Whitin the standard, there are three levels: classes of technological units, technological units and classes of technical elements. The need to work with building components made us to add two further levels: technical elements and building components.
For each component, possile and plausible failures have been individuated through the agents-actions-effects analysis.

3.2 Agents – actions – effects analysis

The analysis consist in individuating all the possible degradation's agents, which the component will probably be subjected to during its service life, then in identifying the related actions htat those agents will develope, and finally in defining all the possible and plausible failures. In the case study the standard ISO 6241:1984 led us to the individuation of the agents.
In this phase, attention should be paid to identify all the agents affecting the most the service life of the building component, not neglecting the agents characterized by low intensity, but with a cyclicity and/or duration that could make critical their effects over time.

3.3 Individuation of the sub-factors

One or more characteristics (which the most influence the service life of the component in reference conditions) are related to each effect. These characteristics, if measurable in the design stage with standardized methods, lead to the definition of a sub-factor, as shown in the following **figure 1**.

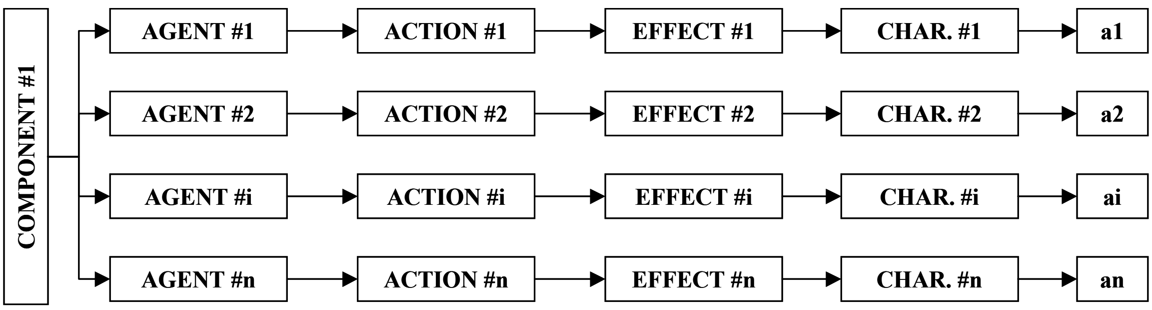

Fig. 1. The way leading from agents to sub-factors.

3.4 Looking for standards with methods to measure characteristics

At this point, for each sub-factor an evaluation system is needed.
Such a method should be always valid for components of the same kind and should, possibly, be based on performance.
The best results are reached if the method provides an evaluation scale (through "performance classes"), and not just minimum performance specifications.
It is opportune, for better reliability of evaluations, searching for standards or others suitable sources (cmp. fig. 4 for some examples of adopted standards).
It is to be noted that the necessary condition that all the sources have to satisfy, to be useful within the Factor Method, is the presence of evaluation's scales able to cover the whole existence field of the performance's component.

3.5 Creating evaluation grids for the reference component

This step is the most complicated one of the proposed procedure and because of this should be done by experts; desirably, manufacturers themselves should provide such evaluation grids within the product's technical documentation.
Evaluation grids are just functions, receiving as input data the performance classes found in the standards and returning particular values.
As the Factor Method is based on the comparison between reference and in-use conditions, and as the evaluation grids are built starting from the reference conditions of the building component, user has just to put the performance specification inside the grid: in such a way, the comparison between the two components will automatically happen.

All the evaluation grids are characterized by the fact that:

- The zero-value is associated to performance class in reference condition;
- All of them contain monotonous (increasing or decreasing) functions, with degree equal or higher than one;
- The value's set goes (conventionally) from a minimum of "–5" to a maximum of "+5".

Fig. 2 shows an example of evaluation grid; the reference condition of the building component coincides with the highlighted performance class. Three fixed points ("–5" e "+5" for the extreme performance classes and "0" for the reference class) and the lack of linearity of the function can be noted.

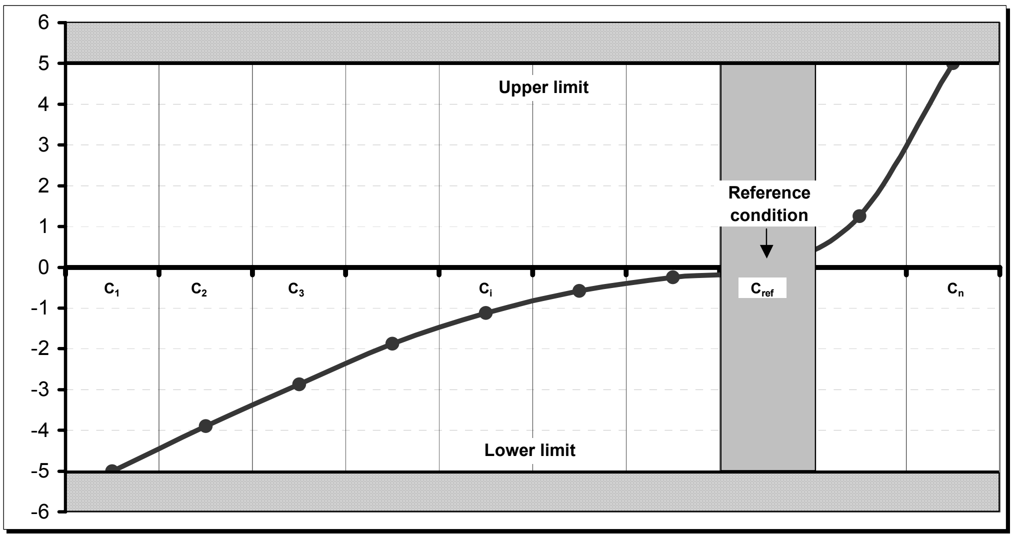

Fig. 2. Example of evaluation grid: performance classes and related values.

3.6 Attribution of weights to the sub-factors

Once defined the "n" sub-factors on the basis of the considarations above, it is possible to evaluate the relative importance of each characteristic in respect of the others.
Attention should be paid to the fact that comparisons among performance, in this step, represent precise design choices; the priority of a sub-factor on another one is determined every single time by the designer, after considerations about:

- The final destination of the environment in which the building component will work;
- Typology of the end user;
- Environmental load and in-use conditions;
- Etc…

As a standardized method in attributing values, the use of a value matrix able to compare all the sub-factors is proposed.

This method involves the complilation of a matrix, called "one-to-one comparison matrix" (see **fig. 3**), with the aim to compare the importance of each single sub-factor in respect of the whole number of sub-factors.

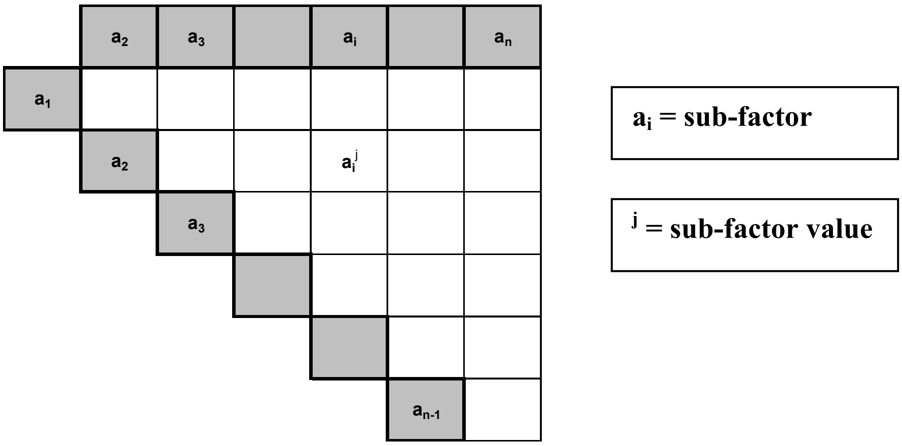

Fig. 3. One-to-one comparison matrix.

The designer might face with three different cases:

- **Equivalence between two sub-factors;**
- **Priority of a sub-factor over the other one;**
- **High priority of a sub-factor over the other one.**

Summarizing, comparisons between sub-factors a_n and a_m can involve into five possible outcomes:

- $\mathbf{a_n^3}$ High priority of a_n over a_m;
- $\mathbf{a_n^2}$ Priority of a_n over a_m;
- $\mathbf{a_n^1 a_m^1}$ Equivalence between a_n and a_m;
- $\mathbf{a_m^2}$ Priority of a_m over a_n;
- $\mathbf{a_m^3}$ High priority of a_m over a_n.

Adding the values obtained from each sub-factor, a temporary classification of importance can be defined. This one will be later normalized in 1 – 10 scale.

3.7 Attribution of values to the design component

At this point is needed to reconsider the same standards used to create the evaluation's scales, with the purpose of evaluating the design component.
To do this, every performance specification of the component is assigned to a class among the evaluation's scales created before, obtaining the value related to the specific sub-factor.
For each sub-factor 3 cases are possible:

- **V > 0:** The design component provides performance better than those ones of the reference component;
- **V = 0:** The design component provides performance similar to those ones of the reference component;
- **V < 0**: The design component provides performance worse than those ones of the reference component.

3.8 Calculation of the A – factor

Now it is possible getting the Ā-factor, through a pondered average of the several values Vi obtained from the performance evaluation (§ 3.7, and column 8, fig. 5) and using the weights Wi deduced before (§ 3.6, and column 9, fig. 5).

The simple formula is the following:

$$\bar{\textbf{A}} - \textbf{factor} = \frac{\sum_{i=1}^{n} V_i * W_i}{\sum_{i=1}^{n} W_i} \quad \text{where} \quad \begin{array}{|l} V = \text{value} \\ W = \text{weight} \\ n = \text{number of sub-factors} \end{array}$$

After the calculation of the **Ā – factor,** is needed a last conversion, shifting the output from 0 (current reference condition) to 1 (classic reference condition for the Factor Method).
The conversion method is based on the assumption (adopted in almost every example of Factor Method found in literatur and also in the example within the ISO 15686-1) that the attribution of values to the factors can be reasonably vary going from a minimum of 0.8 to a maximum of 1.2; such a range, compared to that one defined before ($-5 \le a_i \le +5$), allows to get finally the A-factor.

4 CASE STUDY

A case study is now exposed, where the procedure proposed in the previous pages has been appied to a building assigned to hotel. The procedure has no limitations as regards the final destination of the work; naturally, the necessary condition for its application is the availability of performance standards. The more detailed the information is, the more objective the outcomes of the method will be. Destructuration talked in the § 3.1 allowed us to identify the considered component through an univocal code. Results are related to ceramic tiles, placed in the kitchen of the restaurant.
Once identified the component, we have proceeded with the individuation of the mainly relevant degradation agents, the analysis of the actions and the evaluation of the effects on the tile. Each agent is related to an effect and a performance characteristic, which makes the component able to oppose itself to the degradation over time.
The following **figure 4** shows the way leading from the agents to the reference standards.

Evaluation grids for all the sub-factors, created starting from the reference standards are shown in **figure 5**. One can see how the central part is related to the reference component, whereas the last part is related to the design component. Bold lines refer to the reference classes (also characterized by the value “0”). In the eighth column, values equal to zero indicate preformance very similar between the two components. The last column has been filled after the compilation of the one-to-one matrix (§ 3.6) and, later, the normalization in 1 – 10 scale of the obtained values.

The last step (§ 3.8) allowed us to calculate the **Ā-factor** and, owing to this, the **A-factor**, to be used in the Factor Method.

$$\bar{\textbf{A}} - \textbf{factor} \cong 2{,}32 \xrightarrow{\text{conversion}} \boxed{\textbf{A} - \textbf{factor} \cong \textbf{1,1}}$$

Restaurant – Kitchen floor				Code	3.3.2.1.2
Glazed grès				Code	3.3.2.1.2.2
Classification (ISO 6241)		**Actions**	**Effects**	**Performance characteristics**	**Reference standards**
Nature	**Origin**				
Mechanical agents	Live loads	Scratches of the surface	Lack of homogeneity	Hardness of the surface	UNI EN 101
Mechanical	Live loads	Abrasion of the	Lack of	Resistance to	UNI EN ISO

agents		surface	phisical homogeneity	surface abrasion	10545-7
Chemical agents	Water	Waterfall	Slipperiness	Slip resistance	DIN 51130
Chemical agents	Vinegar, citric acid	Corrosion	Lack of phisical homogeneity	Resistance to chemical attacks	UNI EN ISO 10545-13
Biological agents	Food	Presence of stains	Lack of aesthetic homogeneity	Resistance to stains	UNI EN ISO 10545-14

Fig. 4. Identification of the reference standards to create evaluation grids.

Sub-factor	**Performance characteristic**	**Reference standards**	**Reference component**			**Design component**		
			Classes		**Values**	**Classes**	**Values**	**Weight**
a_1	Hardness of the surface	UNI EN 101	Talc	1	-5	7	-0,25	1
			Gypsum	2	-3,9			
			Calcite	3	-2,875			
			Fluorite	4	-1,875			
			Aptite	5	-1,125			
			Feldspar	6	-0,575			
			Quartz	7	-0,25			
			Topaz	**8**	**0**			
			Corundum	9	1,25			
			Diamond	10	5			
a_2	Resistance to surface abrasion	UNI EN ISO 10545-7	n = 100	C0	-5	C2	-1	1
			n = 150	C1	-2,5			
			n = 600	C2	-1			
			n = 750	**C3**	**0**			
			n = 2100	C4	1			
			n = 12000	C5	5			
a_3	Slip resistance	DIN 51130	$3° < a \leq 10°$	R9	-5	R11	1,25	10
			$10° < a \leq 19°$	**R10**	**0**			
			$19° < a \leq 27°$	R11	1,25			
			$27° < a \leq 35°$	R12	2,5			
			$a > 35°$	R13	5			
a_4	Resistance to chemical attacks	UNI EN ISO 10545-13	No effect	G (L, H) A	5	G (L,H) B	0	4,44
			Soft attack	**G (L, H) B**	**0**			
			Deep attack	G (L, H) C	-5			
a_5	Resistance to stains	UNI EN ISO 10545-14	A-Procedure	C5	5	C5	5	10
			B-Procedure	**C4**	**0**			
			C-Procedure	C3	-1,25			
			D-Procedure	C2	-2,5			
			No removal	C1	-5			

Fig. 5. Definition of evaluation grids and comparison between reference and design component.

5 SENSITIVE ANALYSIS AND RISK ANALYSIS

In order to verify the evaluation grids built for the case study, a sensitivity analysis has been carried out, highlighting that, for the service life of the considered component, the most influencing sub-factors are, in order of importance:

- Slip resistance (a_3) and resistance to stains (a_5);
- Resistance to chemical attacks (a_4).

A risk analysis (using the Monte Carlo method) has also shown the influence of errors which could occur in creating of the grids.
A study on four different kind of tiles has highlighted how this tool may be used to better understand which one, among several building component, is more subjected to mistakes in the phase of realization of the grids (see **fig. 6**).

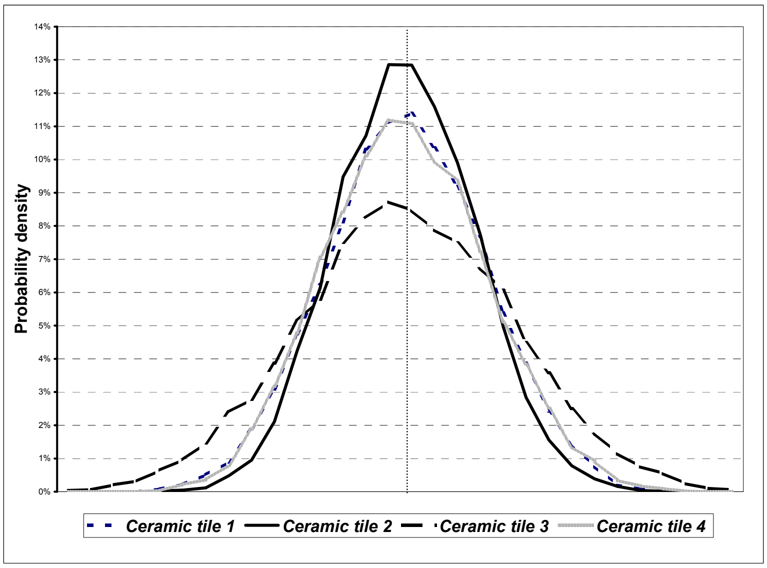

Fig. 6. Behaviour of ceramic tiles in consequence of mistakes in creating evaluation grids.

Different widths of the statistical distributions represent, in fact, the higher or lower reliability of the tiles to possible mistakes in the attribution of values related to performance classes.

6 CONCLUSIONS

The study (first) and the use (later) of the Factor Method have pointed out advantages (as the high degree of semplicity) and disadvantages (as the high degree of subjectivity). Because of this, the lack of guidelines involves the use of the Factor Method only by a small amount of experts.
The procedure described in the paper is a contribution to the spreading of the Factor Method, and has been developed to be a helpful tool for not experienced users in the field of the service life prediction of building components.

7 REFERENCES

Cusmano, G., Lucchini, A., 2003 "Implementing The Factor Method For Service Life Estimation Using Weighted – Factors", Proc. International Workshop on "Management of Durability in the Building Process", Milan, Italy, 25-26 June 2003

DIN 51130:1992 "Testing of floor coverings; determination of the anti-slip properties; workrooms and fields of activities with raised slip danger; walking method; ramp test"

DPR 554/99 "Regolamento di Attuazione della Legge Quadro in Materia di Lavori Pubblici"

ISO 6241:1984 "Performance standards in building. Principles for their preparation and factors to be considered"

ISO 15686-1:2000, "Building and constructed assets – Service life planning: General principles"

ISO 15686-2:2001, "Building and constructed assets – Service life planning: Service life prediction procedures"

L. 109/94 "Legge Quadro in Materia di Lavori Pubblici"

UNI 8290/1:1981 "Edilizia residenziale. Sistema tecnologico. Classificazione e terminologia"

UNI EN 101:1992 "Piastrelle di ceramica. Determinazione della durezza della superficie secondo la scala di Mohs (Ceramic tiles – Determination of scratch hardness of surface according to Mohs)"
UNI EN ISO 10545 – 7:2000 "Piastrelle di ceramica – Determinazione della resistenza all'abrasione superficiale per piastrelle smaltate (Ceramic tiles – Determination of resistance to surface abrasion for glazed tiles)"
UNI EN ISO 10545 – 13:2000 "Piastrelle di ceramica – Determinazione della resistenza chimica (Ceramic tiles – Determination of chemical resistance)"
UNI EN ISO 10545-14:2000 "Piastrelle di ceramica - Determinazione della resistenza alle macchie. (Ceramic tiles – Determination of resistance to stains)"

Service life of building components. Analysis and proposals of definition of the modifying factors.

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ABSTRACT

In the paper the factor method, introduced by the code ISO 15686-1, is examined in a critical way. In particular, the analysis focuses on the reference service life as well as on the number, the value assessment and the nature of the factors that affect the reference service life in order to identify the peculiarity of the case at stake.
The paper offers a chance to underline the common features and, mostly, the differences between the factor method mentioned above and the method for the evaluation of the behaviours on service of building elements, called NIC method, developed by the Department of Building Engineering – University of Naples Federico II.
Emphasis is put on the comparison between the values of reference of both methods (the Reference Service Life and the Mid-Normal value of duration) and on the experimental evaluation of the mid-normal value when the NIC method is adopted.
With respect to the modifying factors, instead, we underline the fact that in the method herein discussed, as opposed to the factor method, the agents of influence considered refer to a purely physiological deterioration of the components. Furthermore, the agents of influence are redefined and vary from one another and the factors have a different degree of relevance, in the formula of the method, in accordance with their actual influence on the deterioration of the same component.
Finally, we stress the importance of the methods of evaluation of the numeric values of such factors which could derive from experimental data, from laboratory tests or from field-collected data, in order to provide scientific grounds to the definition of the factors of correction of the value of reference.

KEYWORDS

Service Life, Durability, Modifying factors, Degradation agents, Factor method.

1 INTRODUCTION

The Factor method, introduced by the code ISO 15686-1, allows an estimate of the service life to be made for a particular component or assembly in specific conditions. It is based on a reference service life (RSL) and a series of modifying factors that relate to the specific conditions of the case.
The method uses modifying factors for each of the following:

- A quality of components
- B design level
- C work execution level
- D indoor environment
- E outdoor environment
- F in-use conditions
- G maintenance level

Any one (or any combination) of these variables can affect the service life. The factor method can therefore be expressed as a formula in which:

$$ESL = RSL \times A \times B \times C \times D \times E \times FxG$$

2 SOME OBSERVATIONS ABOUT THE FACTOR METHOD

The starting point of the factor method is the reference service life. It is the service life that a building or parts of a building would expect (or is predicted to have) in a certain set (reference set) of in-use conditions. The RSL may be based on the followings:

- *data provided by a manufacturer, a test house or an assessment regime (for innovative components it will normally be based on the manufacturer's or supplier's exposure results); this may be a single figure or a distribution of typical performance;*
- *previous experience or observation of similar construction or materials or in similar conditions;*
- *some books which are available and which include typical service lives;*
- *building codes which may give typical service lives of components*.

Wherever possible it should be as reliable and detailed as possible, and thus preferably based on a rigorous service life prediction. Furthermore, the reference case on which the reference service life is based upon should be chosen to be as similar as possible to the specific case studied in terms of the service life conditions. The factors applied in order to take care of any deviation from the conditions of the reference case will thus be as close to unity as possible, thereby minimizing the inaccuracy introduced by the factors.
The choice of values to use as modifying factors may be based on tests or experience from previous use. If the conditions prevailing in a specific case have led to early failure or to an extended service life, similar conditions elsewhere may be used as the basis for applying a modifying factor.
The advantage of the factor method is that it allows everything that is likely to contribute to variations in service life to be examined at the same time and the relative importance of each to be considered [*Jernberg et al. 2004*].
At the same time this condition represents, for another aspect, a disadvantage because all the factors in the formula have the same weight on the degradation of the considered building element.
Furthermore the alack of a different weights of the factors involves that the combination of small modifying factors can have a significant effect overall. That's particularly important when the degradation is affected by a combination of factors, because even a little variation of more factors from the reference case leads to a substantial variation of the service life estimation.
Moreover the situation in which many factors are different from 1 is very frequent because they represent the deviation of too many different aspects, from the assumed conditions, which sometimes have a too much different influence on the degradation of the same building element. Factors like

maintenance level or quality of the work execution or design level are difficult to manage and to translate in a comparable numeric value. Their effects on a building element lead to a pathological aging that is hard to estimate and to define.
It is also important to note that many relevant agents, like climatic or atmospheric agents, are grouped in only one global factor (outdoor environment) but their influence (e.g. wind, solar radiation, rain, temperature, pollution, etc) is often so various and not homogeneous that doesn't allow this kind of approximation.
Another important point is the attribution of the numeric values to the factors. This operation is too subjective and arbitrary – in fact in the same situation two different designers, without specific indications or limit-values or defined range of variation of the values, could give too many different results.
A similar observation regards the determination of the RSL. Actually, the method proposes too many solutions and too many ways of determination, which show different importance and reliability and are not homogeneous. The designer can define the reference value, that is the most important element of the formula, on the base of data provided by various available information.
So, it is possible to state that the factor method does not provide an assurance of a service life – it merely gives an estimate based on what information is available, the reliability depending on the accuracy of the input data.

3 THE NIC METHOD

The statement of the factor method is formally similar to a method - called NIC method - for the evaluation of the service life of building components that has been developed in 1998 by Prof. Nicolella working group of the D.IN.E. (Department of Building Engineering) of the University of Naples Federico II.
The NIC method is based on the assumption that the service life of a building component can be estimated in every environmental context, determining the peculiarity of the case as a deviation from a mid-normal value (or reference value), obtained on an experimental basis.
Really, the method provides the possibility to express the performance of the considered component in the specific conditions of the case-study, in respect to the conditions assumed as an average case, to be taken as a reference value.
The formula on which the method is based, representing analytically this condition, is the following:

$$Dpp = Dmn \ x \ \Pi Fi$$

where:
Dpp is the value of the "most probable duration", corresponding to the reliability of the considered building component in the assumed condition of use;
Dmn is the "mid-normal" value of the "duration", i.e. the reliability of the considered building component in special conditions assumed as "mid-normal";
Fi are the modifying factors which, in a preliminary phase, are associated to each group of agents that influence the service life of the considered building component.
Namely, the Dmn value of a specific building component is the statistical mean of data taken from the study of some sample-buildings; these buildings are selected according to specific criteria and associated on the base of similar conditions assumed as "mid-normal" conditions.
The adopted criteria of choice are the following:

- quality and quantity of the available information;
- possibility to carry out monitoring activities;
- characteristic of the previous maintenance operations;
- homogeneity of the influencing agents.

Once the mid-normal value is defined, the crucial point of the method is the determination of the modifying factors (Fi), derived from the agents of influence.

Climatic Agents	Environmental Agents	Configuration Agents	Technological Agents
Temperature	Exposure	Facade characteristic:	Presence of
Daily ΔT	Facing sea	Roughness and Colour	protected elements
Wind	Pollution	Shape	Critical points
Rain	Facing other buildings	Extension	
Snow	Vibrations	Lying	
Humidity			

Table 1. A set of agents of influence

With regard to the assessment of the modifying factors, it is important to consider the agents that influence the reliability and to define for these agents the corresponding conditions of variation, in order to evaluate the most probable duration of each building component.
A very important aspect is that the considered degradation agent, which affect the service life, are only reference to a natural and physiological aging and not to a pathological aging. So, the modifying factors of the NIC method are defined only for climatic (e.g. temperature, precipitations, wind, solar radiations, etc) and environmental agents (e.g. pollution, salt atmosphere, etc) and for configuration issues (e.g. roughness and colour, shape, extension, lying , etc) and the specific technology of the considered building element.
For these agents, we do not define a single global value, instead every climatic agent is given a value and relative weight; this weight can change with the nature of the considered building components.
Factors as maintenance level or quality of the work execution or in-use contitions are difficult to manage and to translate in a comparable numeric value. Their effects on a building element lead to a pathological aging that is hard to estimate and define.
We consider only some important agents and their choice, with relative weights, changes according to the nature of the considered building element. Thus, it is possible that the asset of the agents, and correspondents factors, are different for a roof cover, for example, and a plaster façade, or their weights vary in the method’s formula.

Another important aspect is the determination of the mentioned conditions of variation for the agents affecting the service life. When possible, the conditions of variation were derived from:

- definition and the classification of criteria suggested in technical code (for example, the Italian laws: D.M. 16 January 1996 for the conditions of variation of the wind or the D.P.R. 412/93 for the temperature);
- subdivisions connected to the intensity of the agents and/or to their homogenous behaviours (for example, subdivision relating to daily ΔT temperature or exposure);
- limit-values often calculated with the statistical treatment of field-collected data.

After the definition of the conditions of variation for all the considered agents per chosen element, the following step is the identification of the modifying factors that translate in numbers the deviation of behaviour of the case in object from the reference one.
The modifying factors, per every considered agents, represent the different between the condition relevant to the mid-normal case and the condition of the case in object.

So, the crucial points are:

a) the definition of criteria and modalities to estimate and translate in numbers the variation of behaviour, for the same building component, in different conditions of influence, regarding to the mid-normal case;
b) the determination of the weights of the agents that influence the degradation of the considered element.

As regards the definition of point a), we can follow a criterion similar to the one already applied for the choice of the conditions of variation.

In fact, the evaluation can be made in the following ways:

- empirical attribution of the values based on the statistical elaboration of data, collected on the field from case-studies in which it's possible to point out similar conditions and behaviours;
- empirical attribution of the values based on the analysis of the results of studies and researches on the subject, on the details and the information provided by designers, producers, constructors, technicians, etc, on the precepts of code or technical rules, or on the data coming from other similar sources;
- experimental determination with the aid of laboratory tests that concur to estimate the relativity of the different conditions of variation of the agents of influence regarding the case assumed as reference (mid-normal values), obtained by statistical data collected on the field.

A solution can be the combined application of all the three above-mentioned strategies, so that some values can be defined through statistical information, some through code or laws and/or results of specific studies and some, with reference to certain climatic or environmental agents, on the base of laboratory tests.

AGENTS	SOURCES
Temperature	Laboratory Tests
Daily ΔT	Observation of the behaviour on service
Wind	Observation of the behaviour on service
Rain	Results of studies and researches
Snow	Code provisions
Humidity	Observation of the behaviour on service
Exposure to radiation	Observation of the behaviour on service Laboratory Tests
Facing sea	Laboratory Tests
Pollution	Observation of the behaviour on service
Facing other buildings	Results of studies and researches
Vibrations	Observation of the behaviour on service
Roughness	Observation of the behaviour on service
Colour	Code provisions
Shape	Results of studies and researches
Extension	Observation of the behaviour on service
Lying	Code provisions
Presence of protected elements	Results of studies and researches
Critical Points	Results of studies and researches Observation of the behaviour on service

Table 2. The list of the agents and their relative operating methodologies for the appraisal of the behaviour referred to the mid-normal case, with the indication of the source of the necessary information to justify the choice of the values. These information are relative to the implementation of the method on external masonry covered with traditional painted facade plaster.

As regards the determination point b), we have to detect in percentage the disparity among the different conditions of variation of each agent; these percentages of disparity, under the same values regarding all other agents, correspond to the different duration and behaviour of the same element in the different conditions defined by the considered agent, with reference to the medium-normal case (Dmn).

AGENTS	CONDITIONS OF VARIATION	FACTORS REFERRING TO Dmn
Temperature	Zones A - B	0.85 Dmn
	Zones C - D	1 Dmn
	Zones E - F	0.70 Dmn
Daily ΔT	Zone A	1.20 Dmn
	Zone B	Dmn
	Zone C	0.70 Dmn
Wind	Zones 1-2	1.20 Dmn
	Zone 3	Dmn
	Zones 4-5	0.90 Dmn
	Zones 6-7	0.70 Dmn
	Zones 8-9	0.50 Dmn
Rain	Sheltered	Dmn
	Moderate	0.60 Dmn
	Severe	0.24 Dmn
Snow	Zone I	0.47 Dmn
	Zone II	0.65 Dmn
	Zone III	Dmn
Humidity	R.H. < 85 %	Dmn
	R.H. > 85%	0.50 Dmn
Exposition	South	0.75 Dmn
	South / West – South / East	Dmn
	East / West	1.20 Dmn
	Nord/ West – Nord/ East	0.85 Dmn
	Nord	0.65 Dmn
Facing sea	Salt atmosphere	Dmn
	Neutral atmosphere	1.75 Dmn
Pollution	High	0.65Dmn
	Mean	Dmn
	Low	1.20Dmn
	No present	1.60Dmn
Facing other buildings	High protection	1.50Dmn
	Mean protection	1.80Dmn
	Low protection	Dmn
Vibrations	No vibrations	1.65 Dmn
	Intense traffic	Dmn
	Special source	0.30Dmn
Roughness	Smooth surface	Dmn
	Rough surface	0.50 Dmn
Colour	Black	0.47Dmn
	Dark colours	0.50Dmn
	Clear colours	Dmn
	White	1.80Dmn
Shape	Plane surface	Dmn
	Curve surface	0.83Dmn

Extension	< 400 m^2 or with windows > 400 m^2 without windows	Dmn 0.50Dmn
Lying	Horizontal Tilted (45°) Vertical	0.15Dmn 0.20Dmn Dmn
Cornice	No present Present	0.40Dmn Dmn
Balcony	No present Present at intervals on facade with L/H < 0,3 Present on all façade with L/H < 0,3 Present at intervals on facade with L/H > 0,3 Present on all façade with L/H > 0,3	0.80Dmn Dmn 1.40Dmn 1.60Dmn 1.80Dmn
Critical points:	Presence:	
Pillars and slabs aligned with the masonry	Yes / No	0.50Dmn/Dmn
Thickness variation of the masonry	Yes / No	0.50Dmn/Dmn
Thickness variation of the plaster, window mouldings,	Yes / No	Dmn/2.00Dmn
drain-pipe inserted in the masonry.	Yes / No	0.50Dmn/Dmn

Table 3. Conditions of variation and modifying factors for every considered agents of influence of external plaster.

The data in the last column are drawn from the experimental or empirical observation (table 2) of the different behaviours of each building component in the mid-normal case and under the other conditions of variation for the considered agents.
More specifically, one of the opportunities for the definition of the modifying factors is given by the possibility to simulate the effects of some climatic or environmental agents in laboratory.
For example, the value related to the Temperature agent is based on the results of the tests in the experimental laboratory of the D.IN.E.
The D.IN.E. Laboratory is equipped with a climatic cell, to test temperature and humidity, one by one or together, and to test the aging of the components caused by the exposure to sun radiations simulated with a xeno lamp; there is also a dry corrosion cabinet, to simulate changes of temperature and tests the reaction of a component in salt atmosphere or in other very aggressive (i.e., polluted) atmospheres.

4 SOME RELEVANT DIFFERENCES

It is possible to notice that in both methods a reference value is modified by some factors that translate the peculiarity of the specific case. But the origin of this value is different, exactly as the nature of the factors.
Briefly, the most important differences between the methods are:

- the nature of the value of reference (*RSL* and *Dmn*);
- the choice of the agents causing degradation;
- the different weight of the degradation agents in relation to the influence on the service life;
- the origin and the ways of determination of the numerical value of the modifying factors.

For the first point, in particular, the mid-normal value is statistically determined by the mean of data collected on the field in similar conditions.
As regards to the RSL, the ISO 15686 suggests to refer to all possible sources available for the designer, in order to adopt an empirical value. So in our method the way for the determination of the Dmn is well defined.

Whereas for the RSL evaluation, the method proposes too many solutions and too many ways of determination which have different important and reliability and are not homogeneous. Actually, the designer can define the value of reference, that is the most important element of the formula, on the base on data provided by available information (by producers, manufactures, designers), previous experience or observation of similar construction, technical researches or building codes.
A very interesting aspect, in relation to the most evident differences, is the choice and the determination of the degradation agents and the relative modifying factors.
For the NIC method the combination of agents changes according to the considered building component; the weight of the agents, in particular, is connected to the nature of this component. These approaches adapt the method to the different cases and so they concur to guarantee a better reliability in the results.
Moreover, the degradation agent, which affect the service life, are only reference to a natural and physiological aging and not to a pathological aging. So the modifying factors of the NIC method are defined only for climatic (e.g. temperature, precipitations, wind, solar radiations, etc) and environmental agents (pollution, salt atmosphere, etc) and for aspects which concern the configuration and the specific technology of the considered building element.
In the factor method, the problem of the attribution of the values to the modifying factors is resolved in the choice among a limited and prefixed series of values which sometimes are too subjective. In fact, without specific input data the designer chooses values of the RSL and of the factors on the base of him experience and so the objectivity and the reliability of the results are too low.
When data, especially the reference service life, largely are based on results from studies carried out according the service life prediction methodology of course the reliability improves.
In our case, instead, the value of the factors is specific for each prefixed condition and is provided by the method and not chosen by the designer.
So in the factor method the designer operates autonomies and arbitraries choices for the determination of the ESL. In the NIC method, instead, the designer selects the conditions of the case in object and, with the factors previously determined and given by the illustrated method, translates the mid-normal duration in the value to estimate.

	FACTOR METHOD	**NIC METHOD**
RSL/Dmn	*Data provided by available information (by producers, manufactures, designers), previous experience or observation of similar construction, technical researches or building codes.*	*Mean of data collected on the field in similar and homogeneous conditions*
AGENTS CAUSING DEGRADATION: *Nature*	*All agents which affect the service life and the same agents for all the components and materials*	*Only agents which produce a natural and physiological aging*
AGENTS CAUSING DEGRADATION: *Weight*	*The same weight for all the agents*	*Each agent has a different weight for different building components*
MODYFING FACTORS	*Numeric values chosen by the designers*	*Prefixed values carried out according the service life prediction methodology (empiric and/or experimental)*

Table 4. The most relevant differences between the Factor method and the NIC method.

5 REFERENCES

Architectural Institute of Japan, 1993*, The English edition of principal guide for service life planning in building,* Architectural Institute of Japan, Japan
BS 7543, *Guide to durability of buildings and building elements, products and components*, BSI, London
De Pascale, A. 2003, *Modifying factors in a method for estimating service life* in Atti del *Management of durability in the Building process*, Milano 25-26 Giugno
De Pascale, A. 2002, *Modifying factors of a method for estimating service life* in Atti del 9th International Conference on *Durability of Building Materials and Components,* Brisbane Convention & Exhibition Centre, Queensland (Australia)
De Pascale, A. 2000, *Determinazione sperimentale del ciclo di vita utile di elementi costruttivi ai fine della programmazione di interventi di manutenzione*, Tesi di dottorato in "Tecnologie per il recupero edilizio e l'innovazione tecnologica", D.IN.E., Napoli
Di Giulio, R. 2001, *Manuale di manutenzione edilizia*, Maggioli, Milano
Di Giulio R. 1995, *Qualità edilizia programmata*, Hoepli, Milano
Hovde, P.J. 2004, *Factor methods for service life prediction,* CIB Publication, Rotterdam
ISO 6241, *Performance standars in building – Principles for their preparation and factors to be considered*, ISO, Geneve
ISO 15686, *Buildings and constructed assets — Service life planning - Part 1: General principles,* Geneve
ISO 15686, *Buildings and constructed assets — Service life planning - Part 2: Service life prediction procedures*, Geneve
Jernberg, Sjöström, Lacasse, Brandt, Siemes, 2004, *Guide and Bibliography to Service Life and Durability Research for Buildings and Components*, Joint CIB W080 / RILEM TC 140, CIB Publication, Rotterdam
Lucchini, A. and Wyatt, D. P. 1999, *A systematic vivendi for design for durability* in Atti del 8th International Conference on Durability of Building Materials and Components, Vancouver, Canada.
Lucchini, A. 1996, 'Durabilità', Proc La manutenzione degli edifici, UNI, Milano
Lucchini, A. 1989 A., *Metodologia per la valutazione dell'affidabilità di elementi tecnici edilizi fuori sistema e problematica della durabilità*, Tesi di dottorato in "Ingegneria ergotecnica edile", Politecnico di Milano, Milano
Maggi, P.N. 1995, *Controllo e qualificazione dell'attività manutentiva*, Progetto Leonardo, Bologna
Maggi, P.N. *et al*. 2000, *La qualità tecnologica dei componenti edilizi: la durabilità*, Epitesto, Milano
Maggi, P.N. *et al*. 2001, *La qualità tecnologica dei componenti edilizi: la valutazione della durabilità*, Epitesto, Milano
Nicolella, M. 2004, *Programmazione degli interventi in edilizia*, UNI, Milano
Nicolella, M. 2003, *A methodological hypothesis for estimating service life*, in Atti del Convegno Internazionale *Management of durability in the Building process*, Milano 25-26 Giugno
Nicolella, M. 2002, *Components service life: from field test to methodological hypothesis*, in Atti del 9th International Conference on *Durability of Building Materials and Components,* Brisbane Convention & Exhibition Centre, Queensland (Australia)
Nicolella, M. - De Pascale, A. 2002, *Artificial stone for external facing: pathologies and durability,* Lux ET Lapis 2002, International Conference, Valtice.
Nicolella, M. 1997, *Valutazione semiprobabilistica del comportamento in servizio de elementi costruttivi per la determinazione dell'affidabilità*, Tesi di dottorato in "Tecnologie per il recupero edilizio e l'innovazione tecnologica", Dipartimento di Ingegneria Edile, Napoli
Nicolella, M. 2000, *Affidabilità e durabilità degli elementi costruttivi in edilizia*, CUEN, Napoli
Picone, M. 1984, *Tecnologia della produzione edilizia*, UTET, Torino
Rejna, M.G. 1995, *Valutazione della qualità tecnologica utile dei prodotti complessi per l'edilizia,* Quaderno n° 4 del DISET- Politecnico di Milano

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Experimental approach for service life prediction of wooden materials

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ABSTRACT

Wood is predominantly degraded by organisms. Thus, compared to other building materials, service life of wooden material is influenced by many more factors, which are divided into direct and indirect factors. Climate, geographical position, and construction criteria count to the indirect decay factors.

Besides material inherent properties (natural durability, wood preservatives), wood temperature, wood moisture content, and the presence of certain species of wood degrading organisms are the strongest direct factors influencing service life of wooden building components.

On this account an experimental set up was developed to quantify these direct decay factors: Field tests were performed in European Hazard Class 3 (EHC 3) to determine the influence of macro and micro climate on decay progress and decay factors such as temperature and moisture content of wood. Therefore Scots pine sapwood (*Pinus sylvestris* L.) and Douglas fir heartwood (*Pseudotsuga menziesii* Franco) have been positioned in double layer test devices since 2000.

Thirty-two sites in Europe and the United States representing preferably different climatic conditions were chosen for exposure of the test devices. For all sites, data on climate were available since they were located next to an official meteorological station. This way all relevant climatic factors (precipitation, air temperature, relative humidity, sunshine duration, wind) are correlated with moisture content (MC) and temperature of the samples to be measured once a minute and logged once a day. Furthermore the samples were evaluated respecting to decay and discolouration every year according to EN 252 (1990).

Preliminary results concerning the influence of macro and micro climate show that this approach will provide a sufficient data base for a better understanding of the most important factors determining decay.

KEYWORDS

Moisture content, service life prediction, wood, decay factors, micro climate

1 INTRODUCTION

Wood as a renewable resource shows many advantages and is still an estimated building material. In contrast, the biological decomposition by insects, fungi and other microorganisms is not wanted during service life of wooden building components. Wood is predominantly degraded by organisms. Thus, compared to other building materials, service life of wooden material is influenced by many more factors, which are divided into direct and indirect factors.

Particularly architects, craftsmen, construction engineers and assurance companies have a particular interest in service life prediction. Therefore detail knowledge about decay-factors and –mechanisms is necessary. In the past single biotic and abiotic influences were investigated by different methods. They range from standard lab tests with small specimens [Wälchli 1977; Viitanen 1996] to field tests [Meierhofer & Sell 1979; Suttie & Orsler 1998] and investigations of wooden components in service (Foliente *et al.* 2002]. But indices and standards [Scheffer 1971; DIN ISO 15686 2000; DIN ISO 6241 1984], which were derived from different correlations, have not been very available for service life prediction of wood so far. To predict the service life of wood reliably on practical conditions the existing cognitions are insufficient [Noren 2001].

2 PRINCIPLE

Biological decay of wood is considered in a system of dose/response relationships. Decay of wood is the response of a combination of doses by different influencing factors and is quantifiable. The different factors have a direct or an indirect effect on decay, whereby the indirect factors determine the direct ones. For example the existence of termites is a direct influence on service life of a wooden component, whereas the geographic position has no direct influence. In this case the indirect influence of the geographical position results from the limited geographical extension of termites.

Besides material inherent properties (natural durability, wood preservatives), wood temperature, wood moisture content, and the presence of certain species of wood degrading organisms are the strongest direct factors, which can be seen in the classification scheme of decay determining factors in Fig. 1. Climate, geographical position, and construction criteria count to the indirect decay factors.

On this note the following tasks were found to be essential for service life prediction of wood:
- Find the direct decay factors (=dose)
- Determine dose-response-curves
- Build a mathematical model
- Verify dose-response-curves by calculating and measuring response on indirect decay factors

Therefore an experimental approach was created, which allows to find dose-response-curves over several years. It is based on the measurement of moisture content (MC) and wood temperature, the identification of decay fungi, and the determination of response (extend of decay) by exposing wood samples in European Hazard Class 3 (EHC 3), which means outdoors out of ground contact.

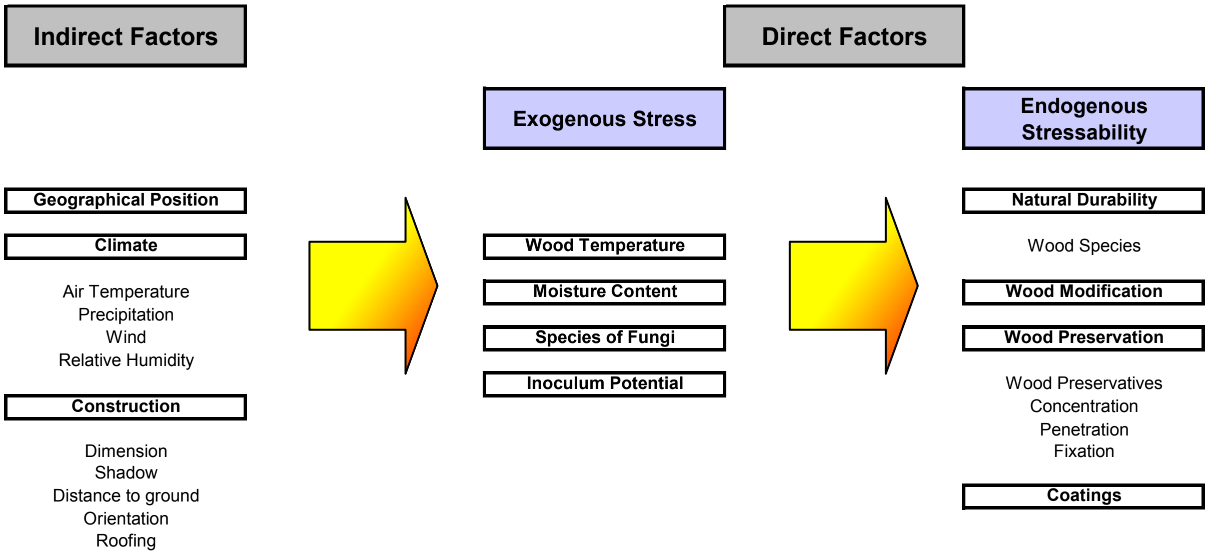

Figure 1 Classification of decay determining factors in European hazard class 3 (EHC 3).

3 EXPERIMENTAL APPROACH

3.1 The double layer test set up

The double layer test method is described in a number of earlier publications (Rapp *et al*. 2001; Rapp & Brischke 2003; Rapp & Augusta 2004) and now a European standard is intended. This test device was developed for testing of wood durability above ground in the field. Therefore the following requirements were defined:

(1) Natural exposure (precipitation, sun, no soil)
(2) Quick infection (by decay fungi in the first year)
(3) Accelerated progress (mean life time 5-6 years)
(4) Realistic moisture content, temperature (similarity to real components)
(5) Objective assessment (even in early stages)
(6) Narrow specimens (for Scots pine sapwood)
(7) Affordable prices (preparation, assessment)

In order to meet the above listed requirements specimens of 500 by 50 by 25 mm³ according to EN 252 (1990) were arranged in a tight horizontal double layer (Fig. 2), supported at the end cuts by beams of Norway spruce with a cross section of 10 by 10 cm², which are covered with a bitumen layer. The whole test set up is heightened 10 cm above ground by the use of paving stones. In case of a non paved ground, a horticultural foliage can be used to avoid the growth of grass. This way a closed wooden deck is formed 20 cm above ground. The upper layer is shifted 2.5 cm lateral to the lower layer.

3.2 Test sites in Europe and the United States

For exposure of the double layer test sets sites with preferably different and typical climates were chosen. Up today thirty-two double layer tests are running in different European countries and the United States since 2000; very different climates are condensed in SW-Germany. Annual mean temperature ranges from 3.3 °C to 13.0 °C, annual precipitation from 500 mm to 2000 mm, and the elevation of the test sites goes from 4 m to 1500 m above sea level. On all sites the availability of reliable climate data was assured by exposing the sets next to an official weather station. Daily precipitation, temperature, sunshine duration, wind speed, and relative humidity were recorded.

Furthermore, some of the sites were provided with a second test set exposed in artificial shadow. Therefore the "shadow sets" were put in plywood boxes covered with water permeable textile sheets (Fig. 2). At the Federal Research Centre for Forestry and Forest Products (BFH) in Hamburg sets were also exposed in a tropical greenhouse during winter, respectively the whole year. This way subtropical (tropical) conditions were simulated.

Figure 2 Shadow created by a plywood box covered with a water permeable textile sheet.

3.3 Dose measurements (MC and wood temperature)

MC of three pine sapwood and Douglas fir heartwood samples in the lower layer of each test set was measured once a day as well as the temperature between the layers. Therefore electrodes of polyamid coated stainless steel cables were glued in predrilled holes of 4 mm diameter in a depth of 25 mm. At the tip of the electrodes the first 5 mm of the plastic coating were removed and put into a conductive glue; the rest of the hole was filled with an isolating epoxy (Fig. 3a).

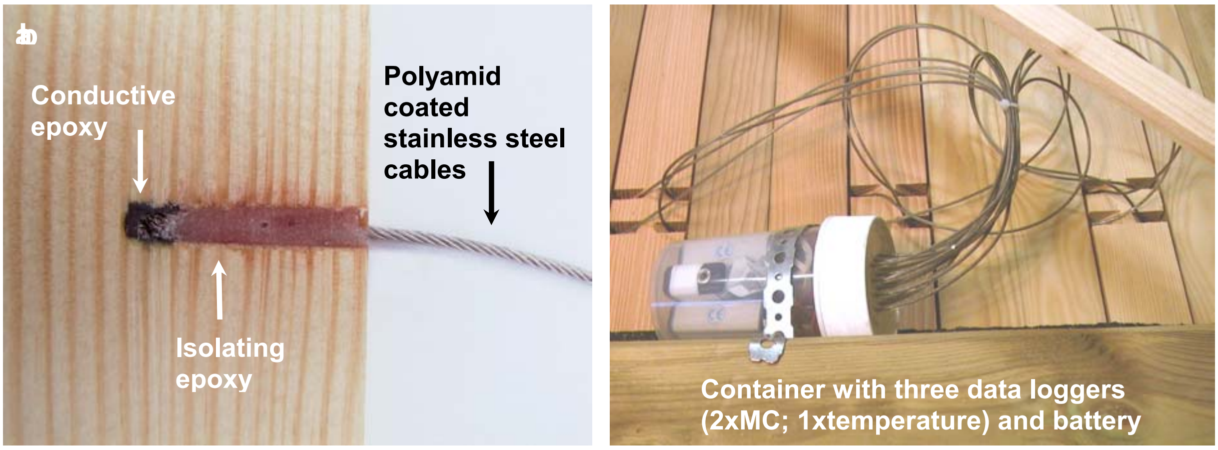

Figure 3 (a) MC measurements with electrodes after gluing in. (b) Bottom side of lower layer with data logger container and electrodes glued in the samples.

The steel cables as well as three temperature sensors were connected with small data loggers, which log the electrical resistance of the wood, minimum, maximum, and average temperature once a day. In earlier studies correlations between the electrical resistance of the wood and MC were found and calibration curves for the data loggers were plotted.

3.4 Response evaluation (Detection of decay)

Evaluation was done yearly by rating the extent of decay according to DIN EN 252 (1990): 0 (sound), 1 (slight attack), 2 (moderate attack), 3 (severe attack), and 4 (failure) as shown in Fig. 4.

Figure 4 Example of severe attack (rating 4 according to DIN EN 252 [1990]).

4 PRELIMINARY RESULTS: INFLUENCE OF MACRO AND MICRO CLIMATE ON DECAY

It was found that artificial shadow increase decay activity on wood by a factor of about 2 compared to open exposure conditions (Tab. 1).

site	*replicates*	*normal*	*shadow*	*factor*
Freiburg	12+12	0,83	1,25	1,5
Stuttgart	12+12	0,42	1,33	3,2
Rhön	12+12	0,42	0,75	1,8
Hamburg	12+12	0,75	1,08	1,4
Mean	48+48	0,61	1,10	2,0

Table 1 Influence of artificial shadow on *Pinus sylvestris* sapwood – Decay rating (0-4) according to DIN EN 252 [1990] after 3 years in EHC 3.

The increase of decay in the artificial shadow box is explained by the occurrence of more non-target organisms, a higher wood moisture content, lower hot-temperatures and higher low-temperatures and by a longer vegetation period (Fig. 6).

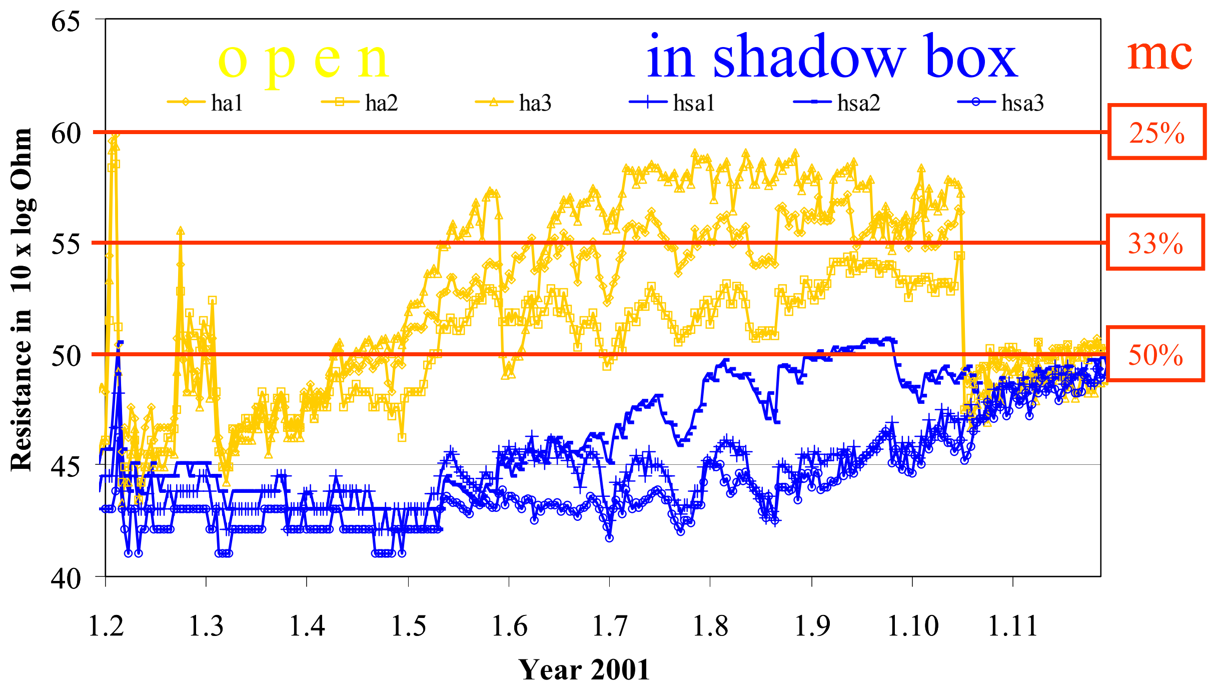

Figure 5 Influence of artificial shadow on *Pinus sylvestris* sapwood – Moisture content (h Hamburg; hs Hamburg shadow box; mc moisture content).

The diagramm in Fig. 6 shows the course of electrical resistance over time. The axis on the right side gives an indication of the wood moisture content corresponding with the electrical resistance.

It is obvious that especially in the summer the curves for pine sapwood in the open double layer (light lines) indicated a lower moisture content than the curves for pine sapwood in the shadow box. (dark lines). In contrast to the samples in the shadow box, the mc of the specimens in the open was not always very favourable for decay.

Furthermore it was found that missing sunlight in the shadow box does not automatically mean lower temperatures. In the beginning of winter the wood temperature in the open was falling down to – 8°C while the temperature of the wood in the shadow box did not fall below 0°C. Only when it became colder, then also the temperature of the wood in the shadow box felt below 0°C. If once the wood in the shadow box is frozen, then it stays. This can be understood by regarding the course of temperatures in spring (Fig. 6).

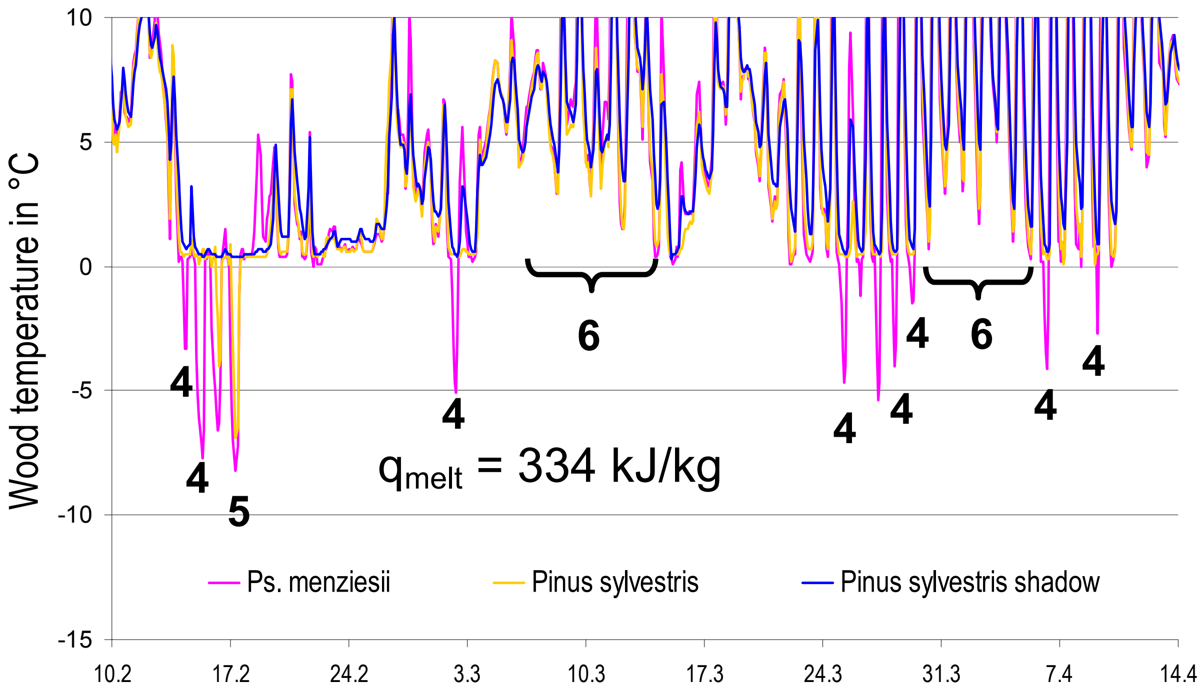

Figure 6 Course of wood temperature in the open and in the shadow box in spring (2h-readings).

The driest wood specimens (Douglas fir heartwood) went below 0°C most often (see mark 4 in Fig. 6). The temperature of the wetter pine sapwood specimens went only once below 0°C (see mark 5 in Fig. 6), when air temperature was very low for one week. And the wettest specimens, pine sapwood in the shadow box never went below 0°C during the whole spring of year 2001 in spite of having many days with minus temperatures as can be seen by the Douglas fir curve (mark 4 in Fig.6).

The explanation for this is simple. It is the thermal energy released while freezing, this keeps the temperature of the wood up. The more water is in the wood the more thermal energy is released. Per kg water to freeze 334 kJ are liberated which prevents the wood from going below 0°C. So the more water is in the wood, the higher is its temperature in winter.

And there is an additional effect which can be seen in cold nights, but not very cold nights, in spring and autumn. See for example the beginning of March 2001(mark 6 in Fig. 6). Here, with temperatures well above the freezing point, still the wood in the shadow box has the smallest drop of temperature during cold nights. This because the textile cover sheet keeps the falling cold air away from the specimens. It is the same principle as applied in agriculture to prevent crops from night frost or cold night temperatures in spring and autumn just by covering it with a sheet. Consequently, the wood covered with the textile sheet in the shadow box always stayed at much higher temperatures at night as wood under open sky. The same effect was also seen in summer (compare mark 6 in Fig. 7 and Fig.6). In summer the night temperatures in the covered box always stayed several degrees higher (sometimes up to 5°C) compared to the specimens under open sky.

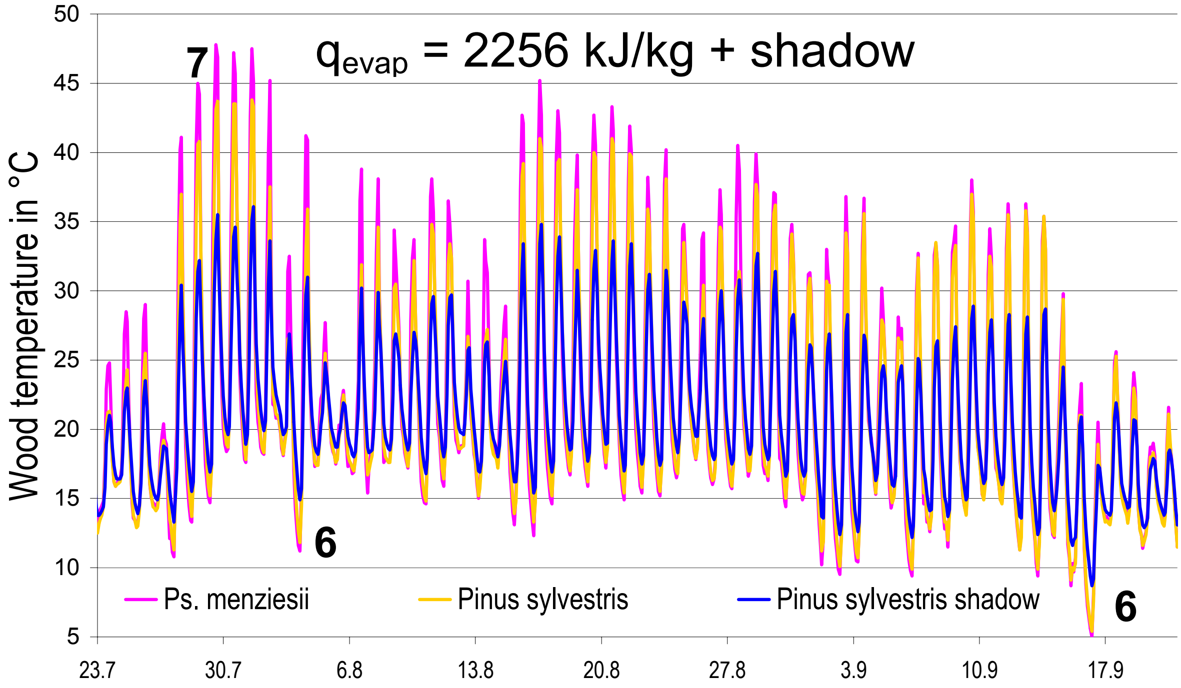

Figure 7 Course of wood temperature in the open and in the shadow box in summer (2h-readings).

About noon on hot summer days the wood temperature in the box seldom climbed higher than 35°C, whereas the uncovered specimens had more than 45 °C (see mark 7 in Fig. 7). There are two causes for this effect:

1) Prevention of direct sun on the samples covered with the textile sheet acting as an awning.
2) The difference in moisture content between the wetter samples and the drier samples

The second cause becomes obvious when comparing the two samples outside the shadow box: The dry Douglas fir and the wetter pine sapwood. In spite of both samples being in the same bright sun, the wetter pine specimen stayed considerably cooler than the dry Douglas specimen (see mark 7 in Fig. 7).

The explanation is simple: High mc means low temperature, because of energy consumption for evaporation of water. For one kg of water the energy of 2256 kJ is needed, this keeps the wood temperature low. Consequently when the pine sapwood (light line) is as dry as the Douglas fir (e.g. after long sunny periods such as in the middle of September), then there is no difference between the highest temperature of Douglas and the highest temperature of pine anymore. However, the pine in the shadow (dark line) always stayed more wet than the samples outside the shadow box and therefore always had lower high-temperatures.

The numerical results over nearly one whole year of 2-hourly logged temperature readings (Tab. 2) can be summarized as follows:

- The amplitudes of temperature in the shadow box are much narrower than under open sky.
  This means much less extremes: less hot days and less cold nights.
- The driest wood specimen had the tallest amplitudes.
  This means the highest amplitudes = hottest days and coldest nights.

Value	*Ps. Menziesii*	*P. sylvestris*	*P.sylvestris shadow*
no. of readings > 35°C	122	79	4
no. of readings > 25°C	258	203	56
no. of readings < 0°C	182	109	50
no. of readings < -5°C	79	49	20
no. of readings < -10°C	11	10	0

Table 2 Numerical evaluation of 2-hourly data logged temperature readings over nearly one year.

5 CONCLUSIONS

The most important direct decay factors, *i.e.* moisture content and wood temperature were recorded. In combination with an automatically working data logging system the double layer test was found to be a very suitable tool for providing a sufficient basis of dose and response data. Therefore an adequate number of samples was exposed and many different climates were covered within this approach. In a second step it should be possible to find some substantial dose-response-function, which are the base for future mathmatically modeling. Preliminary results from the running experiments proved the suitability of this approach:

Shadow increased decay activity on wood by a factor of about 2 compared to open exposure conditions, which means only half of the service life in the shadow. The increase of decay found in the artificial shadow box is explained by the occurrence of more non-target organisms, a higher moisture content, lower hot-temperatures, higher low-temperatures and by a longer vegetation period.

6 REFERENCES

DIN EN 252, 1990, `Freiland-Prüfverfahren zur Bestimmung der relativen Schutzwirkung eines Holzschutzmittels im Erdkontakt´.

DIN ISO 15686, 2000, `Building and constructions assets – service life planning´.

DIN ISO 6241, 1984, ´Performance standards in building – Principles for their preparation and factors to be considered´.

Foliente, G.C., Leicester, R.H., Wang, C.-H., Mackenzie, c. & Cole, I. 2002, `Durability design for wood construction´, Forests Products Journal **52**[1], 10-19.

Meierhofer, U. & Sell, J. 1979, `Physikalische Vorgänge in wetterbeanspruchten Holzbauteilen. 3. Mitteilung: Träger mit direkter Wetterbeanspruchung´, Holz als Roh- und Werkstoff 37: 447-454.

Norén, J. 2001, `Assessment and mapping of environmental degradation factors in outdoor applications´, Diss., KTH, Stockholm, 43p.

Rapp, A.O. & Augusta, U. 2004, `The full guideline for the "double layer test method" – A field test method for determining the durability of wood out of ground´, Stockholm: The International Research Group On Wood Preservation, IRG Document No: IRG/WP 04-20290, 24p.

Rapp, A.O., Augusta, U. & Peek, R.D. 2001, `Facts and ideas of testing wood durability above ground´, Proc. COST E22 meeting in Reinbek, Germany, 9 November 2001, 5p.

Rapp, A.O. & Brischke, C. 2003, ´The influence of macro and micro climate on decay´, Proc. Final Workshop COST E22 in Lisboa, Portugal, 22 23 March 2004, 9p.

Scheffer, T.C. 1971, `A climate index for estimating potential for decay in wood structures above ground´, Forests Products Journal **21**[10], 25-31.

Suttie, E.D. & Orsler, R.J. 1998, `The natural durability assessments of secondary hardwod species – field trials´, Stockholm: The International Research Group On Wood Preservation, IRG Document No: IRG/WP 98-10297, 13p.

Viitanen, H. 1996, `Factors affecting the development of mould and brown rot decay in wooden material and wooden constructions´, Diss., The Swedish University of Agricultural Sciences, Department of Forest Products, Uppsala, 58p.
Wälchli, O. 1977, `Der Temperatureinfluß auf die Holzzerstörung durch Pilze´, Holz als Roh- und Werkstoff 38, 169-174.

Estimation of the Performance over the Time Function for the Protective Epoxy Coatings Executed on Concrete Products

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ABSTRACT

The aim of the work was elaboration of the prognosis procedure for the estimation of changes in the coatings' protective properties (e.g. performance over the time function).
Degradation of two epoxy coatings executed on cement mortar base was investigated. As the measure of degradation, changing in the permeability to: water vapour, CO_2 and chloride ion were determined. Determinations of these parameters were done before and during the accelerated weathering exposition of samples in the laboratory (from 20 to 160 days). The main degradation factors were: temperature, humidity, xenone-arc radiation and as a special chemical factor in the second test was sulphur dioxide.
The analysis of obtained results enabled estimation of epoxy coatings, probable weathering/ageing mechanisms. The empirical equation describing the kinetic of the process of weathering/ageing of investigated coatings was proposed.
Supposing threshold values for the investigated properties was determined service life of tested coatings.
The findings of accelerated laboratory weathering tests were compared with the results of the short-term (450-days) exposure to direct weathering.

KEYWORDS

prognosis procedure, service life, coatings

1. INTRODUCTION

The role of protective coatings for concrete surfaces of construction works is the most limitation of aggressive substance penetration into the concrete from surrounding environment. Required performances of coatings for protection of concrete and reinforced concrete works mostly result from their function and in-use conditions, as well as from aggressiveness of environment [Ściślewski 1995]. From protection of construction work point of view, coatings shall limit the penetration of aggressive substances as long as it is possible, that is - for the longest period effectively protect concrete (and concrete cover).
During the usage coatings are subjected to ageing and their protective effectiveness diminish. The decisive for the rate of this process are: coating structure and resistance of materials to the action of physical and chemical agents during usage[Feldman 1983].
The subject of paper is the trial to estimated the changes of protective properties of two coatings executed on cement mortar base, on the basis of accelerated laboratory tests.

2. TESTS

2.1 The aim of tests

The purpose of described tests was to determine the dynamics of permeability changes in coatings which were applied on cement mortar base, and subjected to weathering exposures.
Accelerated weathering exposures, including physical and chemical agents, were designed to cause the similar coating changes, as they are expected during usage in the atmospheric conditions in the urban environment [Aragon & Frizzi 2002].
Degradation agents in tests were as follows: light simulating of sun light, different relative humidity, negative and positive temperature, water and SO_2.
In such designed processes, the decisive role in degradation of epoxy coatings play the chemical transformations [Feldman 1983]. It may be assumed, that the coating structure modifications, observed in time, are the function of degree of chemical changes, which take place in heterogeneous system: aggressive medium/ polymer coating/ cement mortar base.
Tests covered the determination of changes of water vapour, CO_2 and SO_2 permeability through the coatings, during accelerated laboratory weathering exposures. These parameters were determined also for samples subjected to natural condition in Warsaw.

2.2 Tested coatings

For testing permeability changes in result of natural and accelerated weathering, two epoxy coatings marked EP1 and EP2 were used. Enamels, used for execution of coatings, were the market product, used in construction industry for more than 20 years- in case of EP1, and over 10 years – EP2.
Bases were round- 100 mm in diameter and 5 mm in thickness- made of cement mortar (w/c 0,5; CEM I 32,5; standard sand for strength testing).
Coating EP1 was made from binary solvent enamel, intended for painting of internal surfaces of containers subjected to corrosive action of food products. Coating was made by applying enamel in three layers by brush.
The total thickness of coating after drying was 250-400 μm. Coating was white in color.
Coating EP2 was made from solvent colored enamel, intended for protection of concrete, plaster and other mineral bases against atmospheric effects and against the solutions of bases, acids, salts, sewage, solvents and oils. Coating was composed of ground layer and two overlaying layers. The total thickness of dry coating made on the cement mortar base was 200-350 μm.
Before testing, coatings were conditioned during 21 days in laboratory conditions, at temp. 23 ± 2 °C and humidity 50 ± 5 %, sheltered against natural and artificial light.

2.3 Stand for conducting the weathering exposures

In Department of Durability and Building Protection of BRI, the test stand was arranged for polymer coatings exposure in the atmospheric- simulated conditions.
The test stand was composed of the following devices:
- climatic chamber with controlled temperature and humidity,
- chamber with artificial sun radiation and controlled light intensity from 250 to 765 W/m^2 in spectral range from 300 to 800 nm, and with controlled temperature on sample surfaces,
- climatic chamber for testing in elevated temperature, in wet atmosphere, incorporated SO_2.

2.4 Climatic programs

The coating samples were subjected to the cyclic weathering exposures according to two programs.
<u>Program A - hygrothermal program including the simulation of sun light:</u>
- 2 h radiation with the intensity 400 W/m^2, temp. of black thermometer 40 °C
- 5h freezing in temp. – 20 ± 1 °C
- 17 h conditioning in 23 ± 2 °C and humidity 50 ± 5%
- 3 h in saturated water vapour in temp. 23 ± 2 °C
- 4 h in temp. 40 ± 2 °C and humidity 50 ± 5%
- 17 h conditioning in 23 ± 2 °C and humidity 50 ± 5%.

<u>Hygrothermal program including the simulation of sun light (program A) and chemical operation of wet atmosphere incorporating SO_2</u>
Five cycles were carried out according to program A, in order to activate the coating for chemical operation of wet atmosphere incorporating SO_2. Then 20 cycles of weathering in the chamber with wet atmosphere incorporating SO_2 were carried out (SO_2 dose- 11/3001 of chamber volume). After this part of exposure, the cycles according to program A were carried out again.

The coating samples were exposed also in natural conditions in Warsaw. The exposition lasted 450 days. The mean concentration of SO_2 in the air was 0,0002 μg m^3. The chosen parameters of weathering exposures are given in [Table 1].
After finishing the weathering exposures test samples, sheltered against natural and artificial light, were conditioned for 7 days in laboratory in temperature 23 ± 2°C and humidity 50 ± 5%.

2.5 Test methods

Determination of vapour permeability through the samples of coatings was carried out according to test procedure used in the Building Research Institute accredited laboratory.

Test consisted in the determination of water vapour amount, which in a fixed time penetrate the sample, at the constant partial pressure difference of vapour on both sides of the sample equal 2250 Pa, at temperature 23°C.
Flow of water vapour at the steady state was determined in measurements of mass increment of the vapour absorbing substance. The mass flow of water vapour in the steady state G [g/$m^2\cdot$day] was calculated.

Determination of CO_2 permeability through the samples of coatings was carried out according to test procedure used in the Building Research Institute accredited laboratory.
Test consisted in the determination of CO_2 amount, which in a fixed time penetrate the sample, at the constant partial pressure difference of CO_2 on both sides of the sample equal 10 % vol., at temperature 27°C.
Flow of CO_2 at the steady state was determined in measurements of mass increment of CO_2 absorbing substance. The mass flow of CO_2 in the steady state G [g/$m^2\cdot$day] was calculated.

Determination of chloride ions permeability through tested coatings was carried out according to the

method, which was worked out in the frame of doctor thesis [Sokalska 2002]
Test consisted in the determination of the amount of chloride ions, which in a fixed time are penetrating the coating sample, located in the system of diffusion chambers.
The relationship between total mass of penetrating chloride ions in function of time was determined and chloride ions diffusion coefficient was calculated.

Number of cycles	*Days*	*Radiation*	*Number of "0°C" passages*	*Time of action*				
				temp. -20°C	*Wetting*	*temp. 40°C*	*Normal condition*	*Chemical action SO_2*
		kJ/m²		*[h]*				
Program A								
10	20	28800	20	45	30	60	340	
28	56	80640	56	126	84	168	952	
40	80	115200	80	180	120	240	1360	
80	160	230400	160	360	240	480	2720	
Program A with SO_2 action								
10	30	28800	20	45	410	220	340	160
20	60	57600	40	90	540	280	680	160
50	120	144000	100	225	630	460	1700	160
Natural condition exposure								
Days	*Radiation*	*Number of "0°C" passages*	*Temp.*					
			min	*max*	*Over 24°C*	*Below 0°C*	*With rainfall*	*Sunny*
480	4146 *MJ/m²*	*At least 99*	-20°C	31°C	66 *days*	142 *days*	123 *days*	247 *days*

Table 1 Short description of conditions during weathering exposures

2.6 Test results

Test results are given [Table 2]. On Fig. 1 and Fig.2 the changes of permeability of tested coatings during the accelerated weathering exposure are presented.

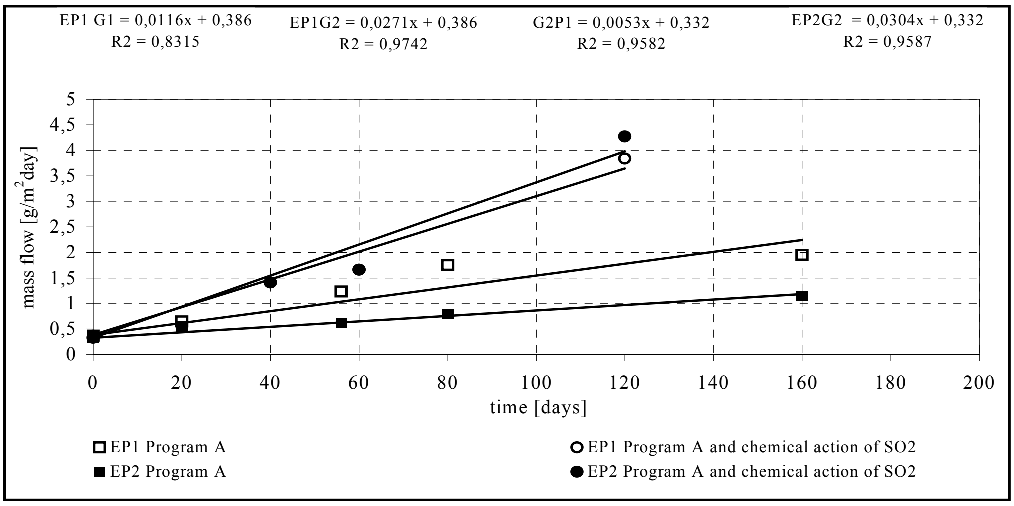

Fig.1 Changes of water vapour permeability through epoxy coatings EP1 and EP2 during accelerated weathering.

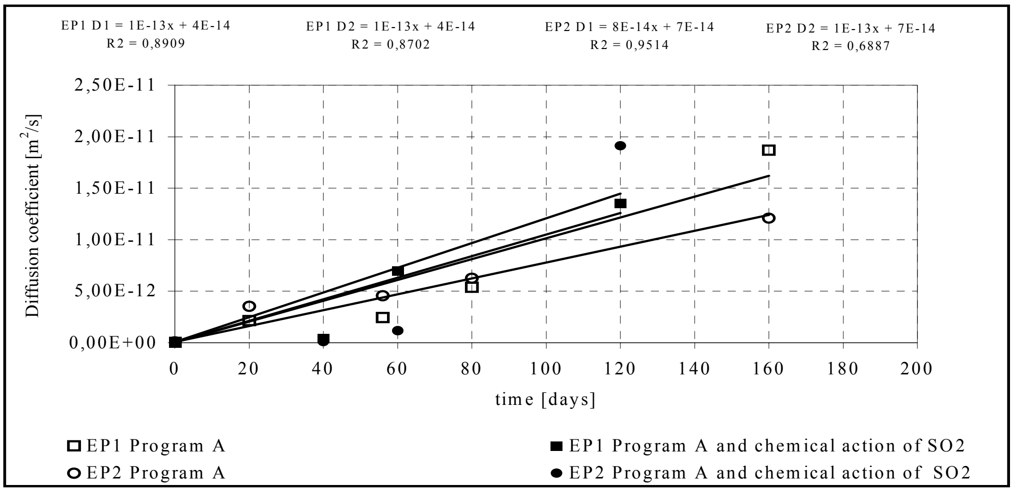

Fig. 2 Changes of chloride ions diffusion coefficients through epoxy coatings EP1 and EP2 during accelerated ageing.

Number of weathering cycles	Days	Permeability		
		Water vapour	CO_2	Chloride ions
		Flow at the steady state		Diffusion coefficient, Cl-
		G [g/m^2*day]	G [g/m^2*day]	D [m^2/s]
EP1 coating (Program A)				
0	0	0,368	0,175	$3,84*10^{-14}$
10	20	0,648	0,590	$2,15*10^{-12}$
28	56	1,236	0,188	$2,43*10^{-12}$
40	80	1,752	1,724	$5,37*10^{-12}$
80	160	1,955	3,015	$1,87*10^{-11}$
EP1 coating (Program A with SO_2 action)				
0	0	0,368	0,175	$3,84*10^{-14}$
10	40	1,412	0,680	$4,10*10^{-13}$
20	60	1,664	4,295	$6,97*10^{-12}$
50	120	4,272	5,508	$1,35*10^{-11}$
EP2 coating(Program A)				
0	0	0,332	0,136	$7,33*10^{-14}$
10	20	0,552	0,680	$3,53*10^{-12}$
28	56	0,620	1,088	$4,55*10^{-12}$
40	80	0,800	1,555	$6,26*10^{-12}$
80	160	1,152	2,570	$1,21*10^{-11}$
EP2 coating (Program A with SO_2 action)				
0	0	0,332	0,136	$7,33*10^{-14}$
10	40	1,412	0,656	$1,11*10^{-13}$
20	60	1,664	0,675	$1,16*10^{-12}$
50	120	1,876	4,412	$1,91*10^{-11}$
EP1 coating (natural condition)				
	480	0,950	0,370	$3,58*10^{-12}$
EP2 coating (natural condition)				
	480	0,868	0,505	$3,25*10^{-11}$

Table 2. Permeability test results for epoxy coatings subjected to weathering exposures.

It was found that permeability of coatings increased during exposures. The conditions of exposures had the influence on the dynamics of those changes. It was confirmed by the increase of coatings permeability during exposure according to program A with the chemical action of wet atmosphere incorporating SO_2, in comparison to the changes of this parameter during exposure without chemical action.

Accelerated weathering exposure	*Linear approximation*	*Sigmoid function approximation* $y(t)=1-e^{-at^p}$	
		p=1	*p=1,5*
EP1 coating			
Water vapour permeability			
Program A	G = 0,0116x + 0,386 R^2 = 0,832	G = -4,02E-05x R^2 = 0,820	G = -3,27E-06x R^2 = 0,538
Program A with SO_2 action	G = 0,0271x + 0,135 R^2 = 0,974	G = -9,35E-05x R^2 = 0,975	G = -1,688-05x R^2 = 0,870
CO_2 permeability			
Program A	G = 0,0165x +0,175 R^2 = 0,845	G = -7,80E-06x R^2 = 0,845	G = -6,79E-07x R^2 = 0,883
Program A with SO_2 action	G = 0,0463x + 0,175 R^2 = 0,825	G = -2,19E-05x R^2 = 0,825	G = -2,13E-06x R^2 = 0,768
Chloride ions permeability			
Program A	D = 1E-13x + 4E-14 R^2 = 0,891	D = -5,37E-06x R^2 = 0,891	D = -5E-07x R^2 = 0,978
Program A with SO_2 action	D = 1E-13x + 4E-14 R^2 = 0,870	D = -5,57E-06x R^2 = 0,870	D = -6E-07x R^2 = 0,922
EP2 coating			
Water vapour permeability			
Program A	G = 0,0157x + 0,332 R^2 = 0,6658	G = -1,81E-05x R^2 = 0,958	G = -1,52E-06x R^2 = 0,811
Program A with SO_2 action	G = 0,0304x + 0,332 R^2 = 0,959	G = -1,04E-04x R^2 = 0,958	G = -1,04E-05x R^2 = 0,988
CO_2 permeability			
Program A	G = 0,016x + 0,136 R^2 = 0,974	G = -7,53E-06x R^2 = 0,974	G = -6,29E-07x R^2 = 0,818
Program A with SO_2 action	G = 0,0289x + 0,136 R^2 = 0,788	G = -1,36E-05x R^2 = 0,788	G = -1,41E-06x R^2 = 0,923
Chloride ions permeability			
Program A	D = 8E-14x + 7E-14 R^2 = 0,951	D = -4,11E-06x R^2 = 0,952	D = -3E-07x R^2 = 0,811
Program A with SO_2 action	D = 1E-13x + 7E-14 R^2 = 0,689	D = -6,38E-06x R^2 = 0,689	D = -7E-07x R^2 = 0,852

Table 3. Equations and correlation coefficient of permeability changes in epoxy EP1 and EP2 during accelerated weathering

Considerable scatter of results in the series of test samples makes difficult to assess precisely the observed dynamics of permeability changes in relation to exposure conditions.
Permeability of coatings samples exposed in natural condition for 15 month also increased distinctly.
Permeability test results for coatings being exposed to accelerated and natural conditions were proportional. In case of EP1 coating, the changes in natural condition were corresponding to the changes observed after 30 – 40 cycles of accelerated weathering according to program A.
In case of EP2 coating, the changes of water vapour permeability in natural condition were corresponding to changes after 40 cycles, CO_2 permeability – to10 cycles, and chloride ions permeability – to 80 cycles of accelerated weathering according to program A.

However, observed differences of permeability changes after weathering exposures in natural and accelerated conditions were not such significant to prove the different mechanism of coating degradation during exposures.

To estimate changes in protective effectiveness of tested coatings, on the basis of above presented test results obtained during accelerated weathering exposures, the trial was undertaken using the following approximations:

- linear function $y(t) = at + b$,
- sigmoid functions approximation, describing the degree of chemical changes in

heterogeneous system: $y(t) = 1 - e^{-at}$, $y(t) = 1 - e^{-at^{1,5}}$

Approximations were made in the program EXCEL 2000. Results are presented [Table 3].

Determined equations, starting at the points corresponding to initial values of coating permeability, was characterized by diversified slopes, specifying the dynamics of changes. Correlation coefficients of linear dependence were more diversified and lower than it was in case of sigmoid function $y(t) = 1 - e^{-at^{1,5}}$.

If in the same exposure conditions, dynamics of coating permeability changes was low, and then it increased, the correlation coefficients of linear dependence, in comparison to sigmoid function, should be lower. It is probable the case observed during weathering exposure carried out in this project.

3. CONCLUSIONS

Tested epoxy coatings, executed on cement mortar base, are degradable in the accelerated and natural exposure conditions. As a result, the changes of permeability are observed. Permeability of coatings expressed by mass flow or by diffusion coefficient was the indicator of degradation, which enables quantitative estimation of protective properties changes in function of time, due to environmental agents, during exposure of epoxy coatings in accelerated and natural conditions.
Assuming that modification of coating structure is a function of chemical reactions during weathering process, sigmoid function, describing the degree of chemical changes in heterogeneous system, can be used for approximation of accelerated test results. Empirical sigmoid function, was characterized by satisfactory correlation coefficient in the range of (0,85-0,99), in the majority of permeability changes estimated in this paper.
Using the empirical sigmoid function for the description of epoxy coatings permeability changes, it is possible to forecast the changes of threshold values of coating protective properties. As a result, assuming the required values for permeability, the time after which the coating loses its protective effectiveness in relation to concrete base can be determined.

4. REFERENCES

Ściślewski Z. 1995., *Trwałość konstrukcji żelbetowych*, Prace naukowe ITB, Seria: Monografie, Warszawa.
Dorel Feldman 1983, *Polymeric Building Materials*, Elsevier Applied Science, London and New York, pp. 496-556.
ISO 15686-2:2001 Buildings and constructed assets -- Service life planning -- Part 2: Service life prediction procedures.
Aragon M. A., Frizzi E. 2002, Correlation Between Natural and Artificial Weathering of Anticorrosion Paints: Analysis of Some Artificial Weathering Cycles, PCE, January 2002, pp. 8-14.
Bretsznajder S., Kawecki W. Leyko J., Marcinkowski R.1973, *Podstawy ogólne technologii chemicznej*, WNT, Warszawa, 1973.
A. Sokalska 2002, Ocena skuteczności powłok ochronnych konstrukcji żelbetowych przed oddziaływaniem jonów chlorkowych, rozprawa doktorska, ITB, Warszawa 2002.

Hwelett P.C. 1990, Methods of Protecting Concrete – Coatings and Linings, Protection of Concrete, Proceedings of the International Conference, held at University of Dundee, Scotland, UK, 1990, pp. 105-134.

The contribution of FMEA and FTA to the performance review and auditing of service life design of constructed assets

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ABSTRACT
The need to demonstrate sustainable practices in all future constructed works suggests that the Performance Review and Performance Audit Process will grow in importance: As will demonstrating Facility and Constructed Asset life time design value and the elements and systems components, proposed service life care plans, projected whole life costs and end of service life scenarios.

Establishing such responses are however difficult, because life time management is in its infancy. Yet the user's experience and the opportunity to draw from time tested products in a wide range of construction environments remains to become an integral part of design decision making. It is against this back ground, that the Paper discusses the working and service life in the context of durability. The Paper suggests that performance, reliability and serviceability should be considered in a relational framework and addresses end of life evaluation and acceptance of fault tolerance in service life planning. In consequence, it is proposed that an additional attribute is adopted in service life planning, that of mistake proofing.

Mistake proofing in effect would be used to demonstrate a building design's capability over time, of meeting the six essential construction product directive requirements and justification of its maintenance or life care plan proposed together with the projected whole life costs.

The Paper considers three responses for design mistake proofing, firstly, using the factor approach, secondly, failure mode effect analysis (FMEA) and finally, fault tree analysis (FTA). In doing so, the Paper suggests that the forward pass of the performance review should focus upon adopting FMEA (as it is a process that offers an established structure in the anticipation and responses to life time matters for all parties involved in a project), whilst for the performance audit, (the backward pass), should focus on the verification of the mistake proofing by adopting FTA. The Paper concludes, by proposing a generic model for mistake proofing focusing on the site's attack regime and constructed works and its configuration's breakdown structure.

KEYWORDS Performance review, mistake proofing, failure mode effect analysis, fault tree analysis, life care plan.

1 INTRODUCTION

Today the proposed or stated designed life of many constructed works may be shorter than the physical replacement cycle of the building stock they join. In such situations, the physical condition and changing performance requirements over time, may pose serious economic and on going serviceability problems which militate against a sustainable hoped for future. So serious is this issue it suggested that the importance of durability, service life and working life will come to assume a critical dimension in both design and the whole life costing of construction.

Durability with its service life core component is also increasingly important for securing sustainable designed life based buildings. Yet durability must be considered in a defined context, that is to say, not only that period of time in service life planning for life cycle costing, but extended to reflect that understanding of material and behavioral changes over time. Especially in systems and element specifications and details proposed in given conditions with *normal* maintenance and their respective life care and end of life management. But subtle distinctions may exist between the working and the service life, especially where the expression, '*performance over time'*- (itself '*the capability of a material or product to with stand degradation in a given environment')* has to be considered (Cib 2004).

This working life (rather than solely service life) is taken to mean *'that period in which the performance **of the works** will be maintained at a level compatible with the fulfillment of the Construction Product Directive's six established essential requirements (*EOTA 1999). By suggesting this, the period of the working life is conditional and may not be how the service life is presented in service life planning, (specially in the use of the factor method) EOTA and the CPD (1993) also suggest or imply, that the working life *'depends upon its inherent durability and normal maintenance.* Perhaps the durability design objective is to, '*keep the probability of failure within a specified time interval or service life and below a threshold that depends upon on the consequence of failure of a component or system and ultimately optimal life cycle maintenance'.* (Lounis et al 1998) Equally, however, CSN TG Durability, N2 TF 207 N, Section E advises that *'Too an optimistic a view of the contributions of a fully performance-oriented approach to the questions related to the evaluation of durability may lead to serious disappointments in the future':* (TF N 207 1999).

Such distinctions are crucial in the introduction and use of failure mode effect analysis (FMEA), defect mapping (DM) or fault tree analysis (FTA) in service life planning. Especially, if the working life of components and systems used remain in service beyond their stated manufacturer's product life. This is also true where the life care to be afforded reflects *normal* maintenance (i.e. without incurring major costs for repair and replacement (CPD 1993) rather than the actual replacement cycle or end of life considerations expected in an environmental life cycle assessment or a whole life sustainable construction. But in terms of mistake proofing the CPD Guidance Paper F's telling comment is foreseeable actions.

In an end of life (EOL) sense, the working and service life may need re-defining at several levels. If this is the case, some feel for the state of the design's construction beyond the intended service life and the specification and details proposed becomes necessary. In turn, this may lead to developments that result in an acceptance of a service loss as a considered response for the design acceptability and where relevant an acceptable life care plan or risk as part of a design intent in service life planning. In short, adopting a fault tolerant approach beyond the design's service life time of any element or system or their components whilst maintaining the constructed work and its products six *Construction Product Directive's* essential requirements. In turn, this may lead to the need to both establish and acceptance of the relationship of the Reference service life (RSL) and service life (SL) with Reliability (R) and Performance (P) within a context of the working life (WL) as illustrated in Figure 1 below. Especially, where the in-service life period extends beyond the constructed work's design life intended, for example, reflecting the time to either the actual clearance or replacement constructed work or its portfolio EOL strategies. As indicated (Figure 1), durability (D) is seen as being over arching in nature between R and P for any constructed work or design. So the service life and in the manufacturer's context stated, working life (WL) may have several thresholds relating to serviceability relationships (S) as well as over time service losses that require specific maintenance responses- that is unless fault tolerance is accepted

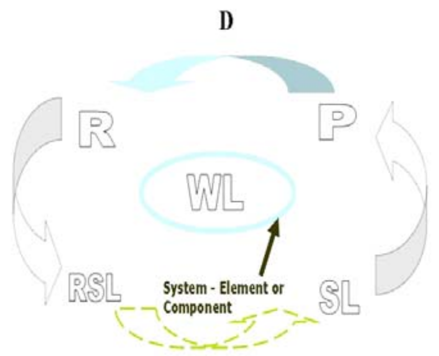

Figure 1 Relationships with the working life

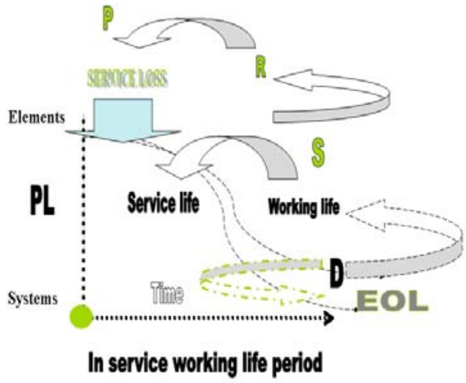

Figure 2 The life time relationships in elements and systems

Such serviceability or service loss may be established against known performances or performance level (PL) over time as suggested in Figure 2. In such a situation, PL becomes a *reference point that* would encourage fault tolerant considerations up to the EOL of a service working life period as well as establishing life cycle costs. But, in pursuing sustainable construction, through service life planning, an additional methodological attribute appears necessary, that of mistake proofing.

2 MISTAKE PROOFING WITH THE FACTOR METHOD

A number of avenues may be pursued for mistake proofing although not necessarily focused upon FMEA, DM or FTA, as follows:

1. A methodological response or framework using a dedicated assessment procedure in flow chart form. For example, in concrete structures (Quillin & Somerville 2002). Here the durability processes involved are mapped and the client is also asked *what constitutes the end of life*? This framework could readily be extended to include FMEA, DM and FTA.

2. A systematic break down of the constructed work into its respective core elements and systems, perhaps as clusters as intimated in Figure 4. Then adoption of a cause and effect extension is possible- but the difficulty of securing quality data should not be overlooked

3. The adoption and development of geographical information systems perhaps on a locality basis together with support from damage atlases as demonstrated by Haagenrud et al (2002).

4. Field based feed back from survey work of a specific construction or element (where probability can be related to common or known failure or recurring problems as proposed by Ansell et al (2002)). Such a practice would assist considerably in mistake proofing design proposals and in improving life care planning.

5. Compilation of a systematic field based DM on an element and system basis as a standard or reference work for use in service life planning design for given constructed work forms or life care management.
6. Otherwise, mistake proofing occurs when comparing a proposal with the likely working life found in practice, e.g. through the experience of a mutual insurer or housing authority who will know where durability problems and other service loss problems have occurred. But a

caution too, for such appraisals may not look beyond a 30 or 60 year period. And in any event, such assessments in cost terms may not necessarily fit the specific focus of a material or component durability, nor life time risks.

In practice many projects do not have high probabilistic needs in design and service life planning so adapting the Factor Approach (ISO 15686-2001) might be used to secure ‘a degree of mistake proofing: Especially for small works, and in areas like low rise housing. So a designer with experience might consider the framework relationships and context, as shown in Figure 3 below. Both to justify their life care plan proposed set against the reference conditions of Factors A-G in the and the design, details and specification as well as the likely consequences in terms of life cycle costs |(LCC) and whole life costs (WLC).

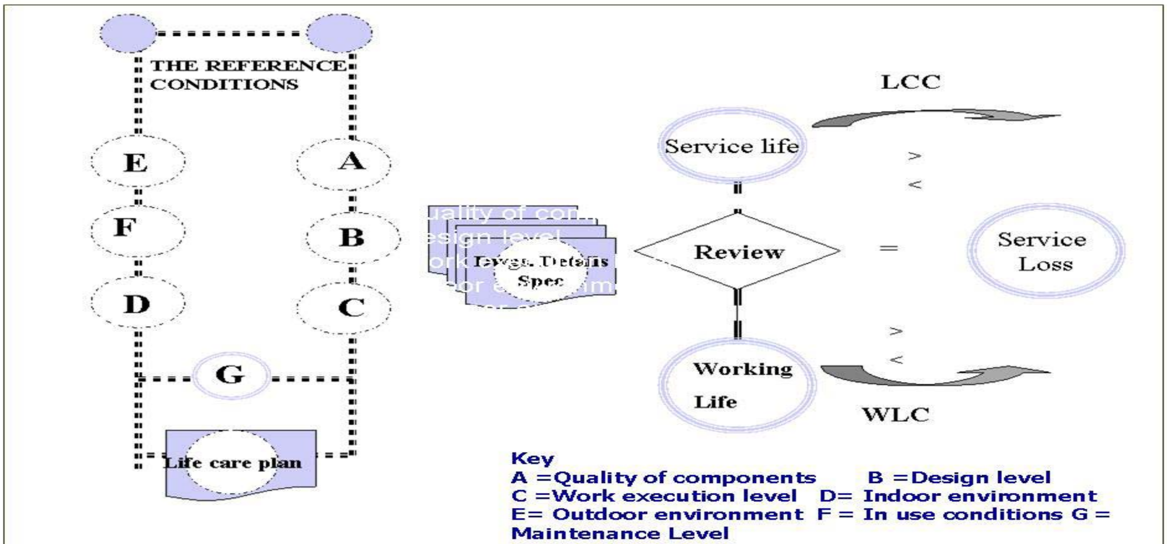

Figure 3 Factor Relationship for Mistake Proofing

The detailed design and specification in Figure 3 means that the technical advice on offer from both National Codes and Manufacturers Guidance should be used. But with a qualitative judgment of those relationships shown in Figures 1 and 2 and any subsequent service life assessment for the proposed design. It is however, the ability to develop the WLC in terms of the service loss anticipated through life over the given life spans and end of life and design life periods that makes robust mistake proofing attractive. Especially, as one can view and treat all components and system as populations with their respective mean time to servicing (MTTS), mean time to repair (MTTR) or replacement or mean time to failure (MTTF) as the case may be. Nevertheless, in Figure 3, in the process of mistake proofing the design and securing an adequate life care plan, (when addressing A, B and C) there is also a need to balance the Factors E, F and D with their support from G. Further, where C is a critical issue, (for example, in insulation and membrane installations), then checking and inspecting needs to be explicit and may have to be also followed up on site. It is also the practice side (See 4 and 5 above) that needs to bought together to ensure that the service life planning process is not re-inventing the wheel, for example, in housing projects using standard systems or house construction specifications.

3 MISTAKE PROOFING THROUH THE PERFORMANCE REVIEW

Lair J & Chevalier (2002) proposed and developed in their paper an integrated design framework with a risk and maintenance focus based on FMEA. In doing so, they suggested that a structural and functional analysis approach through the behavioral analysis of a product and a (roof) element should be adopted. What was particularly important was the focus on improving reliability of a design (as well as being able to apply the approach to the constructed works in use stage).

In essence, this Paper *opened up the way to mistake proofing a design by adopting techniques widely used in product based processes* and the electronic industry. Simply put, one identifies the reference area of the proposed design's elements, systems or components, likely service loss over time, its cause(s), the consequence(s) and risk(s) and where required, EOL status. The core point, however, is that the FMEA presents a structured way of thinking through establishing the likelihood of seriousness of a proposal at a detail level or scaled up to a whole system or element and up to the building level for the working, service or end of life proposed.

It is the ability to establish the construction cluster and context of the constructed work's principal and local relationships that is crucial for whole life assessment. This may require a managed approach for the design team's internal performance review and their data collection. At the same time analysis and modifications to the project design or work packages should coincide at convenient audit stages to avoid a paper work culture. The Principal designer must also make it clear from the client brief onwards that the working life design must take into consideration its attack regime as well as the FMEA, implications and risk. Likewise, the proposed life care plan and its specific intent for systems and components MTTS, MTTR and MMTF must be worked through and costs established.

Concluding, it is believed now that a strong case exists for now making FMEA an integral part of the service life planning review and for in some instances, also adopted for performance audits for example in complex claddings or warm roof construction.

4 MISTAKE PROOFING THROUH THE PERFORMANCE AUDIT

As the constructed work parts and its facility ages, durability and the working life conflict may arise both with its serviceability as well as rising costs or threats of obsolescence. Whilst one cannot fool proof for the real replacement life of the building, one may be able to mix systems and elements to maximize on value and minimize on cost. It is here with the EOL situation now beginning to become a sustainability consideration, that the Performance Audit could identify the working life situation for the constructed facilities systems, like air conditioning, electrical systems and fire detection systems working life times and their associated risks arising from failings or down time.

It is really a question of scale, and seriousness of the likely impact of a short fall in the design, that the balance of judgment has to be made. But, the fault tolerance approach may need to be formally developed. Like wise, the EOL and replacement cycle of the purpose group's population or common systems or elements reviewed from a recovery rather than material waste. At the same time, one should observe too, that the idea of fault tolerant working life is worth considering especially in evaluating the attack regime, and anticipated delays in responding to MTTR and MTTF in practice. So whilst FMEA may be used at a component, element or system level, as the building design progresses to completion and more decisions are fixed, and more information is brought together, a back ward pass may become necessary e.g. for specific design areas. In such instances, there may be a case then to adopt FTA, drawing upon known experience of failures and conditions or characterized the attack regime (e.g. making use of DM). It is useful to consider too Figure 2 and see how the design team has responded to those critical threats of a loss of performance or serviceability, or where reliability is threaten of the core 6 CPD Essential Requirements.

For the third or first party audit (which really is a backward pass), reviewing values, decisions and outcomes and benefits requires clear understanding through a generic break down model of the specific degradation of materials and components that lead to failure: Including those mechanisms, degradation in given environmental conditions and those risk arising from exposure and enlargement. In such a context, the critical events and enlargement need to be identified together with the effectiveness of arresting or terminating such service loss in the life care plan proposed and/or the risk implications.

5 MISTAKE PROOFING WITH FMEA AND FTA

Mistake proofing design proposals may in practice be brought about through the adoption of diagnostic techniques like failure mode effect analysis (FMEA), defect mapping (DM) and fault tree analysis (FTA): (Especially as an integral part of the design review and audit stages in service life planning (ISO 15686 2002)).

FMEA should consider the design and its details on a, what if basis, or, how can this fail? Or, what problems would arise if the construction sequence is incorrect or life care is not fully carried out? For example, if the work under construction is assembled in damp or wet weather or the adequacy of the life care plan itself is not followed nor can not be afforded? At the same time, the Checking, Inspection and Test Plan stage may need to also be included, especially where construction is known to be a cause of early service loss or failure over time if incorrectly carried out.

A wider understanding may by adopting FTA, with or without defect mapping based upon in use experiences of material, component and system failures: (Especially from established service losses, or serviceability failure problems, or associated costs that have arisen in practice). Inevitably here, such failures may include more than one single aspect of a service life component, or material failing, and may extend beyond the point of initial service loss to upwards of a catastrophic failures not originally contemplated in the design, or life care afforded. (Similarly, unexpected or hidden indirect costs not directly associated with the original area of the design itself may also be come to light)

It is particularly in EOL design considerations that FTA has a part to play. In that sense, when FTA is used in performance auditing one would in effect be proof testing a service life design proposal and examining its P, R, S and WLC of ownership and risks.

Concluding, FMEA is appropriate in design work as a forward pass in the service life planning review. FTA may be more appropriate for the backward pass, or proof testing of the in the service life planning through First or Third Party Performance Audits. DM can however, be used in both instances.

6 PROPOSED FMEA AND FTA REVIEW AND AUDIT MODEL

A considerable number of manufacturer's products are remarkably good- especially when the in use experience guidance and good detailing quality come together supported by prudent life care management. Equally, apart from poor specification and detailing or maintenance, one may find where neglect or misuse occurs, significant service loss and often avoidable costs.

It is suggested then, for design, that there is a case for a common forward and backward pass model (Figure 4). Such a model would include the context of the working and service life discussed in this Paper. Here the likely changes and risks of any component failure in an element or system, or other areas threaten would be established. (For example, how will the *six essential requirements, mechanical resistance and stability, safety in case of fire, hygiene health and the environment, safety in use, protection against noise, energy economy and heat retention, and normal maintenance for an economically reasonable working life* be met over varying periods of time, including EOL considerations)

In such situations, to focus on and establish the likely behavior over time of the design prior knowledge (experience and data) is necessary. So Figure 4 represents the bringing together the matters discussed in this Paper within the context of the design's attack regime to be addressed and the need to respond with an appropriate specification, design details, life care and construction method for the life times identified with a balance between working and service life, as shown.

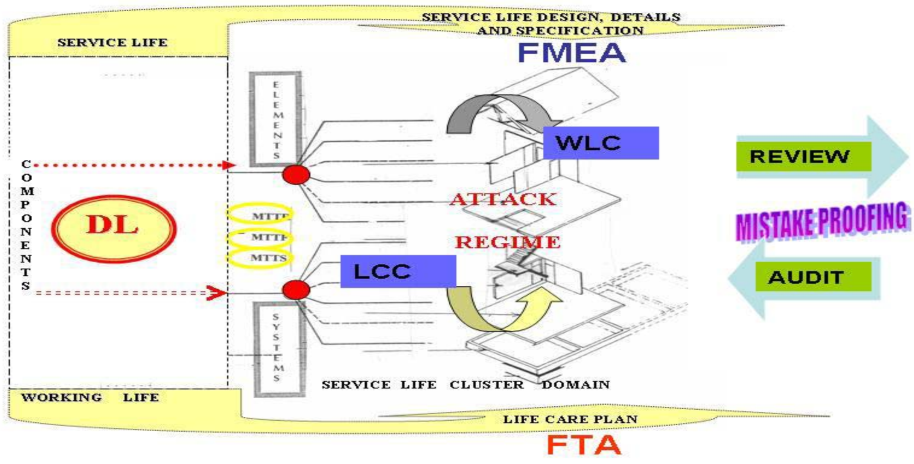

FIGURE 4 FMEA REVIEW AND FTA AUDIT CONTEXT

The site's building location establishes the macro environmental conditions to be designed for and be faced by the specific components and materials to be specified, placed and detailed. But for the given areas of the elevations, layouts and sections a more focused degradation micro and meso-climate regime may exist so the design must also address this. Here such assessments in cost terms, may not necessarily fit the specific focus of a material or component durability. Instead a broader treatment of materials and components may be necessary for given elements and certain key systems that are known to need replacing within the design life (DL) especially within a fault tolerant regime.

The life care plan may include responses from national and legal EOL requirements, as well as the client own requirements. Here through FMEA in order to establish what may need modifying, one may examine the implications of service loss in the service life clusters through their MTTS, MMTR and MMTF. Likewise for sustainable construction all diagnostic approaches have significant relevance for mistake proofing and for EOL Management and climate change scenarios.

DM and FTA may prove useful too, in considering behavioral issues over time and how durability is affected in P, R or S terms: For example, addressing the risks arising from structural movement in a three storey masonry and timber frame block of flats to be built in an expansive clay soil where there is a risk of cracking of the masonry skin, water penetration and eventually possible fungal attack of the timber.

7 CONCLUSIONS

If the EOL becomes part of a brief and or service life planning, the the tools and techniques for reliability and failure diagnostics must become increasingly part of its main stream work. Also as has already been said earlier, because in many cases, the replacement cycle of given building stock suggests a much longer or slow replacement cycle than the design lives being proposed. the issue of the aging effect (Kitsutaka & Matsuyama 2002) needs to be addressed.
It is really the direct application of FTA and FMECA in design that is attractive. Especially in service life profiling and identifying the likely service loss that could arise or their knock on effect and risk. Whilst not specifically focusing upon durability, such approaches nevertheless benefit from a performance review approach, for example based upon Figure 3 or 4, and acceptance of FMEA.

Finally, a method of certification could be developed for elements and systems and should be considered in life time management. (Stenstad & Haagenrud 2001) In consequence, it is believed that the contribution of FMEA and FTA in both the performance review and auditing of service life design constructed assets is now due. Further more it should become an integral part of service life planning and sustainable construction.

8 REFERENCES

Ansell A, Racutanu G & Sundquist H (2002) A Markov approach in estimating the service life of bridge elements in Sweden 9dbmc Proceedings Brisbane Paper 142 Vol. 2 10pp

CiB (2004) Guide and Bibliography to Service Life Durability Research for Building Materials and Components Joint Cib W80/RILEM TC 140 Publication 1-2

CSN TG Durability N2 TF 207 N Section E. Durability Treatment. Contribution to the work of the Task Force CPD/TG on durability 4pp

Construction Products Directive (1993) Guidance Paper F Durability and the Construction Products Directive 8pp

EOTA (1999) Assessment of Working Life of Products EOTA Guidance Document 003 European Organisation for Technical Approval Brussels Belgium

Freitag SA & Bruce SM (2002) Durability Design For New Zealand Concrete Infrastructure 9dbmc Proceedings Brisbane Paper 157 Vol. 2 9pp

Stenstad V & Haageenrud S (2001) Building Certification Measuring The Performance of Buildings Paper CL1 30 Vol. 2 CIB World Congress Performance In Product and Practice. Wellington. NZ 292-298

Haagenud SE, Krigsvo; G, Skancke T, & Gussias A (2002) An integrated GIS based system for service life and maintenance planning of the building stock of Oslo Municipality 9dbmc Proceedings Brisbane Paper 135 Vol. 2 8pp

ISO 15686-(2002) Buildings and Constructed Assets service life planning Part 3 Performance Audits and Reviews Standards Organisation Geneva 23

Kitsutaka Y & Matsuyama Y (2002 Aging Effect of External Building Materials 9dbmc Proceedings Brisbane) Paper 166 Vol. 2-6pp

Lair J & Chevalier JL (2002) Failure Mode Effect and Criticality Analysis for Risk Analysis (Design) and Maintenance Planning (Exploitation). 9dbmc Proceedings Brisbane Paper 032. Vol. 1 10pp

Lounis Z, Lacasse M, Siemes AJM & Moser K (1998) Further Steps Towards A Quantitative Approach To Durability Design Materials and Technologies for Sustainable Construction CIB World Building Congress Symposium A Vol. 1 315-324

Quillin KC, Somerville G, Hooper R & Nixon P (2002) A Framework for Service Life Design of Concrete Structures 9dbmc Proceedings Brisbane Paper 128 Vol. 2 11p

BIBLIOGRAPHY

Hutchinson TW (2002) Integrating Building Components in Roof System Design for Long Term Roof Performance 9dbmc Proceedings Brisbane Paper 146 Vol. 2 10pp

Sjostrom c, Jernberg P, Caluwaerts P,Kelly S, Haagenrud S & Chevalier JL (2002). Paper 10 Implementation of the European Construction Products Directive via the ISO 15686 Standard 9dbmc Proceedings Brisbane Vol .1- 9pp

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Predictive Service Life Tests for Roofing Membranes

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ABSTRACT

Twelve roofing membranes, including poly [vinyl chloride], asphalt glass-felt built-up, thermoplastic polyolefin, atactic polypropylene polymer modified asphalt, styrene-butadiene-styrene block copolymer modified asphalt, and ethylene-propylene-diene rubber membranes were exposed to oven heat, ultra-violet and condensing humidity environment, and two and four-year outdoor exposure at United States Department of Defense sites in Phoenix, Arizona (hot and dry climate); Key West, Florida (hot and moist climate); and Champaign, Illinois (moderate mid-continent climate). Selected mechanical properties were measured before and after each exposure. Each membrane was rated before and after exposure and the membranes were ranked as to relative performance in these physical tests.

The physical tests performed include load-strain, dynamic impact resistance, moisture absorption, and glass transition temperature. Identical test methods were used for each membrane to make the physical properties directly comparable.

With the exception of the TPO (thermoplastic polyolefin) membrane samples, the mean changes in the load to first peak of the membranes at each site had a 0.994 correlation with the before exposure, after oven heat aging, and after condensing ultra violet exposures. The TPO membrane load to first peak more than doubled after two years of exposure and then dropped dramatically after four years of exposure.

The over all results show that testing membrane samples before and after oven heat aging and condensing ultra violet exposures do not accurately predict the final ratings of a dissimilar group of membranes exposed outdoors in a broad variety of climates. These accelerated aging techniques have shown to often be valuable when used to evaluate similar membranes.

The single parameter that seems most useful in tracking weathering is the water absorption test. The average percent water absorbed by the membranes increase in direct proportion to exposure time with a linear coefficient of 0.999.

We hope to be able to conclude this six-year study as soon as the samples finish weathering and report our results to interested parties.

KEYWORDS

Relative durability, roofing membranes, test methods, weathering

INTRODUCTION

This research continues the work previously reported (Cash et al 1993, 2001, 2004 and Bailey et al 2002, 2003). We measured selected mechanical properties and the glass transition temperature of 12 roofing membranes before and after two-year and four-year outdoor exposure at Phoenix, Arizona, Key West Florida, and Champaign, Illinois. We report the unexposed and the exposed data. We used a rating system to enable us to include all the diverse physical values measured (such as tensile load at first peak, water absorption, glass transition temperature, and impact resistance) into a single value for each pre-exposure membrane and each post-exposure membrane modified by the change exhibited after exposure. We then ranked the membranes from the “best” to the “worst” in each exposure used.

Membranes tested

Table 1 lists the 12 popular membranes that are at the core of this on-going study. With the exception of some polymer-modified membranes that were prepared by the manufacturer, the multi-ply bituminous membranes were prepared in the laboratory under controlled conditions.

Sample	Description	Sample	Description
A B	TPO - thermoplastic polyolefin PVCa - poly [vinyl chloride] alloy	G & H	APP - atactic polypropylene - 2 ply polymer modified asphalt
C & D	Asphalt-glass fiber felt BUR - 3 ply with steep asphalt	J & K	EPDM - ethylene-propylene-diene terpolymer rubber
E & F	SBS - styrene-butadiene-styrene 2 ply - polymer modified asphalt	L & M	PVC - reinforced PVC poly [vinyl chloride]

Table 1. Membrane sample designators and descriptions.

Test methods

For a given test, the same personnel tested all the samples using the same equipment under identical conditions in an effort to avoid some of the errors inherent in testing. Table 2 lists the test methods used to measure the parameters we selected.

Parameter	ASTM Method	Test Conditions
Load-strain properties Load at first peak Elongation Energy to first peak	D2523	25 mm (1 in.) wide strips, 100 mm (4 in.) jaw gap, 0.85 mm/s (2 in./min) extension, 23oC (73oF).
Water absorption	D570	1 week in water @ 60oC (140oF).
Glass transition temperature	DMA(D6382)	Change in free volume.
Dynamic puncture resistance	D5635	-18oC (0oF).
Thermal expansion	TMA	-20oC (-4oF) to 90oC (194oF).

Legend: DMA=dynamic mechanical analysis; TMA=thermo mechanical analysis

Table 2. Test methods and test conditions used for these evaluations.

Accelerated exposures

As previously reported (Bailey et al 2003), we tested pre-exposure samples before and after heat conditioning in a forced draft oven at 70ºC (158ºF) for 28 days, and before and after 1500 hours of cycles of 20 hours of ultra-violet light at 60ºC (140ºF) and 2 hours of condensing humidity.

Rating method

We used an arbitrary rating system to estimate the physical condition of each sample. We rated the "best" sample in each test as "100" and the "worst" as "0". As examples: the sample with the greatest tensile strength was rated "100"; the weakest sample was rated "0"; and all the other samples rated linearly in between these extremes. The sample with the highest water absorption was rated "0"; the sample that absorbed the least water was rated "100". We averaged the ratings for each parameter to get a single rating for each sample without using any weighting factors. Table 3 shows the ratings the unexposed samples. These ratings are different from those reported earlier (Bailey et al 2003) because two of the tests, cyclic fatigue and static puncture resistance were not performed on the samples after outdoor exposure. These two tests showed little change in our previous work.

Sample	Rating	Sample	Rating	Sample	Rating
A TPO	31	E SBS	35	J EPDM	71
B PVCa	43	F SBS	69	K EPDM	68
C BUR	28	G APP	45	L PVC	52
D BUR	37	H APP	55	M PVC	57

Table 3. Ratings for unexposed samples.

We rated the change due to exposure. We rated a parameter "100" when a change was not statistically significant and "0" for the parameter that showed the greatest change. We averaged these change ratings with the pre-exposure ratings to obtain the post-exposure rating for each sample after each exposure. We then averaged the pre-exposure rating with the two-year change and the four-year change ratings to find the final rating for the four-year exposure. We then ranked these membrane samples from the one with the highest to the membrane with the lowest rating at each outdoor exposure location.

RESULTS

Space limitations preclude publication of the testing details; the final ratings of each membrane are shown in Table 4. The performances of the membranes in the various parameters tested are discussed in individual paragraphs. Composite ratings: M PVC, H APP, and L PVC are in the top quartile of the rankings at Phoenix and Key West. Samples G APP, M PVC, and H APP are the top quartile of the samples exposed in Champaign.

Samples A TPO, J EPDM, and C BUR occupy the lowest quartile of the samples exposed at Phoenix. Samples H APP, and both BUR samples are at the bottom of the samples exposed at Key West. Samples A TPO, B PVCa, and F SBS were lowest in ranking of the exposures made at Champaign.

Phoenix		Key West		Champaign	
Sample	Rating	Sample	Rating	Sample	Rating
M PVC	83.9	M PVC	82	G APP	80.1
H APP	78.7	G APP	78.6	M PVC	78.7
L PVC	78.6	L PVC	77.7	H APP	75.9
F SBS	77.89	K EPDM	77	J EPDM	73.4
G APP	77.9	J EPDM	74.9	L PVC	71.1
K EPDM	74.6	F SBS	73.6	C BUR	71.1
E SBS	72.6	B PVCa	73	K EPDM	69.4
B PVCa	71.9	E SBS	70	D BUR	68.9
D BUR	66.3	A TPO	66.7	E SBS	68.3
A TPO	65.4	H APP	65.7	A TPO	67.6
J EPDM	59.4	D BUR	64.7	B PVCa	67.1
C BUR	54.3	C BUR	62.7	F SBS	61.7
Mean	71.8	Mean	72.2	Mean	71.1

Table 4. Relative ratings for each membrane after four years of outdoor exposure.

Load to first peak - These data show different weathering patterns. The load-to-peak of the TPO and BUR membranes' increase during the first two years of exposure and then decrease after four years of exposure. In particular, the TPO membranes strength increased an average of 112% after two years of exposure, then lost ~40% of that after two more years of exposure. The load-to-first-peak of the TPO membrane exposed at Key West for two years showed the greatest change.

The PVCs, APPs, and SBSes membranes show a relatively linear decline in strength with exposure time. The changes in average values were modest and may not be statistically significant, but the trend lines are quite linear – some with a least squares regression coefficient of up to 0.998 and one even of 1.0.
The average load-to-peak for EPDM specimens forms a trend line with a slight increase in strength with time of exposure – again, these differences may not be statistically significant.

The changes in load-to-peak were quite consistent with each other (with the exception of the change shown by TPO – mentioned previously). The changes varied with time – not exposure location. There is a 0.994 coefficient of correlation between the average of the unexposed, heat exposed and UV exposed ratings with the average rating for the membranes after four years exposure at all three locations.

Strain to first peak - These data show the BUR membranes and the TPO membrane tend to slightly increase their elongation-at-first-peak after two years of exposure and then decrease thereafter. The balance of the membranes' average elongation declined linearly - the least squares regression coefficients range from 0.940 to 1.0. We found no significant correlation between the average of the unexposed, heat exposed, and UV exposed strain ratings and the average strain rating of the membranes exposed to the weather for four years.

Energy to first peak - These data show the energy-to-first-peak increases after two years of exposure for the TPO, BURs, and the EPDMs; it drops off in the fourth year of exposure. The balance of the membranes shows a linear decline in the energy-to first peak with age at each location. The regression coefficient ranges from 0.931 to 1.0. Again, we found no correlation between the average of the ratings of the unexposed, heat exposed, and UV exposed "energy" ratings and the average "energy" ratings of the samples exposed to the weather for four years.

Water absorption - These data show that the water absorption increases as the weather exposure increases. These increases, for the average of each type of membrane, are linear with exposure time, with coefficients of correlation ranging from 0.82 for EPDM samples and coefficients in excess of 0.99 for the other membranes.. The samples exposed in Phoenix show the greatest increase; the samples exposed in Champaign the least increase. This water absorption test seems to be the most useful in tracking the weathering path of organic membrane samples. We found no correlation between the pre-exposure, heat exposure or condensing ultra violet exposure and the ratings on samples exposed for four years.

Thermal expansion - These data show the thermal expansion coefficient declines as the samples are aged outdoors. The average decline in the thermal expansion coefficient is roughly the same for thee exposure sites, and the decline is relatively linear. There is no correlation between these four-year exposure test data and the accelerated test data performed earlier.

Glass transition - These data show the glass transition temperature gets higher for many of the membranes as they weather. The TPO, PVCs, and BURs show linear increases of approximately 1ºC per year. At this time, age causes little change in the glass transition point of the other membranes. Of special note – we were unable to cut the small specimens required for this test from sample BUR D because the membrane crumpled into dust.

Dynamic puncture resistance - On the average, puncture resistance tends to increase with outdoor exposure time, but many samples show an increase in puncture resistance after two years of exposure, followed by a decline in resistance after four years of exposure. This probably related to the tensile strength that shows an increase after two years of exposure, followed by a decline after four years. There is no direct correlation between tensile strength and dynamic puncture resistance in these samples.

CONCLUSIONS

The following conclusions are preliminary; they may be modified by the additional data obtained from the samples after six years of exposure.

- Neither the unexposed, heat nor UV rankings adequately predicted the rankings of the samples after four years of outdoor exposure.
- The water absorption test appears to be very useful in tracking weather exposure in any of the samples under test. The average water the samples absorbed increased as the outdoor exposure time increased at each location.
- The BUR samples faired very poorly in these ratings, probably due to the lack of gravel or other protective coating on the samples to shield them from the weather.

These rankings are based solely on the change in physical properties measured and may not reflect the long-term weather performance of these products.

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- JPS Elastomerics, Company
- National Research Council of Canada
- Performance Roofing Systems
- Polyglass, USA
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- Tamko Roofing Products
- US Army Corps of Engineers – Engineering Research and Development Center

REFERENCES

Bailey, DM, CG Cash and AG Davies Jr., "Predictive Service Life Tests for Roofing Membranes – Phase II Investigation of Accelerated Aging Tests for Tracking Degradation of Roofing Membrane Materials," USACERL TR-01, April 2002.

Bailey, DM., CG Cash and AG Davies Jr., "Update: Service Life Test for Roofing Membranes," *Roofing Research and Standards Development: 5th Volume, ASTM STP 1451,* W.J. Rossiter and T.J. Wallace, Eds., ASTM International, West Conshohocken, PA, 2003.

Cash, CG and DM Bailey, "Predictive Service Life Tests for Roofing Membranes, Phase 1" USACERL Interim Report FM-94/03, December 1993.

TT4-213, Predictive Service Life Tests for Roofing Membranes, C.G. Cash, D.M. Bailey, A.G. Davies, Jr., A. H. Delgado, D.L. Niles, R.M. Paroli

Cash, CG, DM Bailey and AG Davies Jr., "Predictive Service Life Testing of Low Sloped Roofing Systems, Phase 1" *International Conference on Building Envelope Systems and Technologies,* National Research Council Canada, Ottawa, 2001

Cash, CG, DM Bailey, AG Davies Jr., AH Delgado, DL Niles, and RM Paroli, "Update 3: Service Life Tests for Roofing Membranes," CIB World Building Congress, Toronto, Canada, May 2004

Engineering Models for Biological Attack on Timber Structures

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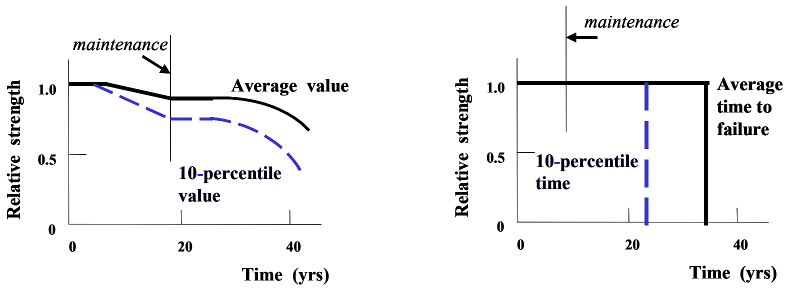
ABSTRACT

A national project nearing completion in Australia has been to develop engineering models for predicting the attack on timber construction by biological agents such as decay fungi, termites and marine borers. Typically such models will be used for predicting structural performance in terms of risk. Examples of such predictions applied to structural strength are shown in Fig. 1. Both the mean and the poorest 10 percentile performance need to be predicted in order to develop probabilistic models of risk.



(a) Attack by decay fungi **(b) Attack by termites**

Figure 1. Examples of required performance predictions.

Basic difficulties that arise in the development of such predictions arise from the fact that (a) quantitative knowledge of the action of biological agents is limited, (b) this action is very dependant on environmental factors and (c) the duration of the durability project is too short to set up meaningful experiments to obtain long duration data.

The models developed have been based initially on the use of existing long-term data obtained from standard field tests using small clear pieces of wood, such as in-ground stakes and above-ground L-joints. These models were then modified using data from full-size structures. Finally, the models were calibrated by information obtained from in-service structures and by expert opinion.

It is believed that this is the first time that such attack models have been developed. Field data from more than 10,000 test specimens and 10,000 in-service measurements were used in the project. The models are suitable for application in conventional structural engineering codes.

KEYWORDS

timber, structural, durability, biological

INTRODUCTION

Whether concern is with bridges, marine structures, power poles or houses, cost-optimised asset management decisions on timber construction rely on being able to make quantified predictions of the incidence and effect of biological attack. A schematic illustration of the procedure for predicting the performance of buildings is illustrated schematically in Fig. 2. Here an attack model is used to predict performance on the basis of input information.

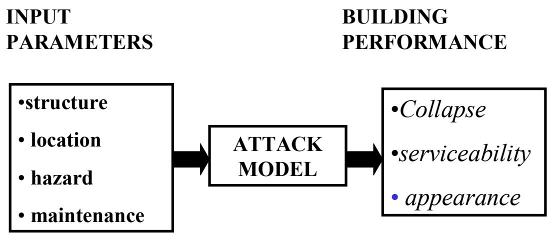


Figure 2. Schematic illustration of procedure for assessing building performance.

This paper will describe the development of reliability models for this purpose. An essential difficulty in making quantified assessments of biological attacks lies in the fact that biologists do not have quantified models of the behaviour of biological agents, and that long term experiments cannot be completed during the course of the current project. The models have therefore been developed on the basis of information derived from a variety of sources. The information used includes the application of physics and biology theory, data from short term laboratory tests, data from long term field trials using small clear pieces of wood, measurements on in-service structures and finally the use of expert opinion.

In the following, lessons learned in the development of the attack models will be described. Some of the models have been described in earlier publications [1–6]. The development of these models has been undertaken as part of a large 7 year program on the engineered durability of timber construction, sponsored and directed by the Forestry and Wood Products Research and Development Corporation of Australia.

DATA SOURCES

Biological models

A fundamental difficulty in modelling attack by biological agents for engineering purposes lies in the fact that while information is available on the type of parameters that influence attack, there is almost no quantified information on the rate of attack. However the biological information does form a very useful starting point.

For example, it is known that the rate of attack by decay, denoted by r, may be written in the form

$$r = A\, f(T, t_w) \qquad (1)$$

where A is a constant related to the type of wood used and $f(T, t_w)$ denotes of function of temperature and the time that the moisture content exceeds the fibre saturation point, typically a value of 28% [7].

Estimates of the form of the function $f(T, t_w)$ may be obtained from biology texts such of that by Zabel and Morell, and the constant A may then be derived through calibration with field test data [4].

Short term laboratory and field tests

By using short term tests to compare the performance of a new product with an existing one, it is possible to make a reasonable estimate of the long term performance of the new product. A procedure for doing this is described in the publication by the Australasian Wood Preservation Committee [8]. In this type of procedure, care must be taken to ensure that the attack mechanism in the short term test is the same as that which would occur under long duration conditions.

Short term tests are also useful for examining the influence of attack parameters, such as for example the response of termites to various environmental conditions.

Long term field tests on small pieces of wood

These tests are relatively inexpensive to undertake and the data from these tests form the basis of much of the modelling of the performance of timber construction. Two of the standard specimens used are shown in Fig. 3. Table 1 gives a summary of the number of test specimens that have been used and Fig. 4 gives an example of the test sites used. The test sites were chosen to cover the climate range of Australia and the test specimens were monitored every two years or so. The use of data from these small clear specimens has been described in previous papers [1–4]. The data has also been used to classify 130 species of Australian timber into four durability classes [9].

Specimen type	Test duration (yrs)	No. of sites	No of specimens per site
In-ground decay	31	5	800
Above-ground decay	11	33	150
marine borer attack	5	3	850

Table 1. Small clear wood test specimens

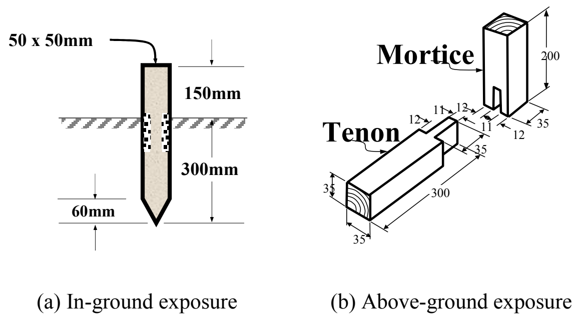


Figure 3. Standard test specimens for assessing the durability of timber.

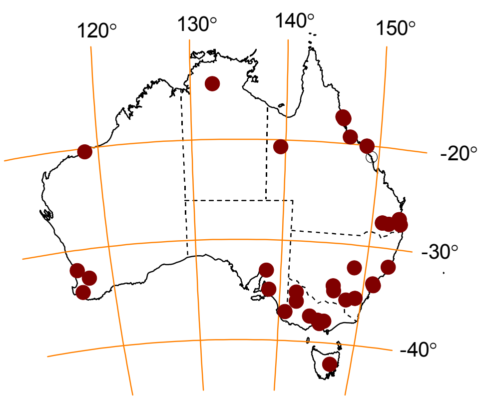


Figure 4. Location of field test sites used to measure above-ground decay.

Long term field tests on full size timbers

Data from tests of this type unfortunately are prohibitively expensive but are extremely useful. For example, one set of tests provided data on some 60 poles that had been placed in ground and carefully monitored for about 25 years. It was found that the use of CCA preservatives led to a markedly different decay rate in the poles in comparison with that observed in small clear pieces of wood [3]. This was probably due to differences in the leaching action of the preservatives for the two cases.

Examination of in-service structures

Since it is not feasible to set up long duration studies within a short term project, it is useful to include the performance of in-service construction as a source of information. Buildings, power poles, bridges and marinas have been found to be useful sources of information.

A summary of the data items obtained during the course of this project is given in Table 2. Some of the difficulties encountered in the use of this type of data arise from the fact that frequently there are considerable uncertainties related to the initial conditions such as the year of installation or the quantity of impregnated preservative. Often there is difficulty in ascertaining the history of maintenance treatments.

Attack type	No. of data points
In-ground decay fungi	230
Above-ground decay fungi	2000
Termites	5000*
Marine borers	4500

* No. of houses

Table 2. Calibration with in-service structures

Expert opinion

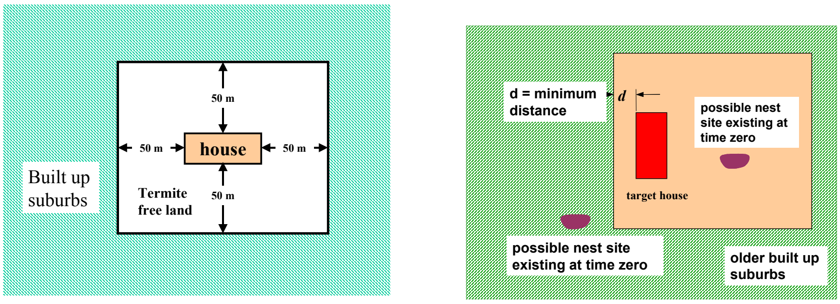
The experienced judgements of experts, such as bridge maintenance engineers, pest management operators and researchers, provide an invaluable method for widening the application range of existing databases. However, for such knowledge to be useful for quantified predictions, the expert opinion must be collected in quantified terms within the framework of a formal questionaire.

TERMITE ATTACK ON HOUSING

The model for predicting termite attack on housing is interesting as an example of the use of expert opinion. For this purpose experts were asked to give estimates of the progression of attack by termites on a house located within a large termite free garden as shown in Fig. 5a. Specifically they were asked to give time estimates for termites to (a) form a mature nest within the garden, (b) travel to the house, (c) breach a termite barrier around the house and (d) enter and destroy some part of the house. The definition of parameters affecting each step, plus an importance rating for each of these parameters were also requested.

Having obtained the expert opinion, it was then possible to draft a quantified risk model that takes into account a more practical scenario such as that shown in Fig. 5b [5,6].

Using this model, a risk estimate can be made on a house by house basis. Figure 6 shows that this risk model, after modification by calibration, provides a very good fit to data obtained for some 5000 houses. An important aspect in interpreting collected data was to take into account the limited memory of the occupants because of their changes of residences. In this case it was assumed that the average occupant would have a memory of a termite attack only if it occurred less than 20 years previous to the survey.



(a) Hypothetical scenario (b) Practical scenario

Figure 5. Scenarios used for modelling termite attacks.

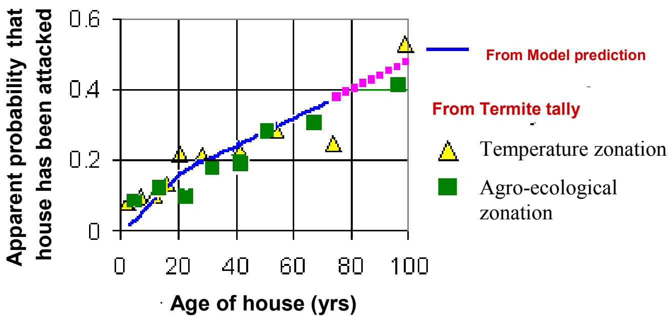


Figure 6. A comparison between the predicted and measured occurrence of termite attack for the average house in Australia.

HAZARD ZONES

One of the first steps in undertaking a risk analysis for biological attack is to establish hazard zones related to the location of the construction. Meteorological information and data on sea water temperatures are used to assess the hazards. Figures 7–10 show the derived hazard zones for the various biological attacks under consideration.

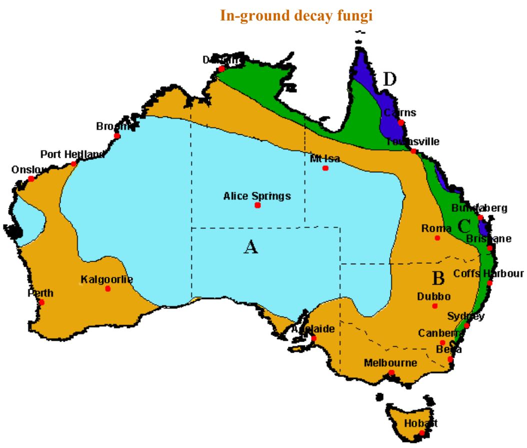


Figure 7. Hazard zones for in-ground decay (D is the most hazardous zone).

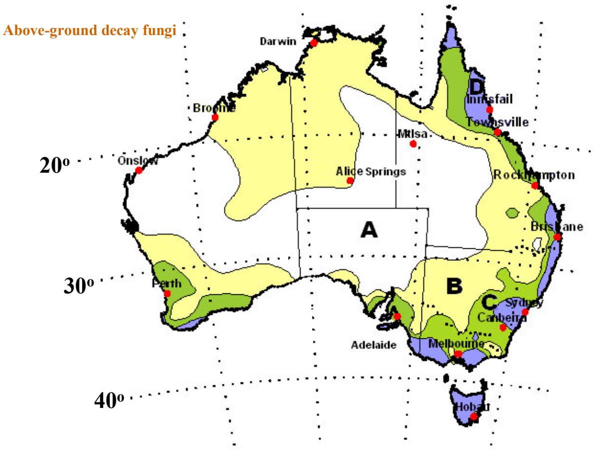


Figure 8. Hazard zones for above-ground decay (D is the most hazardous zone).

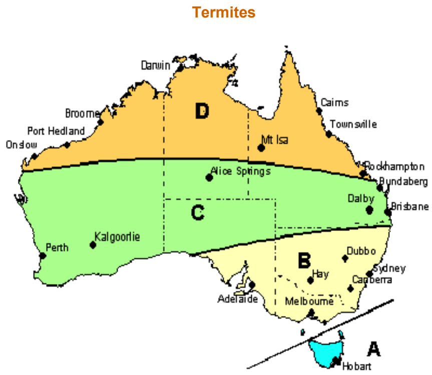


Figure 9. Hazard zones for termite attack (D is the most hazardous zone).

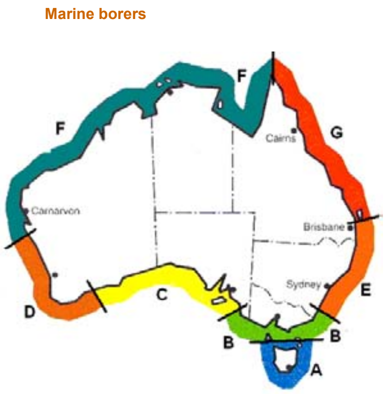


Figure 10. Hazard zones for marine borer attack (F is the most hazardous zone).

THE BUILDING MICROCLIMATE

There is considerable interest in the durability of building envelopes and so methods for predicting the microclimate of buildings are required. Initially, empirical models of microclimate were developed on the basis of measurements made on conventional Australian houses. To do this some 40 occupied Australian houses have been monitored for periods of several years to obtain empirical descriptions of the microclimate within the building envelope and the performance of materials therein. Figure 11 shows the location of the houses monitored and Fig. 12 indicates schematically some of the locations within a house in which microclimate and building performance are measured. In addition, small model buildings, about 5m x 5m square as shown in Fig. 13, are being used to assess the effectiveness of using building physics to predict the complexities of a building microclimate.

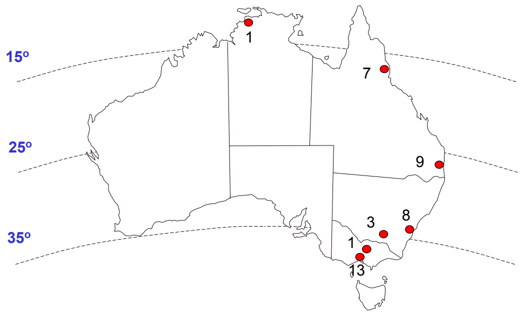


Figure 11. Locations of monitored house (the numbers shown indicate the number of monitored houses to be found at the associated location).

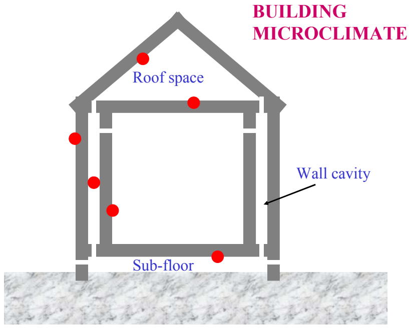


Figure 12. Locations where building performance is measured.

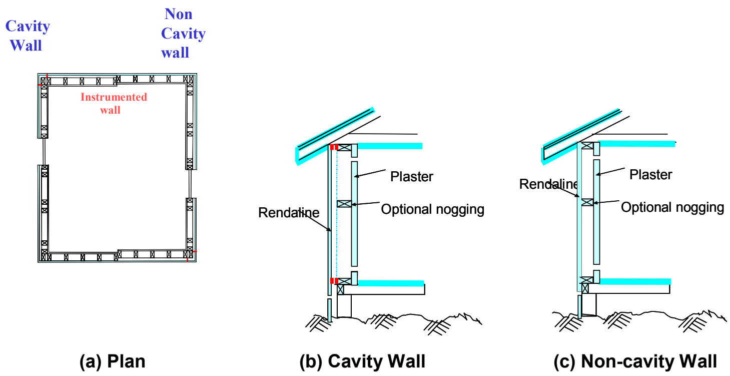


Figure 13. Details of a model building.

CONCLUDING COMMENT

There are an extremely large number of material and environmental factors that affect the pattern and rate of biological attacks. So despite the fact that a large amount of data is available, more than 10,000 field test specimens and 10,000 data points from in-service structures, it was found to be not possible to develop globally applicable models based solely from empirical trends in the data. It is preferable to use the data to calibrate models derived initially from physics and biology concepts.

Finally, it should be noted that in view of the lack of adequate information related to biological attack, risk estimates should take into account not only the natural variability due to biological processes, but also the uncertainty of our quantitative knowledge of the biological process. Fortunately this is quite a simple task to undertake when working within the framework of probability theory.

REFERENCES

1. Leicester, R.H. (2001.). Engineered Durability for Timber Construction. *Progress in Structural Engineering & Materials*, **2**(3), 12 pages.
2. Foliente, G.C., Leicester, R.H., Wang,, C-H., Mackenzie C., and Cole, I.S. (2002.). Durability Design for Wood Construction. *Forest Products Journal*, **52**(1), 10-19.
3. Leicester, R.H., Wang, C–H., Nguyen, M.N., Thornton, J.D., Johnson, G., Gardiner, D., Foliente, G.C. and Mackenzie, C. (2003.). An Engineering Model for the Decay of Timber in Ground Contact. *Proc. 34th IRGWP Annual Meetin*g, Brisbane, Section 2, Paper No. IRG/WP/03, 19–23 May, 21 pages.
4. Leicester, R.H., Wang, C–H., Nguyen, M., Thornton, J.D., Foliente, G.C., Cause, M., Mackenzie, C. and Wang, X. (2004.). Structural Durability of Exposed Timber. *Proc. 8th World Conference on Timber Engineering, WCTE 2004*, Lahti, Finland, June, Vol. II, 571–576.
5. Leicester, R.H., Wang, C–H. and Cookson, L.J. (2003.). A Risk Model for Termite Attack in Australia. *Proc. 34th IRGWP Annual Meetin*g, Brisbane, Section 2, Paper No. IRG/WP/03, 19–23 May.
6. Leicester, R.H., Wang, C–H. and Cookson, L. (2004.). A Probabilistic Model for Termite Attack. *Proc. 8th World Conference on Timber Engineering, WCTE 2004*, Lahti, Finland, June, Vol. I, 201–204.
7. Zabel, R.A. and Morell, J.J. (1992.). *Wood Microbiology–Decay and its Prevention*. Academic Press, New York, 476 pages.
8. Australasian Wood Preservation Committee (1977.). Protocols for the Assessment of Wood Preservatives. CSIRO Division of Forestry and Forest Products, Melbourne, Australia, 24 pages.
9. Standards Australia (2003.). AS 5604: Australian Standard– Timber – Natural Durability Ratings. Sydney, Australia, 25 pages.

Prediction of reinforced concrete service life through the measurement of electrical resistivity of specimens subjected to natural exposure conditions.

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ABSTRACT

Reinforcement corrosion is attracting research interest in many areas due to the economical consequences of the damage generated by the process.

Several proposals can be found on prediction of the time to reinforcement corrosion and service life duration. In present communication a proposals is made on using the electrical resistivity to calculate both the initiation and propagation periods. For the time to corrosion onset, the electrical resistivity represents the porosity and its connectivity and therefore can be used to model transport processes. Due to the reaction of chlorides and carbon dioxide with cement phases, the resistivity has to be factorised by a "reaction factor" accounting for it. Concerning the propagation period, the electrical resistivity is an indication of the moisture content of concrete and therefore, it has a certain relationship with the corrosion cement. The service life can be expressed with the following equation:

$$t_l = t_i + t_p = \frac{x^2 \rho_{es} r_{ClCO_2}}{k_{Cl,CO_2}} + \frac{P_x \cdot \rho_{ef}}{k_{corr}}$$

Based in it, minimum resistivity values can be established according to cover thickness and in function of exposure classes.

KEYWORDS: service life, electrical resistivity, reinforced concrete, natural exposure conditions.

1 INTRODUCTION

Regarding concrete durability Codes and Standards in general contain provisions related to: a) the concrete materials: cement, water, steel and aggregate types, concrete mix proportions, mechanical strength, b) the limit of dangerous substances, such as chlorides or sulphates, c) limitations to the crack width transversal to the reinforcement and d) the recommended cover thicknesses in function of exposure classes. However, there is an increasing demand to incorporate into the current standards more advanced concepts related to concrete durability, due the need to better foresee and prevent distresses, in particular the corrosion of the reinforcement.

Several proposals exist based in modelling the mechanisms of attack [Page *et al*. 1981] [Tuutti 1982] or in the so called "performance" concepts [Baroghel-Bouny 2002] or the use of "durability indicators" [Andrade & Alonso 1997] [Whiting 1981]. Nevertheless, their effective incorporation into the standards seem to be slow and a worldwide controversy exists on which is the best approach, due to the lack of enough tradition and experience of these new proposals.

In present paper a proposal is presented that tries to be comprehensive by responding to the demand related to the introduction of performance parameters or durability indicators and being applicable for modelling for predicting service life. The chosen durability parameter is the electrical resistivity of concrete. Its basis and development of the model is presented briefly applied to the corrosion of reinforcements and some examples are shown on the application of the model to several concrete mixes.

2 REINFORCEMENT SERVICE LIFE MODELLING

The service life of reinforcements, t_l, is usually modelled by assuming two periods: the time to initiation of corrosion t_i and its propagation, t_p. Thus, $t_l = t_i + t_p$. The calculation of the duration of t_l is usually undertaken by considering that the aggressive penetrates through concrete cover by diffusion and therefore, Ficks law's is used to calculate a Diffusion coefficient able to predict the concentration of the aggressive at a certain depth, at several periods of time.

Providing that the aggressive threshold (pH-drop front in the case of carbonation or a certain chloride amount) is defined, the end of t_i indicates the initiation of t_p. The initiation of corrosion established a limit state. The propagation period, t_p, is calculated by assuming a constant or averaged corrosion rate. In a similar manner than for t_i the "limit state" or maximum corrosion has to be defined first in order to account for the length of the period.

3 MEANING OF CONCRETE RESISTIVITY

The model proposed here based in the measuring of electrical resistivity makes use of this parameter as the main one determining both t_i and t_p periods [Andrade *et al.* 2000]. This is due the comprehensive character of the resistivity regarding concrete microstructure. Thus, the electrical resistivity of water saturated concrete is an indirect measurement of the concrete pore connectivity. The potential difference or the current applied by means of two electrodes is carried through concrete pore network by the electrical carriers (ions). As higher is the porosity, lower is the resistivity due the higher volumetric fraction of pores. On the other hand, while resistivity is related to porosity and connectivity, in non-water saturated concrete it is as well an indication of its degree of saturation.

Regarding the influence of the chemical composition of pore solution, its impact in the total resistivity is small providing the concrete remains alkaline. At high pH values the pore solution resistivity varies from 30-100 Ωcm, which is comparatively very small taking into account that the concrete resistivity after several days of hardening is in the range of several hundreds Ω·cm.

When concrete carbonates, then the pore solution is much more diluted and the electrical resistivity of the pore solution may significantly increase and start to be influencing. In chloride contaminated concrete, the chlorides lowers the concrete resistivity but not too much, as the resistivity of an alkaline solution is not lowered very much by the presence of chlorides.

In summary, the electrical resistivity provides indications on the pore connectivity and therefore, on the concrete resistance to penetration of liquid or gas substances, and so resistivity is a parameter which accounts for the main key properties related to reinforcement durability.

4 CALCULATION OF THE INITIATION PERIOD

In chloride environment resistivity is directly related to ionic diffusivity in saturated concrete through Einstein law:

$$D_e = \frac{k_{Cl}}{\rho_{es}} = k_{Cl}\ \sigma \qquad [1]$$

where D_e= effective diffusion coefficient, k_{Cl} is a factor, which depends on the external ionic concentration, ρ_{es} is the resistivity (in this case of concrete saturated of water) and σ the conductivity (inverse of resistivity).

In consequence, if k_{Cl} is established, the diffusion coefficient of the chloride ion can be calculated providing that not reaction of chlorides exist with the cement phases, because the D_e so obtained does not account for binding (that is why usually is named as "effective"). However, the chloride binding has to be taken into account. This is made in the proposed model by means of introducing a new factor, (r_{Cl}= reaction or binding factor). This reaction factor is a "retarder" of the penetration of chlorides. Equation [1] can be now written:

$$D_{Cl} = \frac{k_{Cl}}{\rho_{es} \cdot r_{Cl}} \qquad [2]$$

An "apparent" electrical resisvity, ρ_{as}, in saturated conditions can be then defined as $\rho_{as=}\ \rho_{es}\cdot r$.

In the case of carbonation, it the carbonation progresses when the concrete is partially dry. That is, as higher is the porosity higher will be the carbonation depth. As said, porosity is appraised by measuring the electrical resistivity. Therefore, equation [1] can be also applied to carbonation providing another constant k_{CO2} is considered for the atmospheric exposure. In addition, in a similar manner than for chlorides a reaction factor, r_{CO2}, taking into account the amount of alkaline material able to bind CO_2, has to be introduced:

$$D_{CO2} = \frac{k_{CO2}}{\rho_{es} \cdot r_{CO2}} \qquad [2']$$

Thus, expressions of the diffusion coefficients of chlorides and carbon dioxide in terms of the apparent electrical resistivity, ρ_{as}, in saturated concrete have been deduced.

4.1 Calculation of Reaction factor

A key aspect of the proposed method is related to the so called "reaction factor" that accounts for the amount of penetrating substance not really moving inwards, but being immobilized by the cement phases. It represents the proportion of CO_2 reacting with the alkaline compounds of hydrated cement or the proportion of chlorides being bound.

In the form it is proposed to be taken into account in the expression: $\rho_{as=}\ \rho_{es}\cdot r$. It means the number of times that this reaction makes apparently higher the effective resistivity ρ_{es}. Therefore, r can be formulated as a retarder factor varying from 1 to 5 or 10 times. It can be calculated from specific experiments, for instance, comparing the diffusion coefficients in steady and in non-steady state conditions.

The r factor might be as well derived from experiments in mortar, whose translation into values for concrete can be made taken into account the differences in w/c ration and in cement content through the equation:

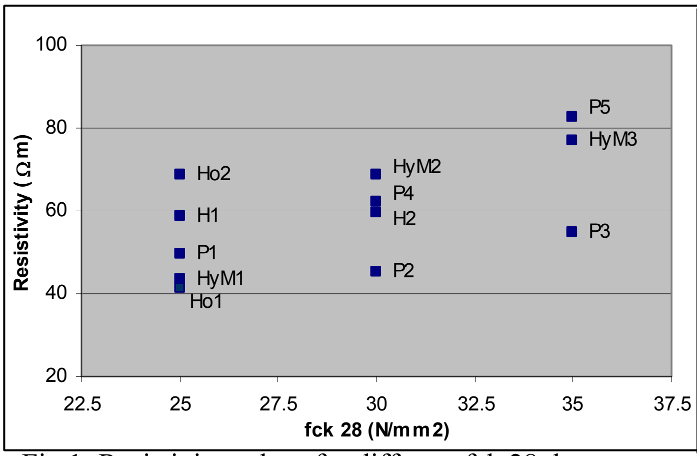
$$r_c = r_m \frac{(a/c)_m}{(a/c)_c} \cdot \frac{c_c}{c_m} \qquad [3]$$

where r_c= reaction factor in concrete, r_m= reaction factor in mortar, $(a/c)_m$= in mortar, $(a/c)_c$= in concrete, c_c= cement content in volume in concrete, c_m= cement content in volume in mortar.

The characteristic value of r_m might be provided by the cement producers in the same manner they provide the cement grading. In this assumption, the concrete producer does not need to test for r_{ce} and r_{CO2} but simply apply equation [3] for each particular concrete proportioning.

4.2 Influence of age and temperature in the resistivity

For the formulation of the model it is not strictly necessary more than taking into account the value of the resistivity after 28 days (see figure 1) of wet curing to obtain $\rho_{es,\,28d,}$ as the reference parameter like in the case of mechanical strength. However, for the sake of a comprehensive presentation, it is worth to mention that models exist for the accounting of the effects of age and temperature on the resistivity values.



	Types of concrete	Cement
HyM1	HA-25/ B/25/IIa	CEM II/A-V 42,5R.
HyM 2	HA-30/B/25/IIa	CEM II/A-V 42,5R.
HyM 3	HA-35/B/25/IIb	CEM II/A-V 42,5R.
HyM 4	HA-30/B/25/IIIa	I 42,5R/SR+ H.S. (6-8%)
HyM 5	HA-35/B/25/IIIb	I 42,5R/SR + H.S.(6-8%)
P1	HA-25/B/20/IIa	CEM II/A-P 42,5R.
P2	HA-30/B/20/IIa	CEM II/A-P 42,5R.
P3	HA-35/B/20/IIa	CEM II/A-P 42,5R.
P4	HA-30/B/20/IIa	CEM I 42,5R
P5	HA-35/B/20/IIa	CEM I 42,5R
Ho1	HA-25/B/20/IIa	CEM II/A-L 42,5R
Ho2	HA-25/B/20/IIa+Qb	II/A-S 42,5 SR
H1	HA-25/B/20/IIa	CEM-II/A-V 42,5R
H2	HA-30/B/20/IIa	CEM-II/A-V 42,5R

Fig.1: Resistivity values for different fck 28 days concrete samples curing in chamber.

It is known that the resistivity of concrete increases with time due the refinement of the pore structure. This evolution is very similar to that of the increase of mechanical strength. The decrease in porosity with the advance of hydration leads to a lowering in porosity which is reflected in both mechanical strength and resistivity. The time law that can be applied for this evolution of ρ_{cs} (see figure 2) with age is of the type:

$$\rho_{es,28d} = \rho_{es,t} \cdot e^{m} \qquad [4]$$

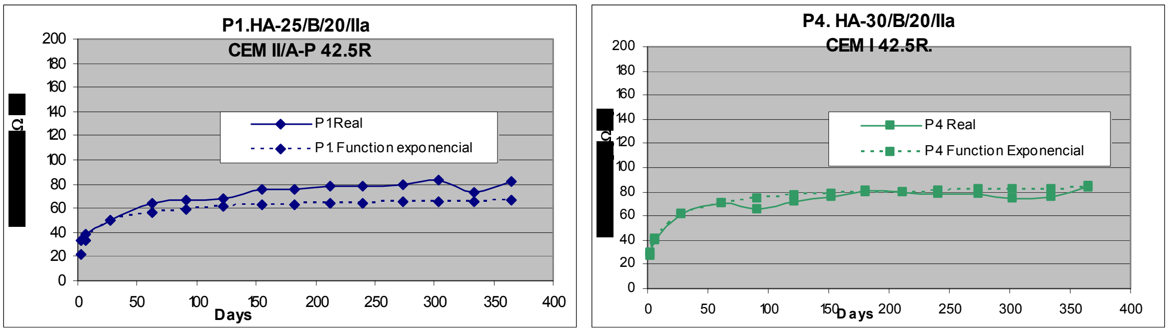


Fig. 2: Evolution of ρ measured in some of the samples and calculated according to [4].

This law may have different power exponents for OPC than for blended cements. There are values in the literature [Monfore 1968] [Hammond & Robson 1955] and work is being done for its quantification.

With respect to the influence of temperature, it has an important effect on resistivity, which only can be generalized if the ρ values are standarized to a reference temperature that it is proposed to be 25ºC [Castellote *et al.*]. An increase in temperature should increase diffusivity, D, and corrosion rate, V_{corr}, however this increase in temperature may at the same time may produce an evaporation, which in turn would effect on the opposite in both, D and V_{corr}. Therefore, the incorporation of temperature effects on models is, by large, still very seldom. In present case, it is incorporated by suggesting to measure $\rho_{es,\ 28d}$ at 25ºC. The standardization of resistivity from other temperatures has been explained in [Castellote *et al.*]

5 CALCULATION OF THE PROPAGATION PERIOD

When the reinforcements start to corrode, it has been found a relation between corrosion rate and electrical resistivity of the type [Alonso *et al.* 1988]:

$$I_{corr} = \frac{k_{corr}}{\rho_{ef}} \qquad [5]$$

where k_{corr} is a constant with a value of $3x10^4 \mu A/cm^2 \cdot k\Omega \cdot cm$

The resistivity, ρ_{ef}, in this case is that of the concrete at its actual degree of saturation and therefore can be that of water saturated conditions or not. In order to calculate the t_p, it can be assumed a certain year averaged concrete moisture content in each exposure class and in function of it, averaged I_{corr} and ρ values can be attributed to each one (considering both moisture and temperature).

6 SERVICE LIFE MODEL BASED ON CONCRETE RESISTIVITY

Assuming a square root relation between penetration of the aggressive front and time $x_i = V_{CO2,\ Cl} \sqrt{t}$, the factor of relation V represents the ease or velocity of penetration, $V_{Cl,\ CO2}$ and therefore the service life can be written in the form:

$$t_l = \frac{x_i^2}{V_{CO_2,Cl}} + \frac{P_x}{V_{corr}} \qquad [6]$$

being P_x the limit corrosion attack depth (loss in rebar diameter or pit depth) and V_{corr}, the corrosion rate.

Based in previous background considerations, the model can be expressed now in the following manner:

1) Initiation period – Assuming the mentioned square root relation between diffusivity and penetration depth of the aggressive, Einstein relation enables to write:

$$x_i = k_{Cl,CO_2} \sqrt{2D_a\, t} \qquad [7]$$

where $k_{Cl,CO2}$ = factor of aggressive penetration, D_a= apparent or non steady state diffusion coefficient taking into account binding and x_i= penetration depth of the aggressive

Substituting in [2] and [2´] in [6] results:

$$t_i = \frac{x_i^{\,2} \cdot \rho_{es} \cdot r_{Cl,CO2}}{2 \cdot k_{Cl,CO2}} \qquad [8]$$

2) Propagation period – substituting [5] in [6] results.

$$t_p = \frac{P_x \cdot \rho_{ef}}{k_{corr}} \qquad [9]$$

Finally the addition of t_i+t_p gives the total service life of reinforcement

$$t_l = t_i + t_p = \frac{x^2 \rho_{es} r_{Cl,CO_2}}{k_{Cl,CO_2}} + \frac{P_x \cdot \rho_{ef}}{k_{corr}} \qquad [10]$$

The main parameter in it is the concrete resistivity measured in saturated conditions, and at 25ºC ρ_{es}, at 28 days of life, and ρ_{ef} given as year averaged value for certain exposure conditions.

This equation considers both initiation and propagation periods which, means that the limit state assumed is until reaching a certain degree of corrosion in the reinforcement. However, it can be as well used by considering the depassivation as the limit state, in which case only the first term of the equation serves.

$$t_l = \frac{x^2 \cdot \rho_{es} \cdot r_{Cl,CO_2}}{k_{Cl,CO_2}} \qquad [10']$$

With regard to the other parameters r and k, **r** should be linked to the type and amount of cement and therefore, to the extension of reaction of the penetrating substance with cement phases, and **k** values are factors which should account for the environmental concentration of aggressive (exposure class).

7 PRACTICAL APPLICATION TO SEVERAL CONCRETE MIXES

Equation 10 enables of calculation of the characteristic resistivity in function of the target service life and the cover thickness. An example is given in table 1 for chloride attack and carbonation using a k_{Cl} = 20000 and k_{co2}= 2000 $\Omega\cdot cm^3$/year, and assuming a limit state of depassivation (equation [10'] in 50 years)

Apparent Resistivity (Ω·m) in saturated conditions at 28 days of curing		
Cover (mm)	*Carbonation*	*Chlorides*
20	250	2500
30	120	1110
40	63	625
80	15	160

Taken these values as "characteristics" they mean that the concrete resistivity in water cured during 28 days concrete has to be higher in 95% of the results than the values of the table, providing ρ_{as} is calculated from the direct measurement of the ρ_{es} in the specimen and multiplied by the reaction factor, $\rho_a = \rho_{es}\cdot r_{Cl,CO2}$.

The formulation of the method as a standard needs the following steps:

1) The classification of exposure aggressivity (environmental actions) to which to refer the characteristic ρ_{as}, the cover thickness and the rest of parameters involved in the method.
2) The establishment of k_{Cl} and k_{CO2} for each exposure class, as well as the averaged V_{corr}.
3) The establishment of the r_{Cl} and r_{CO2} for the particular cement and concrete (by testing by the cement or concrete manufacturer.
4) The measurement of ρ_{es} at 28days in the same concrete specimens used for mechanical strength.
5) The calculation of $\rho_{as}=\rho_{es}\cdot r_{Cl,CO2}$
6) The comparison of the ρ_{as} obtained with the table of characteristic values such as Table 1 or the calculation of expected service life the through equation 10 or 10´.

The use of resistivity as the key parameter to model durability of reinforcements can be made: a) by establishing certain characteristic values to be achieved in standarized conditions as performance requirement or durability indicator or b) by calculating concrete cover thicknesses according to exposure aggressivity through certain equations as a manner of a model. Being a non destructive measurement it results optimum for routing on-site quality control.

8 FINAL COMMENTS AND CONCLUSIONS

The need to provide concrete with a target durability has stimulated the development of models to predict service life. These models are based in parameters figuring concrete permeation on resistance to penetration. In general, diffusion coefficient derived from analytical solutions to differential equations as the reproduction of real boundary and initial conditions of the differential equations is not obvious or easy.

The method here presented is based in a general fundamental of Einstein law relating electrical resistance or conductance with the diffusion coefficient. Stating certain assumptions this basic law can be applied to the advance of carbonation front or chloride threshold, and to the representation of steel corrosion progression. The general expression of service life is:

$$t_l = t_i + t_p = \frac{x^2 \cdot \rho_{es} \cdot r_{Cl,CO_2}}{k_{Cl,CO_2}} + \frac{P_x \rho_{ef}}{k_{corr}}$$

This model can be used for calculating cover thicknesses from actual resistivity values or the minimum resistivity for a certain cover thickness. Resistivity can be as well used as a performance parameter to be fulfilled by standard specimens at a certain age or as durability and corrosion indicator.

Being the measurement of resistivity a non destructive method, it can be as well used for on-site quality control.

9 REFERENCES

Alonso, C., Andrade, C. & González, J.A. 1988, 'Relation between concrete resistivity and corrosion rate of the reinforcements in carbonated mortar made with several cement types', Cement and Concrete Res. Vol. 18, pp. 687-698.

Andrade, C. & Alonso, C. 1997, "A simplified application to carbonation and chloride penetration rates of the performance concept". International ECSN Symposium on "European Concrete Standars in Practice". Copenhaguen (Denmark)

Andrade, C., Alonso, C., Arteaga, A. & Tanner, P. 2000, "Methodology based on the electrical resistivity for the calculation of reinforcement service life" 5th Canmet/ACI Int. Conference on Durability of concrete – Supplementary papers volume. Barcelona, Spain. pp.899-915.

Baroghel-Bouny, V. 2002, "Which toolkit for durability evaluation as regards chloride ingress into concrete? Part II: Development of a performance approach based on durability indicators and monitoring parameters". Proceedings of the 3rd International Workshop "Testing and modelling chloride ingress into concrete" Madrid Spain. C. Andrade and J. Kropp editors.

Castellote, M., Andrade C. & Alonso, C. "Standarization, to a reference of 25 C, of the electrical resistivity values measured at any temperature for mortars and concrete in saturated conditions". ACI Materials Journal.

Hammond, E. & Robson, T.D. 1955, "Comparison of electrical properties of various cements and concretes." The Engineer, pp. 78-115

Monfore, G.E. 1968, "The electricak resistivity of concrete". Journal of PCA. pp, 35-48.

Page, C.L., Short, N.R. & El Tarras, A. 1981, "Diffusion of Chloride lons In Hardened Cement Pastes" Cement and Concrete Research. Vol. 1 1, No. 3, pp. 395-406.

Tuutti, K. 1982, 'Corrosion of Steel in Concrete', Swedish Cement and Concrete Research Institute (CBI) no.4-82, Stockholm.

Whiting, D. 1981, "Rapid determination of the chloride permeability of concrete" – Federal Highway Administration – Report FHWA/RD-81/119.

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Factors Affecting the Service Life of Seams of EPDM Roof Membranes

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ABSTRACT

A quarter century ago, a revolution occurred in the U.S. low-sloped roofing industry with the introduction of a number of new membrane products including elastomeric and thermoplastic polymeric membranes and polymer-modified bituminous membranes. Today, these membranes account for over two thirds of the total low-sloped roofing market in the United States. One single-ply membrane, EPDM (ethylene-propylene-diene terpolymer), gained wide acceptance in that it captured about one-third of the total low-sloped roofing market by the mid-1980s. Also by the mid-1980s, however, surveys conducted by the U.S. National Roofing Contractors Association (NRCA) began to indicate that the EPDM seams were failing leading to roof leaks. These seams were fabricated in the field using liquid-applied polymer-based adhesives. Due to the economic importance of EPDM roofing, the high costs associated with leaking roofs and, most importantly, the urging of the contractor segment of the roofing industry, the National Institute of Standards and Technology (NIST) initiated research to elucidate the factors affecting the service life performance of EPDM seams. The majority of this research was conducted under the auspices of an industry-government consortium. This paper presents a short overview of the NIST experimental findings. At the beginning of the research and in collaboration with industry, NIST attempted to identify the most likely application, design, material, and environmental variables affecting seam service life performance. From this list of over two-dozen variables, the contribution that each variable made acting alone or in combination was assessed through statistically designed creep-rupture experiments. Outcomes of these experiments included the identification of the most important variables affecting seam performance and the derivation of a mathematical model for linking field and laboratory seam joint responses. The results of these experiments lead to recommendations for improving the service life performance of seam joints. These recommendations were adopted by industry and contributed significantly to the improved performance of EPDM seams. Today, EPDM seams fabricated with tape adhesives and the service life performance of these tape-bonded seams are routinely acknowledged by industry as being quite satisfactory.

Keywords

Building technology; creep-rupture testing; EPDM roofing; seam adhesion; service life prediction

1 INTRODUCTION

Twenty-five years ago, a revolution occurred in the U.S. roofing industry with the introduction of elastomeric and thermoplastic synthetic single-ply membranes and also polymer-modified bituminous membranes. Of these new products, one single-ply membrane captured over 50 % of this market, and approximately one-third of the total U.S. low-sloped roof market. It still maintains this market share today. This membrane was EPDM (ethylene-propylene-diene terpolymer), and is the subject of this paper.

Like most new material revolutions, the growth of the new roofing membranes was so fast that many products were introduced into the marketplace prior to gaining any substantial service life performance histories. Not unexpectedly, problems in the service life performance of these products were often observed shortly after they were introduced. The percentage of new membrane use and the number and frequency of associated problems with each membrane type were tracked by Project Pinpoint, which is a U.S. National Roofing Contractors Association (NRCA) lead roof survey program launched in 1974 to provide (1) an early-warning procedure for the identification of problems experienced with in-place low-sloped roofing materials, and (2) baseline information on the membrane materials used and how often they were installed [NRCA 1975, Cullen 1989]. Multiple Project Pinpoint reports were published from 1975 to 1992. From these reports, the use of the elastomeric, thermoplastic, and polymer-modified bitumen membrane materials was almost non-existent prior to the mid-1970s, but by the end of the 1980s, they accounted for about 70 % of the low-sloped roofs installed in the United States—a figure that has remained reasonably constant to the present [Cullen 1993]. Another important finding was that the number and frequency of problems with the service life performance of single-ply membranes also increased from their inception in the mid-1970s.

EPDM is a chemically inert rubber, which makes it attractive for outdoor use as a membrane material, because the membrane itself has excellent weathering properties. However, chemical inertness can be a disadvantage when bonding adjacent sheets in the field to form the seams of a waterproofing membrane. As early as the mid-1980s, Project Pinpoint surveys were indicating numerous problems with the seams which, at that time, were formed using liquid polymer-based adhesives [NRCA 1984, Cullen 1989]. Leaking roofs are of national importance due to high costs associated with degradation of the roofing materials and to consequential damages to the building interior and its contents. With this recognition and, most importantly, with the urging of NRCA, NIST initiated research in the mid-1980s to elucidate the factors affecting seam performance and to recommend solutions for improved performance. This paper summarizes some of the main findings and recommendations from those studies. The majority of NIST research activities were conducted collaboratively with industry.

2 PERFORMANCE OF LIQUID-ADHESIVE-BONDED SEAMS

The NIST seam studies were initiated in the mid-1980s using specimens prepared with liquid-applied polychloroprene- and butyl-based adhesives, the primary adhesives available for making EPDM seams at the time. In developing background information for the studies, NIST research staff reviewed the literature and held numerous discussions with individuals knowledgeable regarding the performance of EPDM roofing including manufacturer representatives, contractors, consultants, and adhesive technologists. Based on the information obtained, numerous possible application, design, material, and environmental 'faults' were identified that, acting alone or in combination, were felt to lead to the failure of EPDM seams (Fig. 1). The selection of these factors was largely based on field experience. Specifically, over 85 % of the reported roofing problems were indicated as developing within the first three years after installation of the roof with the majority of these early failures, ≈ 60 %, occurring within

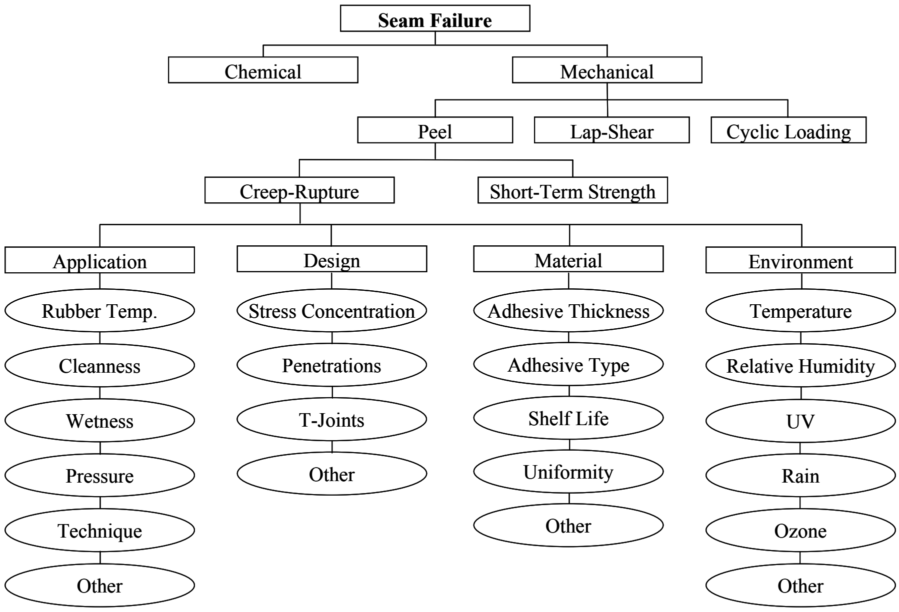

Figure 1. Factors Affecting the Durability of Seams of EPDM Roofing Membranes

the first year after installation [NRCA 1984]. Additionally, the longer the roofing seams appeared to survive without problems, the longer the carefree maintainance of the roof. From these observations, it was concluded that the failure of many EPDM seams was probably not due to the chemical degradation of the seam joint, because the longer a seam joint remained defect free, then the longer its service life. NIST field inspections of EPDM roofing also supported feedback obtained from the roofing community that seam failures were often associated with EPDM roofs that exhibited ripples resulting from the contractors inability to lay the membranes down flush with the roof (Fig. 2). NIST researchers reasoned that such ripples would subject the seams to small peel loads which over time could lead to a creep-rupture failure. Based on this a priori assumption, they developed creep-rupture test protocols suitable to apply a creep-rupture peel stress to seam specimens and observed their times-to-failure [Martin et al. 1990]. The better performing seams had longer times-to-failure. The factors investigated included material parameters such as the adhesive type and its applied thickness, mechanical parameters such as the magnitude and type of load (i.e., peel and shear), environmental parameters such as temperature, moisture and ozone, and application parameters such as the cleanness of the EPDM rubber surface. Consistent with field observations, during these initial studies, it was found that seams were far more resistant to sustaining shear creep-rupture loads than they were to sustaining peel creep-rupture loads. Consequently, the subsequent investigations focused on understanding the peel creep resistance effects of small changes in application, material, and environmental factors.

Martin et al. [1989, 1990] described initial NIST creep-rupture experiments and major findings in a study undertaken to examine the importance and ranking of the following material and application variables on creep resistance and short-term strength: adhesive thickness, cure time, mechanical load, adhesive type, and surface cleanness. From these studies, it was concluded that the greatest effects on creep resistance were due to adhesive thickness, the magnitude of the mechanical load, and the cleanness of the EPDM surface. Figure 3 illustrates the effect of adhesive thickness, wherein it is evident that creep resistance varied exponentially with thickness. This was the first reported instance of the effect of adhesive thickness

Figure 2. Ripples in an EPDM Membrane and Seam

on creep resistance of EPDM seam specimens. Its benefit could not be underestimated since, in practice, some contractors admitted to NIST research staff that they did not always apply adhesives at recommended quantities, if for no other reason than to preserve adhesive, or for the more practical reason of lessening the time necessary for application. Consistent with such admissions, NIST observations from field inspections showed, for example, that the adhesive thickness was often less than EPDM manufacturers' recommendations [Rossiter et al. 1991].

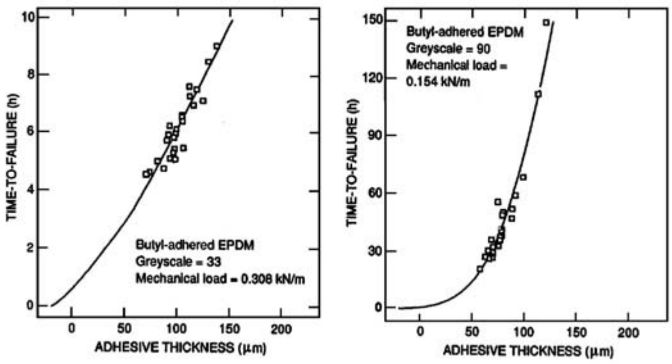

Figure 3. Time-to-Failure versus Adhesive Thickness [from Martin et al. 1990]

Martin et al.'s [1990] adhesive thickness measurements demonstrated the folly in thinking that there was no adverse effect in applying thin adhesive layers. On the positive side, the findings on peel load and surface cleanness reinforced practices that, in service, peel loads should be kept as low as possible, if not avoided, and that EPDM surfaces should be cleaned as well as possible before adhesive application.

Having demonstrated the effects of adhesive thickness and surface cleanness, subsequent experiments were designed to further address the importance of these parameters [Rossiter et al. 1994]. Thus, a creep experiment was conducted using EPDM rubber and butyl-based liquid adhesive. Three variables were included: adhesive thickness (thin and thick), EPDM surface cleanness (well cleaned and particulate contaminated), and type of load (peel and shear). Only a few of the shear specimens failed and the failures were for specimens having thin adhesive and contaminated EPDM. In contrast, the majority of the peel specimens failed. From Fig. 4, the ranking, in decreasing order of creep resistance of the peel specimens was: thick, clean >> thick, contaminated > thin, clean > thin, contaminated where the symbol > signifies "better than" while >> signifies "very much better than." The effect of adhesive thickness was greater than the effect of surface cleanness. Observe also in Fig. 4 that the thick, clean specimens displayed significantly longer mininum times-to-failure, by a factor of at least 100, than did those fabricated using the other three combinations of fabrication conditions. It is noted that, in this and the other NIST experiments described herein, whenever the EPDM surface was well cleaned, the creep-rupture failure mode was cohesive within the adhesive layer.

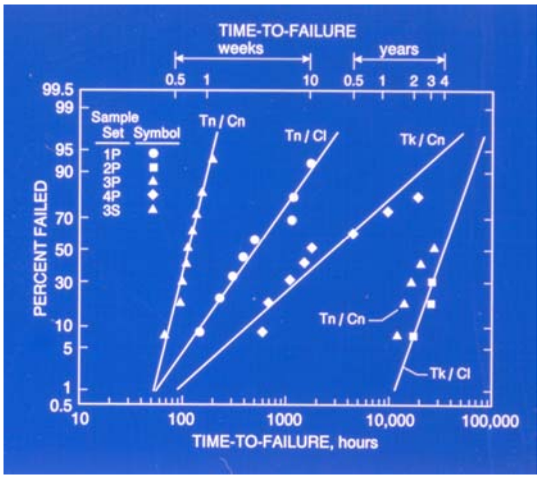

Figure 4. Effect of Adhesive Thickness, Surface Cleanness, and Type of Load on Creep-Rupture Times-to-Failure [from Rossiter et al. 1994]

Overall, the NIST research emphasized that, in practice, significant benefits in service life were accrued when the EPDM surfaces were cleaned and a thick adhesive was applied. The NIST results for adhesive thickness had not been foreseen by anyone with whom discussions were held when the studies on liquid-adhesive seams were initiated and considerable efforts were needed to alter the mindset of many EPDM practitioners. Fortunately, although the relationship between adhesive thickness and seam performance was surprising to many, its implications were taken seriously. For example, in efforts to reach contractors, the NRCA published an article entitled "Is Your Adhesive Layer Thick Enough?" in its monthly trade magazine [Martin et al. 1991].

Finally, it is noted that the importance of adhesive thickness on seam performance was only demonstrated through creep-rupture experiments. Measurements of short-term peel strength showed only a slight effect due to thickness. Consequently, in 1993, ASTM published, based on NIST research, Standard Test Method D5405, "Conducting Time-to-Failure (Creep-Rupture) Tests of Joints Fabricated from Non-bituminous Organic Roof Membrane," so that the creep-rupture procedure would be available to all having interest in elucidating factors affecting seam performance.

3 PERFORMANCE OF TAPE-BONDED SEAMS

In the early 1990s, as NIST was completing its studies on liquid adhesives, the EPDM roofing manufacturers introduced a new generation of adhesives based on preformed butyl tapes. Many roofing contractors and other practitioners, due to inherent problems with accepting a new product without any tangible field performance data, received the introduction of tape adhesives with little enthusiasm. On the other hand, proponents believed that tape adhesives had advantages over liquid adhesives such as enhanced seam performance, product uniformity, installation robustness, lessened environmental impact because they were solvent-free, and lower seam fabrication costs. In 1994, the EPDM industry requested that NIST establish an industry-government consortium to gain an understanding of the service life performance of tape-bonded EPDM seams [Rossiter et al. 1995]. The original consortium was comprised of three EPDM membrane material manufacturers, two tape adhesive manufacturers, two roofing industry associations, and the U.S. Army Construction Engineering Research Laboratories (CERL). The objectives of the consortium were to (1) compare the creep-rupture performance of tape-bonded and liquid-adhesive-bonded seams of EPDM membranes, and (2) recommend a test protocol for evaluating creep-rupture performance of such seams. The consortium experimental program was designed in three phases. The major findings are summarized in the following paragraphs. For complete details on the experimental procedures and methods of analyses, the reader is referred to the original references.

3.1 Phase I — Effect of Load on Peel Creep

In Phase I, the creep-rupture response of tape-bonded and liquid-adhesive-bonded seam specimens was determined as a function of peel load [Rossiter et al. 1996]. Specimens were fabricated using two commercial tape systems (i.e., tape and primer) and a butyl-based liquid adhesive applied to well-cleaned EPDM rubber. The results are summarized in Fig. 5, which is a plot of mean time-to-failure (TTF) as a function of load. The plot characters represent the mean data points; whereas the curves represent the fit of the data to the model:

$$\ln(TTF) = b_0 + b_1 \mathrm{A}\,\mathrm{Load} + b_2 \mathrm{A} \exp(b_3 \mathrm{A}\,\mathrm{Load}).$$

In all cases, the mean time-to-failure data points fall on or are close to the fitted curves. It is evident in Fig. 5 that the relationship between time-to-failure and load is relatively linear at the higher loads and nonlinear at lower loads. Note that no data points are included in this figure for tests conducted below 5 N, because no specimen failures were observed after more than 16 800 h (about 23 months) of testing at the lowest load of ≈ 3 N. The major conclusion was that the tape-bonded specimens had times-to-failure that

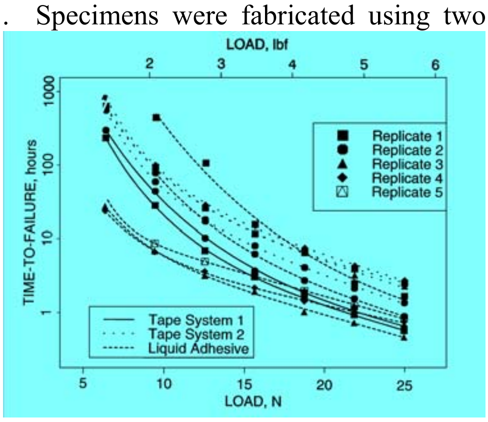

Figure 5. Time-to-Failure Versus Load for Tape-Bonded and Liquid-Adhesive-Bonded Seams [from Rossiter et al. 1996]

were in most cases comparable to, or greater than, those of the liquid-adhesive-bonded specimens. In addition, the tape-bonded specimens provided time-to-failure results that were reproducible between replicate sets. This reproducibility may have been associated with the tapes being factory-made (i.e., pre-formed) products and not subject to some of the non-controllable application variables associated with the liquid-applied adhesives. Observe in Fig. 5 that the liquid-adhesive-bonded sample sets were less reproducible than the two tape systems, particularly at the low loads (e.g., 12.5 N). A consequence of this wide variability was that, under certain conditions, the liquid adhesive provided seam sample sets which displayed substantially longer creep lifetimes than either other liquid-adhesive-bonded sample sets or tape-bonded sample sets. However, the conditions that produced the relatively long-lived liquid-adhesive specimens were not known and not reproducible.

3.2 Phase II — Effect of Material and Application Parameters on Peel Creep

The Phase II research was performed to investigate the effects of material and application factors on the peel creep-rupture response of tape-bonded seam specimens [Rossiter et al. 1997]. The specimens were prepared using the two commercial tape systems from Phase I. Thus, two material factors (tape system and thickness) and five application factors (EPDM surface condition, primer, application temperature, application pressure, and time-at-application-temperature) were examined in a two-level statistically designed experiment. For each parameter, the levels were chosen far enough apart so that the range of practical importance was generally covered. Some tapes had thicknesses typical of those commercially available at the time of the study, and were designated as having ‘standard’ thickness. The thicknesses of 'standard' and thin tapes (which were experimental products for the study) were approximately 0.9 mm and 0.6 mm, respectively. Specimens were prepared either primed or unprimed using EPDM that was either cleaned or contaminated. Application temperatures were low, 5 °C, or high, 60 °C, and application pressures were low, 0.2 MPa, or high, 2 MPa. The time at which the specimens remained at the application temperature was short, about 24 h, or long, 672 h to 960 h. A full factorial design was beyond the scope of the Phase II study, so a half-fraction of the full factorial design was chosen. This design included the four combinations of material factors, and 16 of the 32 possible combinations of the application factors. The same 16 combinations of the application factors were assigned to each of four combinations of the material factors. The main conclusions from the Phase II investigations included:

- Primed, clean EPDM provided the longest times-to-failure. This result, although not unexpected, emphasized to contractors that proper application is a critical parameter affecting tape-bonded seam performance.
- Primed, clean EPDM and 'standard' thickness tape afforded times-to-failure that were greater than the minimum mean times-to-failure of well prepared liquid-adhesive-bonded specimens in the Phase I investigations.
- The application temperatures and application pressures used in the investigation did not affect the times-to-failure of sample sets prepared with primed, clean EPDM that had long times-to-failure. This finding was noteworthy, because application temperatures and manual application of pressure during seam fabrication are impossible to control in practice.
- 'Standard' thickness tape provided significantly longer times-to-failure than thinner tape. This finding was comparable to the adhesive thickness effect found for liquid-adhesive seams [Martin et al. 1990, Rossiter et al. 1994], and provided evidence that seam time-to-failure could be compromised if tapes (having comparable chemical formulations) were made commercially available at thicknesses less than those of the ‘standard’ thickness available at the time of the study.

3.3 Phase III — Effect of Exposure and Shear Load on Creep Response

In Phase III, four tasks investigated the effects of: (I) elevated test temperatures while being subjected to a peel creep-rupture load, (II) elevated temperature exposure prior to being subjected to a peel creep-rupture load, (III) exposure to two industry-developed aging protocols, and (IV) preparation of specimens at cold temperatures prior to peel creep-rupture loading. An additional task (V) examined shear testing [Rossiter et al. 1998]. Specimens were prepared as in Phases I and II using the two commercial tape systems (i.e., tape and primer) and one liquid adhesive applied to well-cleaned EPDM rubber. For each task, comparisons of the creep-rupture responses of tape-bonded and liquid-adhesive-bonded samples were made. Main conclusions of this phase of the research included:

- As the temperature and creep-rupture peel load increased, the times-to-failure of the three adhesive systems decreased. For all treatments, the tape-bonded sample sets had longer mean peel times-to-failure than did the liquid-adhesive-bonded sample sets.
- When peel creep-rupture specimens were exposed to elevated temperatures before loading, times-to-failure of liquid-adhesive-bonded samples were either unaffected or became longer versus the times-to-failure of control specimens (i.e., not temperature exposed). In comparison, peel times-to-failure of one tape-bonded system increased, and that of the other tape-bonded system became shorter (versus the controls) or were unaffected. In cases where the tape-bonded specimen times-to-failure became shorter, they were generally comparable to those for the liquid-adhesive bonded specimens.
- Many shear creep-rupture tests, particularly those at room temperature, produced few failures within the allotted test time. At 70 °C and loads of 24.9 kN/m and 28.0 kN/m, both tape systems had shorter shear-creep lifetimes than the liquid adhesive system. The practical significance of these results is questionable, because the thermal expansion coefficient of EPDM is usually much greater than that of other roofing system materials. The EPDM would tend to generate ripples as opposed to being stressed in shear at high temperatures.

3.4 Overall Results of the Tape-Adhesive Studies

Consistently throughout the consortium study on tape-bonded seams, specimen sets fabricated using either one of two tape-adhesive systems and cleaned, primed EPDM had mean peel times-to-failure that were generally comparable to, if not greater than, those of liquid-adhesive-bonded specimen sets. Moreover, although some laboratory exposures resulted in shorter peel times-to-failure for some tape-bonded seams versus the controls, the resultant times-to-failure were yet comparable to values measured for some field-sampled tape-bonded seams that had performed well in service. EPDM practitioners were enthusiastic about the findings and considered that the consortium study played a key role in accelerating the acceptance of tape-bonded seams in the United States. In 1998, the NRCA marked the study conclusion with the acclamation that "laboratory and field studies confirm the viability of tape-bonded seams" [Smith 1998]. Additionally, the second consortium objective on development of a test protocol for evaluating creep-rupture performance of tape-bonded seams was successfully met. Specifically, the results provided the technical basis of ASTM Standard Practice D6383, "Time-to-Failure (Creep-Rupture) of Adhesive Joints Fabricated from EPDM Roof Membrane Material." Tape adhesives are now well accepted by EPDM practitioners to the extent that they are the cornerstone adhesives for EPDM seam fabrication.

4 FINAL COMMENTARY

Today in the United States, EPDM membranes systems are routinely installed on low-sloped roofs. They provide architects, engineers, contractors, building owners, and others a viable choice among the many low-sloped systems available to the U.S. roofing industry. The issue of satisfactory seam performance is no longer topical. Advances in adhesive technology by EPDM manufacturers and proper seam application by roofing contractors have set aside the issue. In the United States, creep-rupture testing has become a well-accepted method for generating service life data on the performance of seams of EPDM membranes. Creep-rupture testing, particularly in peel, should be an integral part of any methodology that evaluates the service life of EPDM seams.

5 ACKNOWLEDGMENTS

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6 REFERENCES

Cullen, W. C. 1993, *Project Pinpoint Analysis: Ten-Year Performance Experience of Commercial Roofing 1983-1992*, National Roofing Contractors Association, Rosemont, IL, 12 pages.

Cullen, W. C. 1989, *Project Pinpoint Analysis: Trends and Problems in Low-Slope Roofing 1983-1988*, National Roofing Contractors Association, Rosemont, IL, 27 pages.

Martin, J. W., Rossiter, W. J., Jr., and Embree, E. 1991, *Is Your Adhesive Layer Thick Enough?* Professional Roofing, National Roofing Contractors Association, Rosemont, IL, May, pp. 30-37.

Martin, J. W., Embree, E., Stutzman, P. E., and Lechner, J. A. 1990, *Strength and Creep-Rupture Properties of Adhesive-Bonded EPDM Joints Stressed in Peel*, Building Science Series 169, National Institute of Standards and Technology, Gaithersburg, MD, May, 59 pages.

Martin, J. W. and Embree, E. 1989, 'Effect of Contaminants and Cure Time on EPDM Single-Ply Joint Strength,' *J. Materials in Civil Engineering*, **1**[3], 151-166.

NRCA 1984, *Survey Compares Practices to Problems*, The Roofing Spec, U. S. National Roofing Contractors Association, Rosemont, IL, October, pp. 37-40.

NRCA 1975, *Project Pinpoint: The First Returns*, The Roofing Spec, U. S. National Roofing Contractors Association, Rosemont, IL, March, pp. 12-13.

Rossiter, W. J., Jr., Vangel, M. G., and Kraft, K. M. 1998, *Performance of Tape-Bonded Seams of EPDM Membranes: Factors Affecting the Creep-Rupture Response of Tape-Bonded and Liquid-Adhesive-Bonded Seams*, Building Science Series 177, National Institute of Standards and Technology, Gaithersburg, MD, July, 55 pages.

Rossiter, W. J., Jr., Vangel, M. G., Kraft, K. M., and Filliben, J. J. 1997, *Performance of Tape-Bonded Seams of EPDM Membranes: Effect of Material and Application Factors on Creep-Rupture Response*, Building Science Series 176, National Institute of Standards and Technology, Gaithersburg, MD, May, 61 pages.

Rossiter, W. J., Jr., Vangel, M. G., Embree, E., Kraft, K. M., and Seiler, J. F., Jr. 1996, *Performance of Tape-Bonded Seams of EPDM Membranes: Comparison of the Creep-Rupture Response of Tape-Bonded and Liquid-Adhesive-Bonded Seams*, Building Science Series 175, National Institute of Standards and Technology, Gaithersburg, MD, May, 73 pages.

Rossiter, W. J., Jr., Lechner J. A., Seiler, J. F., Jr., and Embree, E. 1995, 'Performance of Tape-Bonded Seams of EPDM Membranes: Initial Characterization,' Proc. 11th Conference on Roofing Technology, U.S. National Roofing Contractors Association, Rosemont, IL, September, pp. 78-89.

Rossiter, W. J., Jr., Martin, J. W., Lechner, J. A., Embree, E., and Seiler, J. F., Jr. 1994, 'Effect of Adhesive Thickness and Surface Cleanness on Creep-Rupture Performance of EPDM Peel and Lap-Shear Joints,' *Roofing Research and Standards Development*: 3rd Vol., ASTM STP 1224, eds. T. J. Wallace & W. J. Rossiter, Jr., ASTM International, West Conshohocken, PA, June, pp. 123-138.

Rossiter, W. J., Jr., Seiler, J. F., Jr., Spencer, W. P., Lechner, J. A., and Stutzman, P. E. 1991, 'Characteristics of Adhesive-Bonded Seams Sampled From EPDM Roof Membranes,' Proc. 3rd International Symposium on Roofing Technology, U.S. National Roofing Contractors Association, Rosemont, IL, April, pp. 167-179.

Smith, T. L. 1998, *EPDM Research Results*, Professional Roofing, The National Roofing Contractors Association, Rosemont, IL, August, pp. 20-22.

Risk assessment and remediation costs for asbestos-containing material

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ABSTRACT

Asbestos-containing material (ACM) must be managed in place until it becomes necessary to remove it. Removal is most often done prior to renovation or demolition of a building, but may be appropriate sooner if the risk to workers and others exceeds acceptable limits. The decision to do so can be based in part on quantitative assessments of the Current Condition of the ACM and its Potential for Disturbance.

A protocol for the quantitative assessments has been developed and incorporated in an industry consensus standard. [1] This protocol far surpasses any previously used for assessing ACM in its versatility and value of the information to the user. Current Condition is rated from 1 (poor) to 10 (good) on the basis of observed damage and debris. Potential for Disturbance is rated from 1 (low) to 10 (high) on the basis of physical activity and accessibility, and environmental factors such as water damage and vibration. These ratings are tabulated and plotted on a graph to establish priorities for removal of the ACM.

Software has been developed that

- Displays the Current Condition and Potential for Disturbance assessments ratings and the quantities of ACM on a three-dimensional multi-color decision chart. The chart vividly shows which materials have the highest priority for removal and which can be managed in place.
- Calculates, tabulates and graphically displays the cost of removal, which is a consequence of having made the decision to remediate the risk, in relation to the assessment ratings.
- Applies a probability matrix to the removal costs to calculate, tabulate and graphically display the probable cost of removal for budgeting purposes, factoring in the assessment ratings for each material.
- Calculates, tabulates and graphically displays the life-cycle cost of managing the ACM while it remains in the building, considering the complexity and frequency of anticipated work.

The software, [2] which will be demonstrated during the presentation, can be used as a stand-alone product or incorporated into the user's facility management or risk management software.

KEYWORDS: Asbestos Management, Remediation, Risk Assessment, Cost Estimating, Software

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[1] E2356 Standard Practice for Comprehensive Building Asbestos Surveys, © ASTM International, 100 Barr Harbor Drive, West Conshohocken, PA 19428. www.astm.org

[2] *Customized Compliance Program for Asbestos*, © Environment-i-media, Inc., 900 Route 620 South, Austin, TX 78734. www.environment-i-media.com

INTRODUCTION

The health effects of breathing asbestos fibers are so well known that they need not be documented in this paper. Historically, the affected individuals have mostly included workers in the mines, mills and factories where the fiber and numerous products were produced, as well as those who installed the products in buildings and facilities. More recently, attention has been focused on those who work around asbestos-containing materials (ACM) as part of construction or maintenance activities.

The management of ACM in buildings and facilities consists of an on-going Operations and Maintenance (O&M) program and abatement when necessary, usually by removal. Assessing the risk of exposure to airborne asbestos fibers involves several factors, as does estimating the costs associated with managing the installed materials and their eventual removal.

This paper describes an approach to assessing the risk posed by ACM on the basis of its Current Condition and Potential for Disturbance. A graphical presentation of the assessments for all ACM in a building or facility allows one to prioritize response actions and decide which materials to remove and which to continue managing in place. The *Customized Compliance Program for Asbestos* software applies unit costs for removal and O&M to calculate the cost of the risk-based decisions for abatement and O&M, and to develop life-cycle costs for managing and removing ACM as long as it remains in the building or facility.

ASSESSING ASBESTOS-CONTAINING MATERIALS

The protocol for assessing the Current Condition and Potential for Disturbance of ACM appears in ASTM E2356 Standard Practice for Comprehensive Building Asbestos Surveys. [ASTM 2004] Those who are familiar with inspection and assessment protocols developed for compliance with the U.S. Environmental Protection Agency (AHERA) regulations issued under the Asbestos Hazards Emergency Response Act [EPA, 1987] will find that the methodology in E2356 yields a greater amount of more usable information. The *Customized Compliance Program for Asbestos* software further improves on the E2356 protocol.

Asbestos-containing materials in a building or facility are assessed during the activity described in E2356 as a Baseline Survey. In addition to taking bulk samples of materials that may contain asbestos for analysis in a laboratory, the inspector assesses the Current Condition and Potential for Disturbance of different types of ACM in various locations. The basis for this assessment is explained next.

Based on visual observation by the inspector, which may include touching to determine friability, the *Current Condition* (CC) of each material is categorized as shown below: A rating of "1" represents the low end of "Poor" and "10" represents the high end of "Good," i.e. completely intact material.

Qualitative Ranking	Numerical Ratings	Description of ACM
Poor	1, 2 or 3	Extensive damage and/or visible debris
Fair	4, 5, 6 or 7	Moderate amounts of damage and/or visible debris
Good	8, 9 or 10	Little or no damage or visible debris

Table 1. Assessment factors for Current Condition of asbestos-containing materials

Anticipating what might happen to suspect ACM in the future – its *Potential for Disturbance* (PFD) -- is more complex. A regulatory definition of "Disturbance" is "...activities that disrupt the matrix of ACM or PACM, crumble or pulverize ACM or PACM, or generate visible debris from ACM or PACM."[OSHA 1994] The inspector assesses each material based on one or more of the following criteria.

Qualitative Ranking	Numerical Ratings	Physical disturbance		Environmental disturbance			
Low	1, 2 or 3						
Medium	4, 5, 6 or 7	*accessibility*	*activities*	*vibration*	*air / dust*	*water damage*	*corrosive*
High	8, 9 or 10						

Table 2. Assessment factors for Potential for Disturbance of asbestos-containing materials

Physical disturbance considers the *accessibility* of the material by workers during normal facility operations, including maintenance and repair, and the *activities* performed near the material - what people do and how often they do it. *Environmental disturbance* considers sources of *vibration*, such as operating machinery, HVAC equipment, whether *air currents* are strong enough to dislodge loose ACM or if *airborne dust* can erode the material. *Water* from a leaking roof or pipe may have damaged the material. The material may be subjected to a *corrosive atmosphere or liquids* that can erode the matrix and expose asbestos fibers. **NOTE:** This is not an algorithm! The ratings are not added, multiplied or arithmetically combined in any manner. They are tabulated and plotted as shown below.

ASSESSMENT TABLES AND CHARTS

Table 3 contains survey and assessment data for a small boiler plant. The table has been sorted to place the materials in the worst condition (lowest CC rating) at the top, and if there are two or more materials with the same PFD rating for the same CC rating, a second sort was performed to rank these materials according to the highest PFD rating (most accessible). A glance at the table shows which materials in what locations are the most in need of attention. This is the first step in deciding which ones to remove and which to keep managing in place.

Location	Area	Asbestos-Containing Materials	Quantity	Assessment			
				Current Condition		Potential for Disturbance	
				Rating	Based on	Rating	Based on
G	Boilers #1, #2 & #3	Roofing	500 ft²	1	Damage & debris	7	Frequent access
D	Southwest corner	Tank & fittings insulation	120 ft²	2	Damage & debris	9	Frequent access
C	Southwest corner	Pipe insulation	150 ft	3	Damage & debris	8	Frequent access
E	Boilers #1, #2 & #3	Steam drum insulation	250 ft²	3	Missing covering	7	Elevated location
F	Boiler #4	Steam drum insulation	100 ft²	7	No visible damage	5	Elevated location
B	East aisle and southeast corner	Pipe insulation	440 ft	9	No visible damage	5	Elevated location
A	East aisle and northeast corner	Breeching insulation	1500 ft²	9	No visible damage	4	Elevated location

Table 3. Assessment ratings and quantities of asbestos-containing materials in boiler plant

Two-dimensional charts

E2356 includes a two-dimensional chart called the Abatement vs O&M Decision Chart on which these ratings are plotted. Figure 1 shows such a chart for the boiler plant example in Table 3. The closer to the upper left corner the rating for a particular ACM is plotted, the greater the risk of exposure to asbestos fibers and consequently the higher the priority is for removing the ACM. The area above the curved line is called the Abatement region. Below the line is the O&M region, and the closer to the bottom right corner the rating for a particular ACM is plotted, the lower the risk of exposure and managing it in place is more feasible.

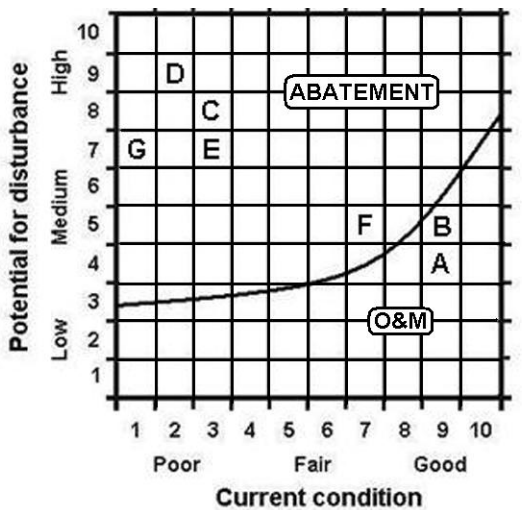

Figure 1. Abatement vs O&M Decision Chart for boiler plant (biased toward abatement)

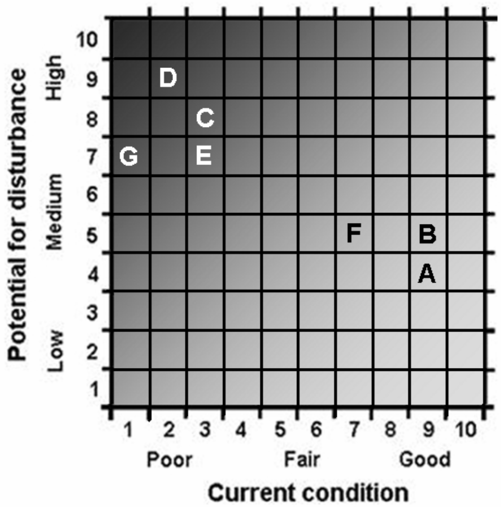

Figure 2. Abatement vs O&M Decision Chart with abatement priorities shown by shading

The position and shape of the line in Figure 1 biases decisions toward abatement, which occupies more area of the chart than O&M. One reason is that O&M tasks for remaining ACM in this boiler plant would be of more-than-average complexity and frequency. Complexity takes into account the difficulty of access to the ACM, the difficulty of preparation such as de-energizing and cooling pressurized lines, disruption of operations, type of PPE required and other factors in addition to the difficulty of the work itself. Frequency is simply to expectation of how often the O&M work will need to be done. In this example, the roofing and steam drum insulation on boilers #1, #2 and #3 are clear candidates for removal, as is the thermal system insulation in the southwest corner. The remaining ACM is close to the line and whether to remove it or leave it in place is a matter deserving consideration of other factors. These might include the proficiency of the O&M crew (in-house or contractor) and whether including these items in an abatement project for other ACM is cost-effective.

Making decisions about removing ACM or managing it in place is not a simple matter of drawing a line on a chart. The benefit of doing so is to encourage an honest evaluation of the overall asbestos management program as part of the decision-making process. The risks associated with ACM should be viewed as a continuum of severity that can be represented by the shading on the chart in Figure 2. The darker area in the upper left corner includes ACM with the higher priority for removal than that in the lighter area toward the lower right. Notice that the line has been removed – it is the relative positions of the ACM that drive the priorities in conjunction with other factors such as complexity and frequency of O&M work.

Three-dimensional charts

Figures 1 and 2 do not differentiate among the types of ACM or the quantities of each – only the identifying notations in Table 3 are shown. The *Customized Compliance Program for Asbestos* software [Environment-i-media, Inc., 2004] displays the Current Condition and Potential for Disturbance ratings on the “floor” of a three-dimensional chart, with the vertical axis representing the amount of each ACM. The three-dimensional chart for the boiler plant is shown in Figure 3.

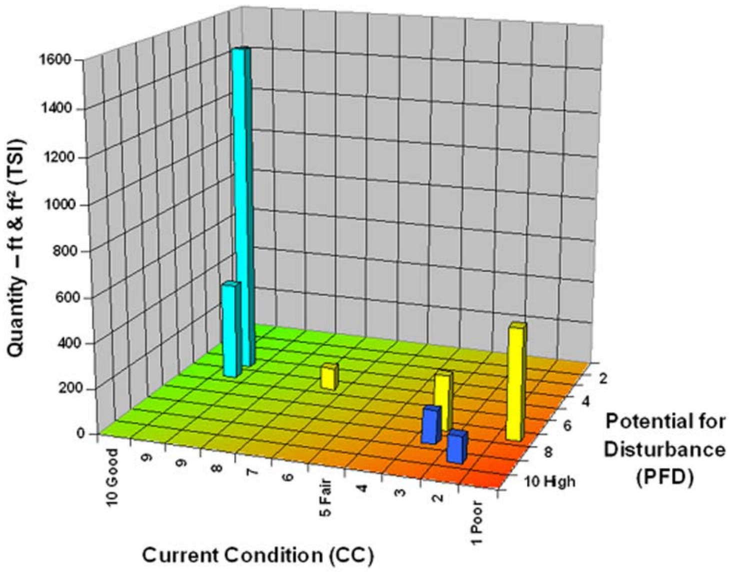

Figure 3. Three-dimensional Abatement vs O&M Decision Chart

The horizontal axes in Figure 3 have been arranged so that the ACM in the poorest Current Condition and the highest Potential for Disturbance are in the right front corner of the "floor." Also, the color of the "floor" goes from green in the left rear corner to red in the right front corner. The further into the green area the ACM is located, the more amenable it is to being managed in place. The further the ACM is into the red area, the higher the priority for abatement. Figure 3 visually separates the ACM in the boiler plant into two items in the green area that are clearly amenable to O&M, four clear candidates for removal in the red area and one in between for which either option may be acceptable. The vertical axis shows the quantities of each of these items of ACM. Even though the breeching and some pipe insulation (aqua-colored bars) constitute the largest amounts of ACM, they are also the most amenable to being managed in place.

The Abatement vs O&M Decision Chart supports the process of making asbestos management decisions on the basis of the exposure risk associated with the ACM, and the three-dimensional representation introduces the quantities of the ACM into the process. The *Customized Compliance Program for Asbestos* software also determine the costs of implementing the decisions.

POTENTIAL AND PROBABLE COSTS

Many asbestos survey reports include an estimate of cost for removing ACM based on unit costs for each type of material found. If not, typical values can be used in preparing Table 4, using data from the same boiler plant as in Table 3. The column titled "Potential Cost" is the amount that would be spent *if* the material was removed. It is the cost of abatement, for example, that would be incurred in event that renovation or demolition required prior removal of ACM that might be disturbed by construction activities. (EPA, 1990) However, the probability that the material would be removed *based solely on its Current Condition and Potential for Disturbance ratings* is almost certainly less than one, and is actually a function of those ratings.

The *Customized Compliance Program for Asbestos* software contains a "probability matrix" for all combinations of Current Condition and Potential for Disturbance. At one extreme, material in perfect condition with almost no chance of disturbance (mastic under intact non-asbestos floor tile is a good example) is found at CC=10 and PFD=1; therefore the probability of removal equals 0.01. On the other hand, it is quite certain that heavily-damaged pipe insulation close to the floor, with ratings of CC=1 and PFD=10 would be removed; hence a probability of removal of 1.0. In between are 98 other combinations of ratings and their associated probabilities of removal *based solely on the Current Condition and Potential for Disturbance of the ACM.*

Location	Area	Asbestos-Containing Materials	Quantity	Ratings		Removal costs		
				CC	PFD	Unit cost	Potential cost	Probable Cost
G	Boilers #1, #2 & #3	Roofing	500 ft²	1	7	$15	$7,500	$5,250
D	Southwest corner	Tank & fittings insulation	120 ft²	2	9	$10	$1,200	$972
C	Southwest corner	Pipe insulation	150 ft	3	8	$10	$1,500	$960
E	Boilers #1, #2 & #3	Steam drum insulation	250 ft²	3	7	$15	$3,750	$2,100
F	Boiler #4	Steam drum insulation	100 ft²	7	5	$15	$1,500	$300
B	East aisle and southeast corner	Pipe insulation	440 ft	9	5	$15	$6,600	$660
A	East aisle and northeast corner	Breeching insulation	1500 ft²	9	4	$20	$30,000	$2,400
						Total	**$52,050**	**$12,642**

Table 4. Potential and probable removal costs for asbestos-containing materials in boiler plant

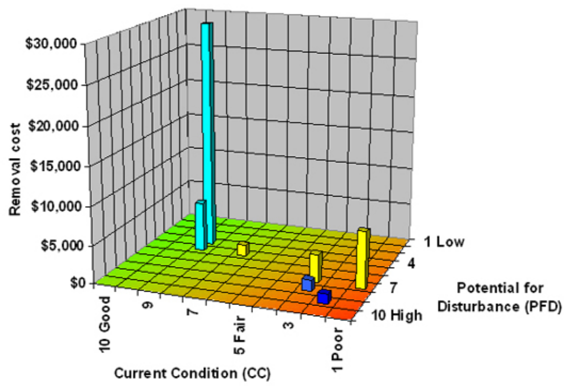

Figure 4. Potential removal cost for ACM in boiler plant

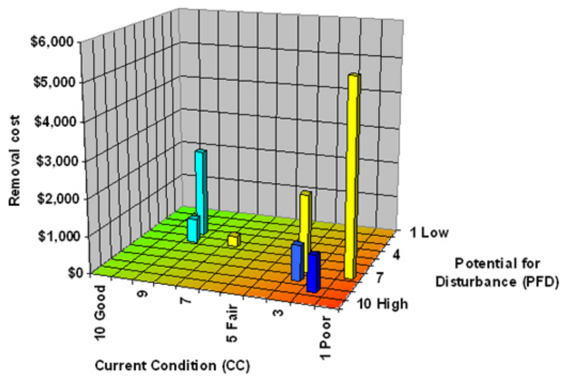

Figure 5. Probable removal cost for ACM in boiler plant

The last column in Table 4 applies these probabilities to the Potential Costs for each material to create a set of values called the Probable Costs. By summing the Potential Cost and Probable Cost columns, Table 4 shows that the expected abatement costs for the facility are reduced significantly by taking into account the fact that relatively intact, inaccessible asbestos-containing materials will continue to be managed in place as long as they stay that way. Consequently, the amount that must be budgeted for removal under these circumstances (absent any planned renovation or demolition) is a fraction of what could (and eventually will) be spent.

Figures 4 and 5 are three-dimensional representations of the Potential Cost and Probable Cost that were created for the above example by the *Customized Compliance Program for Asbestos* software. Both figures show the cost of removing the ACM in the context of its assessment ratings, and the vertical bars are in the same positions on the "floor" as in Figure 3. Notice the difference in the relative heights of the bars, however, and the different scale of the vertical (cost) axes in the figures.

This example is of necessity a simplified illustration. A large building or facility would have several sets of tables and charts, each depicting the conditions in a location within the building or facility, such as a floor of a multi-story building, a mechanical room or a separate building. The *Customized Compliance Program for Asbestos* software also has the capability to sort these tables and charts according to different types of ACM and to combine the data from two or more tables and charts into an ensemble representing part or all of a building or facility.

The cost estimating tools just described serve short-term budgeting purposes but do not address the long-term costs of managing ACM until it is removed. These costs plus the eventual costs of removal comprise the Life Cycle Costs that are discussed next.

LIFE CYCLE COSTS

Life Cycle Costs include the cost of managing the material in place (Operations and Maintenance, or O&M) as well as its eventual removal. Life-cycle Costs are based on two important assumptions:

1. The costs of managing ACM in place (O&M) can be expressed as a function of the cost of removing the ACM.
2. The relationship between removing ACM and managing it in place depends on the complexity and frequency of the O&M work that can be expected to occur.

The *Customized Compliance Program for Asbestos* software calculates Life Cycle Costs from the following factors.

The remaining service life (RSL) of the component or structure on which the ACM is installed -- Some ACM will remain installed until the building is demolished, while other ACM will be removed when some systems B such as HVAC and piping B are replaced or a space is renovated. The RSL is the number of years until the building is expected to be abandoned and demolished, until a system is scheduled for replacement or a space is scheduled for renovation. The RSL applies to the building components, not the ACM. For example, structural steel can be expected to last until the building is demolished, while a mechanical room may be scheduled for renovation in the near future.

Probability of removal before the end of the RSL -- Materials that have existing damage or Potential for Disturbance are not expected to remain until the end of the RSL of the components they are installed on. The probability the ACM will need to be removed before the end of the RSL is taken from the probability matrix for the combinations of CC and PFD discussed earlier.

Years of O&M B The higher the probability of removal before the end of the RSL, the sooner such removal can be expected to occur. The *Customized Compliance Program for Asbestos* software contains an algorithm for estimating the time until removal occurs. The ACM will have to be maintained and O&M costs will be incurred during this time.

Type of O&M B Different O&M activities are required for different types of ACM, depending also on their location. These activities include clean-up from fiber release episodes, HEPA vacuuming and wet cleaning, use of PPE for work near ACM and maintenance of asbestos-containing flooring materials. Training and surveillance also continue during this time.

Complexity & frequency of O&M B The cost of these activities will depend in part on the complexity involved and the frequency with which they must be done. These factors were described earlier in the discussion of the two-dimensional Abatement vs O&M Decision Chart. The *Customized Compliance Program for Asbestos* software contains a Complexity-Frequency matrix with five levels of complexity: very complex, complex, average complexity, simple and very simple, and five levels of frequency: very frequent, frequent, average frequency, infrequent and very infrequent. . For example, work in a mechanical room may be rated as complex because of the tight working conditions, hot environment and pressurized piping systems, with the frequency of repairs rated as average due to the age of the equipment.

O&M Cost Ratio B Each of the 25 cells in the Complexity-Frequency matrix includes the ratio of O&M cost to the Potential Removal Cost. Except for floor maintenance, O&M work involves many of the same activities and requires similar equipment as removal, although it can be done by workers with less training than abatement workers. For example, the cost of O&M for a task of average complexity that is done infrequently may be 8% of the Probable Removal Cost of that material, while the same task done very frequently could be 60% of the removal cost.

The O&M cost for each ACM if it were managed in place for the AYears of O&M@ is calculated as:
O&M Cost = (Potential Removal Cost)(O&M Cost Ratio)(Years of O&M/RSL)

The sum of the O&M Costs for all asbestos-containing materials is the expected cost of O&M for the building or facility as long as ACM remains in it.

Life-cycle cost B All of the ACM will ultimately be removed and the cost of the final is added to the O&M Cost to obtain the Life-Cycle Cost. The removal cost is realized when the material is removed for renovation or demolition. It will not be reduced by having done O&M, even in cases where O&M includes replacement of ACM with non-asbestos material. The amount of ACM removed during O&M is small relative to the total amount, and the replacement material will probably be removed along with the original ACM. All of the cost figures calculated by the *Customized Compliance Program for Asbestos* software are in current dollars and do not reflect inflation, which could be significant over a long RSL. Equally important, the costs used in these calculations do not consider the competitive nature of asbestos abatement contracting or the trade-offs between in-house and outsourced work.

Graphical Presentations of Life-Cycle Costs

Figure 6 shows the O&M costs associated with ACM in a building that contains surfacing material, thermal system insulation and miscellaneous materials. The largest expense is associated with the fireproofing in the area labeled as A-1/A-2, which drops out after it is abated in the 10th year. The next largest, which is associated with the fireproofing in the area labeled as S-2, continues for 16 years. The figure shows the cumulative costs of O&M for each ACM as well as the total cost (red bars on right). This chart shows that ~85% of the O&M expense is incurred in the first nine years.

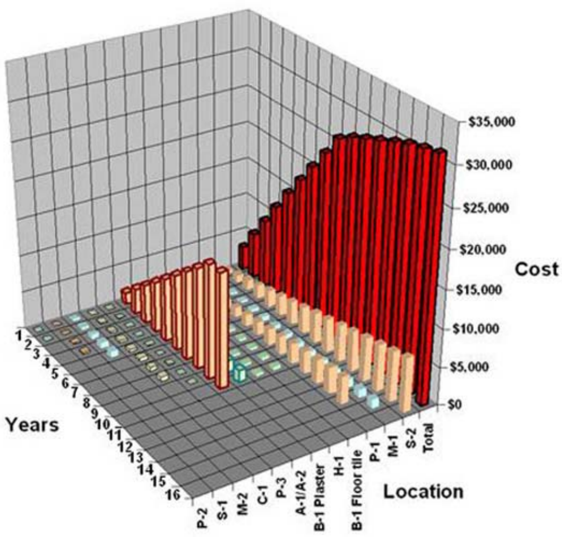

Figure 6. Cumulative O&M Costs for several types of ACM

Figure 6 shows that the largest abatement projects occur in the tenth, fourteenth and sixteenth years. Considering the cost of these projects and smaller ones, the total Life Cycle Costs – abatement plus O&M – for this building reach $140,000 in the tenth year, $200,000 in the fourteenth year and $250,000 in the sixteenth year.

CONCLUSION

The assessment protocol for asbestos-containing materials in ASTM E2356 provides risk-based information with which to prioritize decisions as to whether the ACM should be removed or managed in place. The *Customized Compliance Program for Asbestos* software allows the user to visualize these priorities in color and three dimensions, and also estimates the costs associated with the decisions. The Life Cycle Costs include on-going O&M and eventual removal, and consider several factors specific to the ACM and the building or facility.

REFERENCES

ASTM 2004, E2356-04 Standard Practice for Comprehensive Building Asbestos Surveys. ASTM International, 100 Barr Harbor Drive, West Conshohocken, PA 19428. www.astm.org

EPA 1987, Asbestos-Containing Materials in Schools, 40CFR Part 763, Subpart E. U.S. Environmental Protection Agency, Washington, D.C. Federal Register, October 30, 1987.

OSHA 1994, *Occupational Exposure to Asbestos: Final Rule.* 29 CFR Parts 1001, 1915 and 1926. U. S. Department of Labor, Occupational Safety and Health Administration.59 FR 40964 - 41162, Federal Register, August 10, 1994

Environment-i-media, Inc. 2004, *Customized Compliance Program for Asbestos*, 900 Route 620 South, Suite C101, MS 101, Austin, TX 78734. www.environment-i-media.com/ccpa

EPA 1990, *National Emission Standards for Hazardous Air Pollutants: Subpart M - Asbestos.* 40 CFR Part 61. U. S. Environmental Protection Agency, Washington, D.C., Federal Register, November 20, 1990.

A comparative study on repair techniques of reinforced concrete structures used in Maceió

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ABSTRACT

The durability of reinforced concrete structures depends on the aggressive nature of the environmental. In Maceió, like other cities of Brazilian coast, there is a high risk of reinforcement corrosion due the chloride action. Many structures were repaired using different materials and technologies along the past ten years. This is a study of some interventions in damaged structures, comparing the results of using concrete and mortars with additions like active silica, superplasticizers or carbon fiber reinforced polymers laminates in structures that needed a corrective maintenance.

First it was doing a diagnosis from the cause affecting the concrete and the rebar, to understand the whole problem and then accompany the recuperation and its behaviour until now. It was chosen four structures to study: one bridge, two commercial buildings and one pier located at the beach of Atlantic Ocean.

All the structures have been in use today and the results indicate that the first step in a repair study is understand the aggressive environment and then take the decision about the technology to use. If the repair is done without this large understanding it would be more expensive with the continued aggressively environment acting on the concrete reinforced structure.

KEYWORDS

Concrete structures, Durability, Repairing Techniques

1 INTRODUCTION

Durability of reinforced concrete (RC) structures has been the aim of several studies in all of the World. Some places have climate and geographic characteristics that request more attention in building concrete structures, like coastal areas where the risk of attack by aggressive ions like chloride must be considered still in the designing moment. Maceió is situated in Northeast Coast from Brazil. Its climate is semi-hot tropical, with medium temperature 28^0C, and relative humidity always high, over 75%. In last years, there have been a large number of bridges and buildings presenting pathologies and needing repairs in concrete and steel of structure. Different techniques and materials were used to structural repair, like mortar and concrete containing additives: microsilica, polymeric resin and superplaticizers and carbon fiber reinforced polymers laminates.

This report deals with some of the structures submitted to repairing and shows the behavior of them along the time. Understanding the whole story about those structures is understanding the story from reinforced concrete construction in Alagoas. Like in other places of the world, in Alagoas reinforced concrete became the most useful material to build structures since residential or commercial buildings, to bridges and towers. It was thought that this material had a high durability being resistant to aggressive environment, and this is not true for all situations. In many cases pathologies are observed and can implicate in structure stability and functionality losses. The cost to repair is high, so the chosen techniques must evaluate the efficiency and durability from repair, to avoid new damages in short time.

2 BACKGROUND

Corrosion of steel in concrete can be assumed like an electrochemical process, that begins when the passive layer is broken and constituted a cell with sufficient electrical potential in the steel bar. As the results of this corrosion process there are reaction products (hydrated iron oxide compounds) that occupy greater volume than its original reactants. This arise in volume depends upon the composition of the compounds, generating pressure within the concrete, which may exceed the tensile capacity of this material and result in shrinking, cracking, delamination and spalling.

Repair is restoring good conditions to the structure, demanding compatibility with existing material and durability of repairing material. Can be related repairing systems using mortars or concretes (with or without additives) to repassivate an affected location and protecting only the repaired area, physical barriers applied on concrete surface that protect all the structural area, chemical barriers like corrosion inhibitors that work over a limited area and cathodic protection providing also a limited defense. There is fundamental to study compatibility between existing and new materials, avoiding corrosion cells that could be installed due the difference of electrochemical potential caused by contact of these materials, taking in account many factors, like physical and chemical properties (density, porous e permeability).

3 CASES STUDIED

3.1 Submarine Drainage Channel

The Submarine Drainage Channel (SDC) of Maceió is the responsible for the disposal of the city's sewer in the high sea. Its structure consists of a reinforced concrete bridge that sustains all the ducts coming from the Sewer Treatment Station (STE) until the final rest of the wastewater, on the ocean's floor, as shown in Fig. 1.


Figure 1. Overview of the SDC.

All these structures were built back in 1986 and are, since then, open to one of the most danger environments to the reinforced concrete; the marine ambient. The attack of ions (chlorides, sulfates, and more) present in the water, the sensible temperature gradients and the constant shock of the waves led all the structure to be in danger of collapse since many fissures and spalling of concrete were caused by the generalized corrosion of the structural concrete steel. The worst injury happened to the stakes – most of them apparently defied physical laws while standing even when they were cracked all over their extension, as shown in Fig. 2.

Figure 2. Aspect of one of the bridge's stake.

Looking to avoid the loss of the Drainage Channel, the Water Supply and Wastewater Treatment Company of Alagoas (CASAL) intervened for the first (and, until now, one and only) time between 2000-'01 to repair all the structural damages and strengthen the bridge.

3.1.1 Repairing measures

As procedure in case of steel corrosion, all the compromised concrete and steel had to be removed from the structure to be replaced by new, and preferably stronger, material. In the case of the SDC bridge, the materials were new steel and concrete added with active silica.

Due the nature of the structure, it was too hard and not economically interesting to temporary prop the bridge in the sea. But, in the other hand, it was extremely dangerous for anyone to work in the bridge without guarantees of its stability. The responsible engineer took advantage of the fact that the stakes were hollow and chose to fill the columns with cement mortar added with silica fume (SF). By making a little hole in the top of every stake it was possible to introduce the special mortar with minimum hazard to the workers. This measure gave to the system enough resistance to remove all the cracked concrete and corroded steel by air and sand jets to, latter, be replaced by the new and final material.

After the columns were free from the contaminated concrete and the steel was substituted by another with the same effective area, the concrete with SF was laid, in a manner to make the repaired stake wider than the original. This was made to provide a protective layer to the new steel just placed in the structure. This shielding layer thickness is 10 cm and it has a disposable non-structural steel reinforcement that works to warn when the corrosion is attacking again.

Investigations made with scuba divers showed that, underneath the low tide level, no signs of the structural degradations presented above the water surface were visible. At this level, the stakes were covered with some marine mollusks and plants. So the repair works were limited to a few centimeters under the low tide level.

While one team was occupied with the columns, another was in charge to repair the beams of the bridge. To reach these structures it was used platforms stayed in the bridge's main beam. The method to repair the beams was similar to what was already been doing with the stakes: eliminate the compromised concrete and steel and replace them by new and sane material. Again the concrete added with active silica was used to cover the steel and to increase the thickness of the original piece, adding extra protection to the structure.

3.1.2 Today conditions

Late 2003 and early '04, surveys took place in the SDC to know about the conditions of the structure and to estimate how the materials are reacting to the environment. The corrosion, as seen in 2001, ceased to exist in all the repaired columns and beams. Little fissures are still visible at isolated places and seen to be result of the retraction of the paste – once the silica requests more water than ordinary concrete – and thermal dilatations [Mehta & Monteiro, 1994].

Another occurrence of degradation was a thin slack film of mortar, visible at specific places in the repaired beams. X-Ray diffraction tests revealed that this film was composed basically by quartz (from the sand), silicon sulfide and magnesium silicate. The two lasts are salts composed by very peculiar substances: silicon from the cement and active silica, sulfur and magnesium from the marine environment.

The only truth worry about the Submarine Drainage Channel is that not all the beams were repaired (and not all the repaired beams were treated through their entire length) during the intervention three years ago because, back then, these structures didn't show any symptom of corrosion and, for economical reasons, the constructors decided not to repair them. Today it's possible to see that the untreated parts of the beams are starting to crack, showing new signs of corrosion by expulsing red materials – the familiar signature of rusty steel. This is not seen among the stakes, since all of them were completely restored and reinforced.

3.2 The bridge over Águas de Ferro river channel

Built between 1979 and 1983, the bridge over Águas de Ferro river channel is an important infrastructure building, which serves the population of Maceió and the tourists that always come to visit this tropical city. The bridge is located right in the mouth of the river, away 80 meters from the beach and has the simple but functional structural model of a double supported flag, with 10 meters long and 10.8 meters width.

In the middles of 2003, investigations revealed that the bridge urged an intervention to prevent its collapse; corrosion had attacked the entire bottom the bridge's panel exposing the whole armor of the structure while cracking and throwing out layers of concrete. Having received all the prejudicial agents, present both in the river and ocean mist, during twenty years without any maintenance, made the bridge a real threat for anyone that had the need to use it.

Figure 3. Conditions of the bridge before repair measures were taken.

3.2.1 Repairing measures

For protective measures, the bridge was closed so it could be repaired without the traffic flowing over it. Being a bridge, it could not remain blocked for too long, so the structural recuperation had to adopt special techniques that would allow a fast but reliable reinforcement of the structure. By that time, the rain season had passed and the water level of the river was low, permitting to easily stay all the bridge while the repair works took place. To improve the work, sand jets were used to fast remove the injured material. Steel needles and pneumatic hammers helped take out the rests of concrete and rusted metal that couldn't be eliminated by the sand jetting.

The process divided the bridge into five lanes of action – each one of them was repaired at a time to avoid weaker too much the bridge structure. After one was finished another started to be reinforced. Following the same steps taken with the Submarine Drainage Channel, the steel was replaced by another with similar effective area to be later covered with jetted concrete. Again, looking to speed up the work, the steel was welded in quadratic frames, composed by 16 mm diameter steel bars in one direction and 6.3 mm diameter bars in the other direction.

The concrete used in the reinforcement of the structure was added with active silica and, using the same device that jetted the sand, the concrete was projected onto the flag. The final thick of the covering layer was increased to maximize the protection of the new armor and, at some portions, a finishing was made with mortar to eliminate the rough appearance that is natural of this kind of concrete.

3.3.2 Today conditions

In the beginning of 2004, inspections were made to evaluate the conditions of the bridge. At first sight, the appearance of the structure was highly improved: no more cracks or signs of corrosion can be seen in the bridge and its loading capacity was significantly improved by the use of stronger reinforced concrete.

A closer look revealed some minor fissures on the lower edges of the bridge flag. Opening these small fractures showed that they didn't pass through the finishing mortar cap applied over the structure, which lead to the conclusion that the fissures were result of a poor grip between the layers and the differential work among them.

3.3 Alagoas' Medical Science College

The Medical Science College (MSC) is an old public complex of buildings, constructed back in the sixties, that counts with a Tropical Diseases Hospital to serve both to teach the students and take care of the nearby population. Its structure consists of the classic spatial rigid frame model, made with flags supported by beams that are supported by pillars and has never received any maintenance.

By the middle of year 2000, the college was having its structure expanded (to better hold the increasing number of students) and was noted that the corrosion had spread all over the building, putting in danger the stability and safety of people that work, study or receive treatment in the College. Large amounts of concrete were already expulsed by the expansive products of the corroded steel, especially among the columns, leaving gaps with almost no concrete to resist the compression forces.

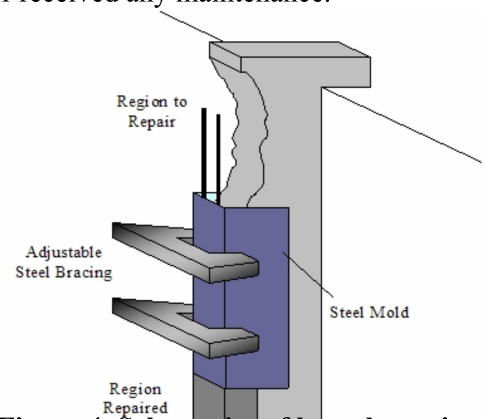

Figure 4. Schematics of how the main columns were repaired.

3.3.1 Repairing measures

Since the beginning, the main pillars, with more than 10 meters high, became a challenge to repair – intervention could cause the extremely weaken structure to collapse, so, all the work had to be done very carefully. One column was particular risky to repair as it was doing the role of an energy line pole as well. By staying this part of the structure with wooden pillars and beams, which passed through the building's windows, it became possible to continue the structural restoration.

Decided to start the repairs by the less affected pillars to later work on the most deteriorated ones, the engineer in charge opted to use concrete with microssílica and superplasticizers as recuperation material. The process, like described before, followed the standard order to eliminate the loose concrete and rusted armor, add new steel bars with the same area once before existed, and finally concrete the structure.

Detachable steel molds (Fig. 4) were used to repair the columns from the bottom to the top of them. This method (called "jumping molds") added some speed in the process and allowed the same molds to be used several times again. Adjustable steel bracers helped tighten the mold on the columns and rubbers in the corners of the mold prevented the concrete from escaping the repairing region.

The recuperation work extended through almost all the buildings: auditoriums, water reservoir, classes, and so forth. In these cases, the steel molds couldn't be used since the shape of the pillars and beams were too different from each other, requiring special crafted wooden molds. Although there were many structures, the repair material was always the same: Portland cement, sand, coarse aggregate, microssílica at a rate of 10% of the cement weight and chemical superplasticizer.

3.3.2 Today conditions

Surveys made in the beginning of 2004 appraised the conditions of the Medical Science College after the recuperation made four years earlier. The deterioration aspects caused by the corrosion, seen in 2000, ceased to exist, and now the buildings are safe to use and exhibit a clean, solid appearance.

Unfortunately, as seen in the SDC, not all the structures were repaired. Deck and beams, which showed few or no signs of degradation four years ago, are now cracking and spalling concrete motivated by the corrosion of the steel. The reason why these structures were not investigated nor repaired in 2000 are unknown, but they are proof that the corrosion problem can't be treated only in the collapsing-risk structures, or it will strike again few years later.

Another problem found was in the auditorium. One column revealed severe cracks on its base but with no evidence of corrosion. Inspections made using a hammer discovered large gaps inside the pillar structure. This fact proved that the engineer's prescription for the repair material wasn't been completely followed – at least in the auditorium's pillars – and, instead of concrete, they used mortar as covering material. Although cracked, the column was very hard to investigate. Even using a hammer, several hits were needed to open a small hole so the inner gaps could be seen.

3.4 Chemical industry bridge and installations

Inaugurated in 1977, the chemical industry C[1] is the largest in Alagoas. It manipulates and produces a large range of chemical products that are transported through a dedicated bridge to ships and from there to the national and international market. The entire facility is located in the south coast of Maceió and is heavily affected by the marine ambient and the chemical components its produce.

Such is the aggressiveness of the environment where the buildings are located that little repair actions are constant within daily. The less affected construction is the bridge, which passed through recuperation in 1992. The facility received a huge treatment in 2000 using a pre-made concrete grout called G[1]. The cause of all the interventions was always the corrosion of the reinforcement.

3.4.1 Repairing measures

The structural model of the C industry bridge is identical to the SDC, as the same company constructed them. But the similarities just go this far, the solution adopted to repair them was completely different: while concrete with SF was laid in steel molds in the SDC, the C industry bridge was repaired using ordinary jetted concrete in the columns and beams. A designing particularity of the beams made the engineers think it would be a good and simple solution to just weld together the adjacent beams with concrete, after the steel was cleaned and substituted by some new armor. To completely close the gap between the neighbor beams, some elastic rubber was placed in the upper space of the beams as shown in Fig 5.

[1] Names omitted.

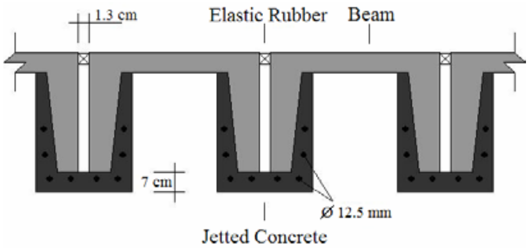

Figure 5. Transversal cut of the beams and recuperation method.

In the industry installations, the last repair took place in 2000 and used the grout G to fill the beams and columns of the spatial rigid frames within the facility. The most attacked part of the whole structure is the hydrolysis cells depot, where high voltage energy transformers are 24 hours a day working to process chemical components. As result, a constant flow of salty water mist is always saturating (and deteriorating) the structures.

After removing the contaminated concrete and cleaning the rusty armors, the beams and columns of the cell depot were covered with the special grout, then chemically cured and finally painted with epoxy-based ink to ensure maximum protection against future corrosion problems.

3.4.2 Today conditions

In 2004 visits, the conditions of both the bridge and the facility were reviewed to study the performance of the repairing materials used. Regardless all the efforts to prevent the corrosion on the cell depot, it attacked again as fiercely as usual, expulsing layers of concrete and shrinking the diameter of the steel bars. Even the use of special epoxy-based paint didn't stop the corrosion for as long as was expected. The destruction caused by the corrosion was so evident and dangerous that another repair is currently in progress since last year in the cells depot and the material is the same as the one used four years ago: the grout G. Engineers blame erroneous use of techniques and methods as the cause for the premature deterioration, freeing the material from any culpability that it may have had.

3.5 Downtown shopping mall

The SM[2] shopping mall is commercial building that sells all sorts of products, from clothing to electronics, passing through toys, cosmetics, food, etc. It's located in the downtown of Maceió and is visited daily by customers coming from almost every district of the city. In the end of 2000, the SM structure suffered the effects of a large proportions fire – which started when a short-circuit in one of the company's computer wasn't correctly treated [Correa & Chaves 2002] because of negligence in the fire-fighting procedures.

The fire burned the entire building, consuming all the products within it. The high temperature made large layer of concrete to be expulsed from the flags and the armors began to dilate so severely that the resulting stress cracked pieces of columns and beams concrete. The building became instable and with an ugly black appearance all over the wall, obligating an intense structural recuperation before it could be opened one more time to the public.

3.5.1 Repairing measures

The measures to repair the SM consisted in taking away the burned concrete; assuring that only well merged and fixed concrete remain in the structure. The armors were also investigated to find if there occurred any area lost or if the steel was too deformed. Where needed, the steel was complemented by equivalents in size and diameter. The damaged structures, after positioning the new steel, were

[2] Name omitted.

covered with pre-made mortar M[2] and, later, coated with carbon fiber. The fiber was largely applied among the flags, and the use of a special epoxy-based resin was a requirement to attach the carbon films to the repaired structure. To finish, a plaster ceiling board covered all the evidences of the structural recuperation.

3.5.2 Today conditions

Since Christmas of 2000, the SM is opened to business and its structure seems to be reacting very well to the repairs made almost 4 years earlier. The plaster ceiling boards avoid any further investigation, especially between the flags. Samples of the carbon fiber and the epoxy resin were taken to make experiments and evaluate how the new material would react to fire. As previously known, nowadays resins aren't particular famous by their fire resistance – in fact, even temperatures as high as 100 ^{o}C could melt the epoxy resin and free the carbon fiber from the structure it's supposed to reinforce. Surprisingly, the flame of a match is enough to not just dissolve this glue, but also ignite it, as seen in the experiments made with the samples.

The reason to choose the carbon fiber as material to restore the SM structure was to haste the activities and open the shopping mall to the population at Christmas time. This kind of measure was a success in terms of completing the work inside the schedule, but the price of this haste may become too expensive, if someday the structure happen to be attacked by the fire again.

5. CONCLUSIONS

Based on the cases studied above, the concrete with SF proved to be a good material to repair structures attacked by the corrosion of the steel armor, may the reason be the known filler and/or pozzolanic effect. Although good, the SF concrete is not perfect; signs of degradation due magnesium sulfate attack and cracks from the higher water demand of this type of concrete are visible in the SDC. Another problem presented was the limited range of the repair activities. When differential pore zones are created within the concrete, a corrosion cell is initiated and the steel is corroded. This is surely happening in the SDC and MSC, especially among the structure's beams.

Just as important as the material is the technique employed to repair a building – misconceptions can often ease the access of damaging substances to the concrete, just like it happened with the chemical industry bridge. Also, when a structure needs repair, it's important to evaluate not only the economical/time aspects but the repairing material efficiency too. The use of carbon fiber has its niche of application, which certainly excludes a previously burned (and still potential to catch fire) building.

4. REFERENCES

Almeida, D. C. de 2004, Estudo do desempenho de técnicas utilizadas em recuperação estrutural. *Engenharia Civil*. Maceió, Universidade Federal de Alagoas:121.

Brown, M. C. 1999, Assessment of Commercial Corrosion Inhibiting Admixtures for Reinforced Concrete. *Civil and Environmental Engineering*. Blacksburg, Virginia Polytechnic Institute and State University**:** 163.

Correa, C. N. & Chaves, D. M. 2002, Estudo das patologias e da recuperação de estruturas de concreto armado. *Engenharia Civil*. Maceió, Universidade Federal de Alagoas:86.

Lima, R. C. 2000, Avaliação da eficiência de inibidores de corrosão em reparo de estruturas de concreto. *Escola Politécnica*. São Paulo, Universidade de São Paulo**:** 169.

Mehta, P. K. & Monteiro, P. J. M. 1994, *Concreto: estrutura*, *propriedades e materiais,* PINI, São Paulo.

Estimating survival functions of building stocks

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ABSTRACT

When dealing with building stocks, one is often faced with the quite hard problem of obtaining sufficient information for studying its behaviour over time. In particular, a survival analysis of buildings in large building stocks seems an impossible task, if one wants to include all available data on construction and demolition of individual buildings in the stock. Random sampling seems also not feasible, because a priori information on the number of demolished buildings is needed.

In this paper, an example-based methodology is developped for estimating survival functions of large building stocks for which the lifetime data is not very easily acquired, e.g. for townships whose corresponding data has to be extracted from paper archives. By considering demolished as well as non-demolished buildings, techniques from censored and truncated data are applied: the event considered being demolition, non-demolished buildings are then right-censored, thus leading one to calculate the Kaplan-Meier estimator for the survival function. This methodology is applied to a German middle-sized small town, where, due to the data acquisition problem, the right-censoring itself is estimated by a random sample from a cadastral plan, and a complete inventory count is performed for the demolished part of the building stock. The resulting estimates for the survival functions are compared for various substocks generated by stratification (residential and non-residential buildings by age classes). In a further step, parametric survival functions are fitted in order to allow prediction of the behaviour of a building stock. The example building stock was studied in the project "Validierung eines integrierten, dynamischen Modells des deutschen Gebäudebestandes" (Validating an integrated, dynamic model of the German building stock) funded by the Deutsche Forschungsgemeinschaft 1999-2003, and aiming at developping methods for collecting and analysing large-scale building stock related data (such as for townships).

KEYWORDS

Building stock, right censoring, survival analysis, Kaplan-Meier estimator.

1 INTRODUCTION

The building stock is the largest physical, economical and cultural capital of a society. Nevertheless, its size, structure and the dynamics of its change are known only rather sparsely. In order to estimate effects of changing conditions, the building stock in concern must be known and models for its dynamics must be available. Therefore, information is necessary about the processes to which the changes in e.g. function, size or age of the stock are subject.

As data on these processes are not ready at hand, a methodology for dealing with this sparse information has to be developped. For doing so, a model township building stock has been studied in a research project funded by the Deutsche Forschungsgemeinschaft (DFG). Of particular interest was to find out in which way does the demolition behaviour of buildings depend on age and function. This article describes how this was performed and shows the results obtained in this study. However, it is to be mentioned that the emphasis lies on the methodology.

2 ACQUIRING DATA

Among the most important sources used for the study are the archive of the building insurance ”Sparkassenversicherung Badische Gebäudeversicherung” for obtaining dynamic building data for the town under study: Ettlingen in the Northern part of Baden (a region in the South of Germany which is about half the province Baden-Württemberg) was chosen as a model town for the study because of its size—it is a medium-sized small town almost 39,000 inhabitants—and for convenience as being easily accessible for the authors' project team. Its size makes it also a representative of the biggest class of German towns (about 24% of all German towns belong to the class of towns with 20,000-49,999 inhabitants) in which approximatively 48% of Germany's residents live. The named archive will be abbreviated as BI.

The building stock in concern are the residential and non-residential buildings of the kernel district of the town Ettlingen, which for brevity we call “Ettlingen”. It contains the historic part of the town.

Another important source was the digitalised cadastral plan of Ettlingen from the year 2002 showing the registered buildings in the year 2000. For brevity, it will be referred to as ”the cadastral plan”, and its buildings are defined to make up the building stock of Ettlingen in that year. Of course, it does not quite represent the real building stock at the time, but it seems a good enough approximation to it, and serves for defining the initial conditions.

The cadastral plans (which unfortunately are updated by the office of survey without retaining a copy of the state before the update, so that a spatial time series of a town's building stock seems difficult at the moment) contain information on location, surface and function of the buildings in concern. The most important variable for the study here is not readily read off from cadastral plans: the age of the buildings. Also missing (among other) are the (time-dependent) number of storeys and gross volume of buildings (abbreviated as gv). The values of these variables can be obtained from the BI. A complete inventory count would be desirable and is theoretically possible for the time slot 1936-1994, but not without rather large amounts of resources.

The time slot 1936-1994 is explained on the one hand by the general revision in 1936 of buildings for which the general obligatory insurance was issued by the BI which then held the monopoly, and on the other hand by the fact that the monopoly as well as the general obligatory insurance ended in Germany in the year 1994. In order to minimise errors when identifying the buildings from the BI which were not demolished until 1994 in the cadastral plan, the gap 1994-2000 was filled by looking up addresses in the office of building regulation (OBA).

3 A COMPLETE INVENTORY COUNT OF DEMOLITIONS

An inventory count of all demolitions in Ettlingen for the period 1936-1994 was taken from the BI in two steps: In a first step, all addresses in the BI were inspected upon whether they contain demolitions or not. The second step consisted in extracting detailed information about demolished buildings in the addresses found in the first step to contain such. This gives the demolitions between 1936 and 1994.

For the period 1995-2000, the OBA was consulted (for details cf. [Bradley 2004]). However, the exact number of demolitions between 1936 and 2000 remains unknown due to the quite obvious incompleteness of the OBA records (only 47 demolitions between 1995 and 2000), and due to the loss of so-called ”dead files” when the documents from the BI were archived centrally after the monopoly ceased in 1994: formerly, the building insurance files were kept locally, in each town. A dead file was often created when an address was completely demolished and no buildings were constructed on this address for some time. These files were kept in a special section of the local archive and could be ”revived” when there was a new construction. When all local archives were gathered together after 1994, only the files containing an insured building were transferred to the central archive.

The demolitions found in the way as described above are considered to be very close to a complete inventory count of the demolitions occurring in Ettlingen between 1936 and 2000, and hopefully underestimates the true complete inventory count by only a small number. The result is shown in [Table 1].

RB	NRB	changing function	total
146	846	12	1004

Table 1. The number of demolitions between 1936 and 2000 found in the BI.

4 ESTIMATING SURVIVAL FUNCTIONS

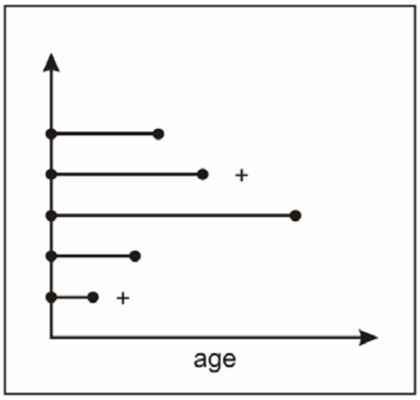


Figure 1. Illustration of right censoring. The + at the end of a beam indicates a right censored age.

The survival function of a building stock is defined as

$$S(t) = \text{probability that demolition occurs after reaching age } t.$$

In order to have a good estimation of the survival function, not only all ages of demolished buildings are needed, but also at least the observed life-times of all undemolished buildings should be taken into consideration. This is an example of right censoring: the beginning of the life-time is known, but not its ending. If for a building X denotes its life-time until (possible) demolition, and C denotes the censoring time (here: the time elapsed from construction to the year 1994 or 2000), then the observed life-time is

$$T = \min\{X, C\}.$$

This means that right censoring is equivalent to $X > C$. [Figure 1] illustrates this type of censoring: the length of each beam is the observed life-time, and + indicates a right censored building. Under some mild, but essential, conditions on the variables T for all buildings (called *independent censoring*), one can use the *Kaplan-Meier estimator* as a non-parametric estimator for $S(t)$. It is defined as

$$\hat{S}(t) := \begin{cases} 1, & \text{if } t \leq t_1 \\ \prod_{t_i \leq t} \left(1 - \frac{d_i}{Y_i}\right), & \text{if } t \geq t_1. \end{cases}$$

Here, the t_i are an increasing sequence of different life-times of demolished buildings, d_i the number of buildings demolished at age t_i, and Y_i the number of buildings having reached an age of at least t_i. The latter variable is called the number of buildings *at risk* at time t_i. [Table 2] presents a small example illustrating the Kaplan-Meier estimator. More on this subject can be found e.g. in [Klein & Moeschberger 1999].

lifetime in years	buildings at risk (Y_i)	number of demolitions (d_i)	$\hat{S}(t)$
50	21	3	$[1 - \frac{3}{21}] = 0,857$
71	18	1	$[0,857] \cdot [1 - \frac{1}{18}] = 0,810$
76	17	0	$[0,807] \cdot [1 - \frac{0}{17}] = 0,810$
79	15	5	$[0,807] \cdot [1 - \frac{5}{15}] = 0,540$
98	10	1	$[0,540] \cdot [1 - \frac{1}{10}] = 0,486$
103	9	0	$[0,486] \cdot [1 - \frac{0}{9}] = 0,486$
105	8	0	$[0,486] \cdot [1 - \frac{0}{8}] = 0,486$
121	6	1	$[0,486] \cdot [1 - \frac{1}{7}] = 0,405$

Table 2. An example illustrating the Kaplan-Meier estimator.

5 ESTIMATING RIGHT CENSORING

In our example building stock, right censoring is given uttermostly by buildings not demolished by the year 2000. As it is impossible to obtain all values of censored life-times without unreasonable efforts, a random sample of undemolished buildings of the year 2000 was taken in the way sketched in the following paragraphs.

First of all, it is a general condition for the analysis that a building stock be homogeneous. In order to homogenise the building stock, it was a priori partitioned into residential buildings (RB) and non-residential buildings (NRB) as well as into age classes as given in [Table 3]. The defining interval of each age class was chosen by considering constructive, economical and political circumstances.

Age class	1	2	3	4	5	6	7	8	9
Begin		1835	1871	1919	1934	1950	1965	1977	1995
End	1834	1870	1918	1933	1949	1964	1976	1994	today

Table 3. The age class definitions according to ifib.

As the only reliable source on the total number of buildings at a given time is the cadastral plan, it was defined to be the building stock at the time. In order to be able to perform analyses the cadastral plan displaying the stock of the year 2000 was incorporated into a geographic information system (GIS), and each building was given a unique identification number, called a *GIS-index number*. This number is reserved for the corresponding building througout its entire lifetime and is not assigned to any other building in the case of demolition. In order to determine a reasonable size of the random sample, a priori information on the mean gross volume and its variance was used to determine the length of confidence intervals. Sampling theory then gives the required sample sizes as shown in [Table 4].

	RB	NRB
Stock of year 2000	3084	2166
Reduced stock 2000	3084	2116
Required sample size	635	716
Sample size [db]	918	858
Reduced sample size [db]	918	808

Table 4. The Ettlingen building stock of the year 2000 with samples.

[Table 4] also shows the total stock of RB and NRB without the 1543 garages and 8 institutional buildings which were excluded from the study. Since the BI keeps their building information by address, it seemed practical to collect the data of all buildings in an address containing a sampled building. However, some matching problems had to be overcome: identifying buildings on the cadastral plan in the other source BI was very difficult if the corresponding address contained many entries. The only way was comparing the geometric information from the BI, obtained by superposing the available floor plans, with the cadastral plan. For only one of the two addresses in concern, this task was realised. This explains the reduced stock and reduced sample size in [Table 4]. In fact, the database [db] contains also entries for the remaining address (50 NRB), but these are obtained without superposition of floor plans, so their years of construction are very likely to be incorrect. Therefore, the building stock is redefined by discarding the buildings from this particular address.

The resulting estimated distribution of age classes is given in [Figure 2]. More details on the sampling of the undemolished part of the stock can be found in [Bader *et al.* 2001].

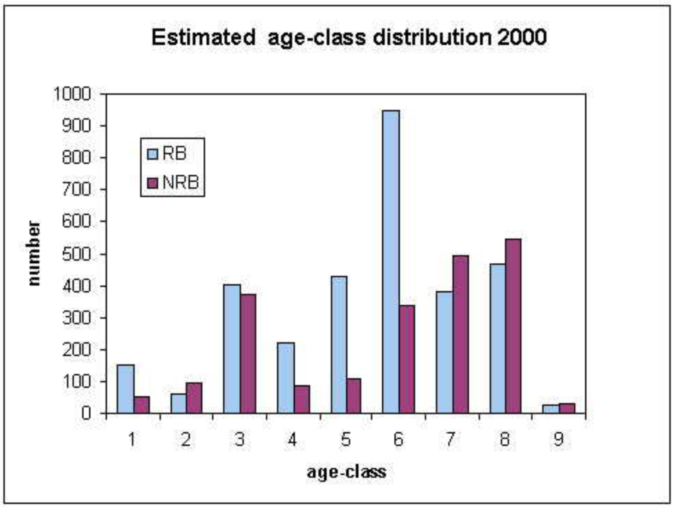


Figure 2. Estimated sizes of age classes in the reduced Ettlingen building stock of the year 2000.

However, the sample of buildings with known year of construction is not large enough for estimating the number of buildings constructed in each year. So, in order to obtain a finer distribution of the building age, a partition of each defining interval of the age class was generated from the sample (using the R-function `hist`), and the number of buildings constructed within these smaller intervals was estimated with the sample. Finally, the buildings not contained in the sample are assigned the midpoint of the corresponding interval as year of construction, as shown in [Figure 3].

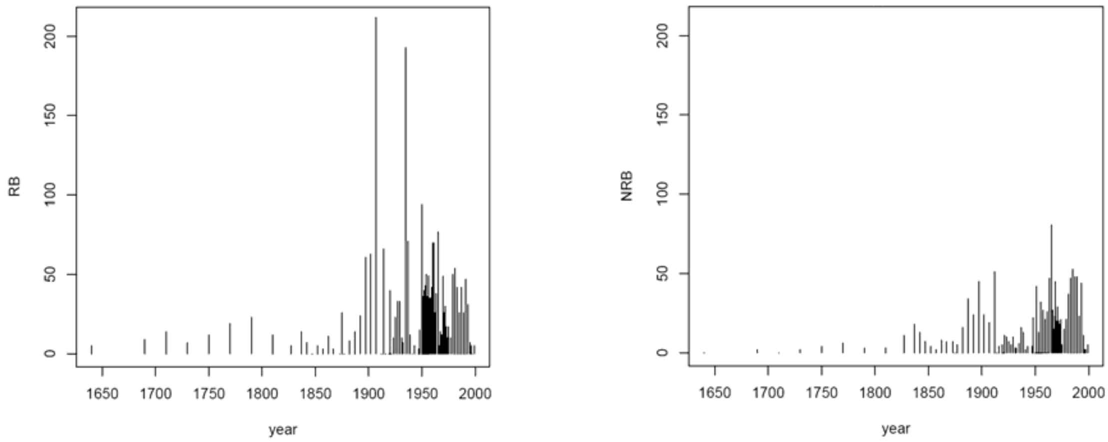


Figure 3. Assigned construction years to unknown RB (left) and NRB (right).

The very left interval was chosen to be [1600,1680), in order to take account of the fact that Ettlingen was almost completely destroyed in 1689 when French troops occupied the region during a war for succession. There do exist undemolished buildings from the time before 1698, e.g. St. Martin's Church (built in 1479), but the eldest sampled building dates back to 1670. In any case, the midpoint 1640 is guessed to be not too far off the true mean construction year for this (refined) age class.

6 KAPLAN-MEIER ESTIMATORS

The results obtained so far now allow the calculation of Kaplan-Meier estimators for the survival function. In [Figure 4] are shown some Kaplan-Meier estimates for residential and non-residential buildings. As the right censoring itself is estimated by a random sample we abstain from giving estimators for the variance or confidence intervals.

The observed long period of survival equalling 1 is due to a *left truncation* of all buildings demolished before the year 1936—these did naturally not enter the study and so did not change the value of $S(t)$.

It can be observed that the survival function of non-residential buildings seems always to be less than the survival function of residential buildings of the same age class (the pictures for the age classes other than 4 look similar to that in [Figure 4, bottom right]). Also do the first approximatively 100 years of the survival function seem to coincide for residential buildings of all age classes. For non-residential buildings this phenomenon does not seem to occur. However, at the time of writing this article, these observations remain conjectural, and have yet to be validified statistically.

7 FITTING PARAMETRIC SURVIVAL FUNCTIONS AND PROGNOSIS

In order to fit a parametric survival function, a log-log plot of the *cumulative hazard function*

$$H(t) = -\log S(t)$$

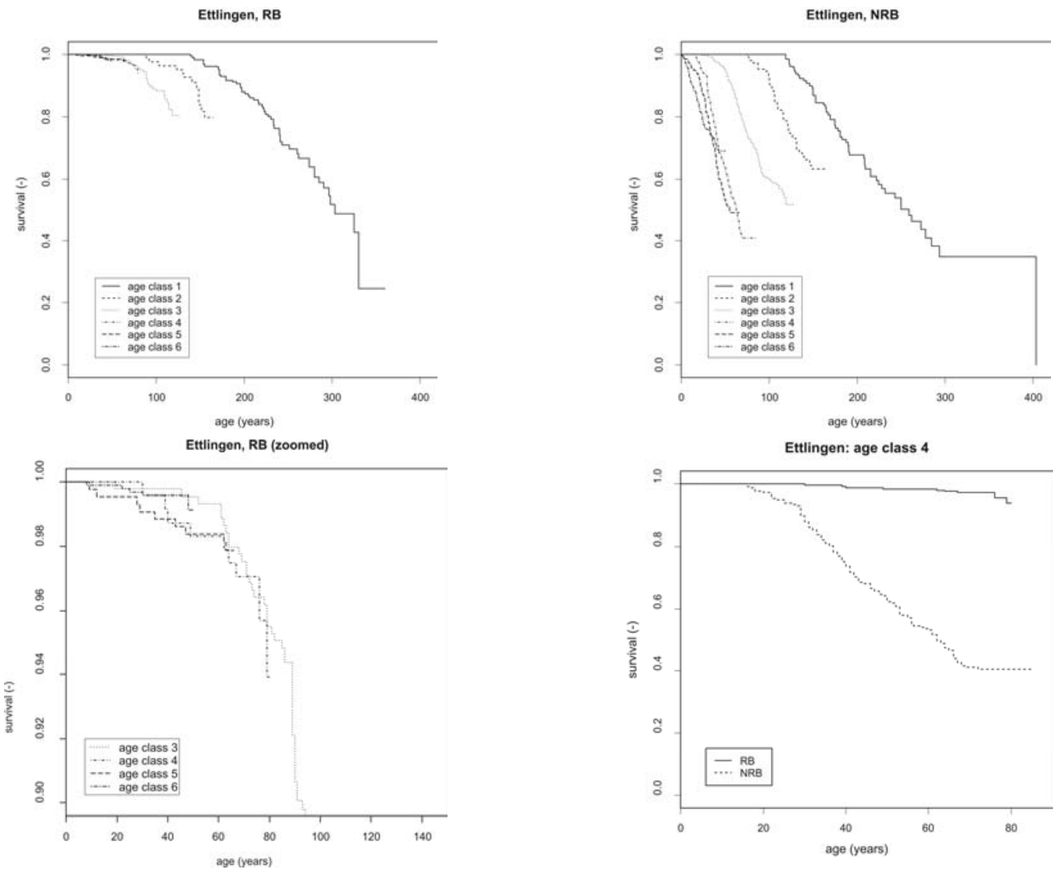


Figure 4. Kaplan-Meier estimates for survival function; counterclockwise from top right: NRB, RB, RB (zoomed), RB and NRB of a fixed age class.

is often helpful. [Figure 5] indicates that *H(t)* is approximately linear, wherefore it is not unreasonable to assume a Weibull distribution which is of the form

$$S(t) \; = \; \exp - (kt)^{a}$$

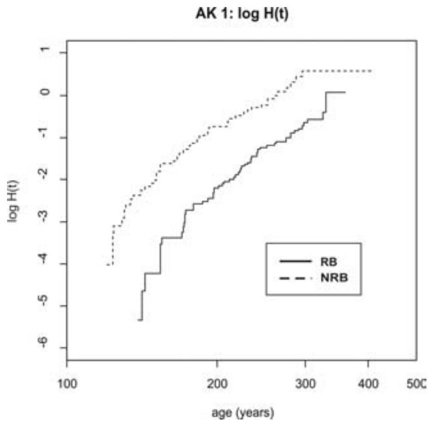
with $k = \exp\,(\boldsymbol{bx})$. The vector $\boldsymbol{b} = a$ times $(1, b_{\mathrm{AK},}\, b_{function})$ determines the shape and depends on the covariates (age class and function), and $\boldsymbol{x}$ is the covariate vector. The statistical programming package *R* yields for our dataset the following parameters:

```
> summary(survreg(Surv(meanl,ancens) ~ AK + strata(nucens)))
      .. ..
              Value Std. Error        z         p
(Intercept)  5.922    0.02453 241.4145  0.00e+00
AK          -0.213    0.00762 -27.8945 3.11e-171 # age class
nucens=r    -1.433    0.03950 -36.2874 2.55e-288 # residential buildings
nucens=n    -0.427    0.02940 -14.5286  7.99e-48 # non-residential buildings

Scale: nucens=r nucens=n  # Scale = a
          0.238    0.652

Weibull distribution
      .. ..
```

[Figure 6], [Figure 7] and [Figure 8] show that in our example building stock the predicted sizes of each age class will not have changed much in about one hundred years.



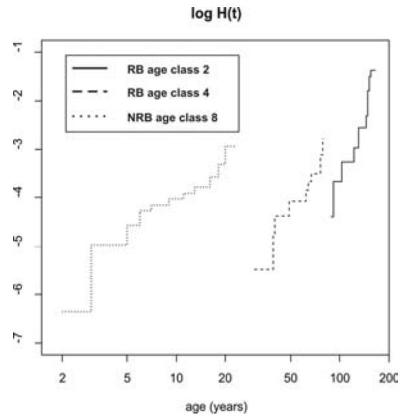


Figure 5. Log-log plots of the cumulative hazard function *H(t)*.

SUMMARY

An example-based methodology is developped for estimating survival functions of building stocks for which lifetime data is not easily obtained (e.g. German townships) and applied to a German middle-sized small town. Using a complete inventory count of all demolished buildings from 1936 to 2000 and a random sample of non-demolished buildings of the year 2000, a Kaplan-Meier-type estimator of the survival function is calculated for residential and non-residential buildings of various age classes. A difference in survival behaviour of residential and non-residential buildings can be seen from the plots of the corresponding Kaplan-Meier estimators. Finally, Weibull distributions are fitted to the survival functions in order to obtain a prognosis of the sizes of each age class in the year 2100.

ACKNOWLEDGEMENTS

The authors thank the BI and the town of Ettlingen for allowing access to data, and also the many helpers for their extraordinary job on the quite tedious and time-consuming data collecting and processing. Also, Wolfgang Bischoff and Bernhard Klar are warmly thanked for fruitful discussions.

REFERENCES

Bader, G., Schwaiger, B. & Ferrara, C. 2001, *Methode zur Analyse von Gebäudebeständen, Alterungsmodelle und Prognose von Stoffströmen mit Anwendung auf den Gebäudebestand der Stadt Ettlingen,*Research Working Paper in the version of Aug. 15, 2001, Institut für Industrielle Bauproduktion, Universität Karlsruhe.

Bradley, P.E. 2004, *Validierung eines integrierten, dynamischen Modells des deutschen Gebäudebestandes: Statistischer Teil,* Technical Report 2004-01, Institut für Industrielle Bauproduktion, Universität Karlsruhe.

Klein, J.P. & Moeschberger, M.L. 1999, *Survival Analysis. Techniques for Censored and Truncated Data,* Statistics for Biology and Health, Springer, Corrected third printing.

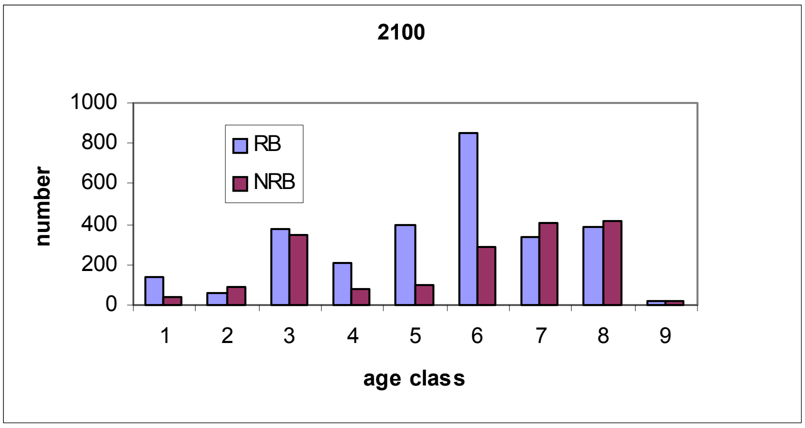


Figure 6. Estimated age class distribution of the year 2100.

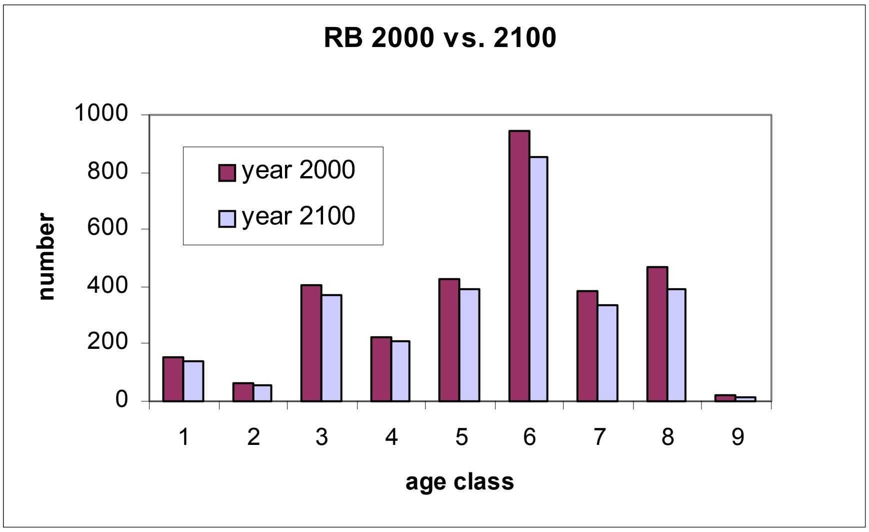


Figure 7. Estimated age class distributions RB.

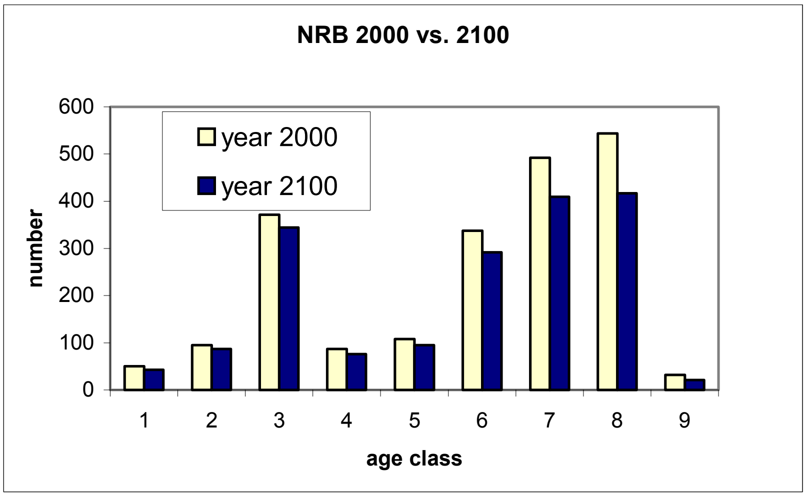


Figure 8. Estimated age class distributions NRB.

Durability and Planned Different Obsolescence through Structure/Envelope Technologies: a Case Study

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ABSTRACT

Durability, a main goal in the building process, and also the seventh "hidden" essential requirement considered in 89/106/EEC directive, regards the capability of a construction work to maintain its performances for a specified time span in terms of normal use, within a specified maintenance plan. Durability represents a key challenge for the building process, with particular reference to planned different obsolescence for different components.

Starting from these preliminaries, the present paper illustrates a method to face this challenge through Structure/Envelope technologies; such building technologies, already implemented in many examples of vernacular architectures, consists in a complete separation between structural elements and facilities. The walls are defined after the structure elements: the walls mold an internal box that is developed within an external box. Such technology allows the usage of drywall systems, which are speedy to assemble and independent from the previous structural phase; moreover easy adjustment can be made also after the completion of the works.

The method is introduced through the case study of a housing building for immigrants to be reconverted in 25-30 years; this requirement was so compelling the designers considered different time spans for each component while considering space flexibility in internal distribution to take advantage of lower costs of rehabilitation in comparison to new construction costs. Structure/Envelope technologies were selected to match with dynamic durability, reversibility, maintainability, performance improvement, products selection optimization, assembling and disassembling options, and sustainability. Besides, such technologies seemed to be fit also to satisfy acoustic insulation requirements as well as the social integration challenge related to the specific building destination.

The paper, starting from the pre-design phase, sketches a path for building management with planned obsolescence of different components used in Structure/Envelope technologies. Finally, the paper presents a comparison between Envelope/Structure technologies and traditional technologies in terms of:

- global costs, with particular reference to maintenance costs, with reference to comparable buildings;
- durability management, considering also deterioration and service life, tentatively establishing the performance limit state according to prefixed durability parameters.

KEYWORDS

Durability, Planned Distinct Obsolescence, Structure/Envelope Technologies, Service Life.

1 INTRODUCTION

Durability represents one of the key-elements for pinpointing efficient building processes. According to the 89/106/EEC directive (Council Directive 89/106/EEC of 21 December 1988 on the approximation of laws, regulations and administrative provisions of the Member States relating to construction products), member states have to take care not only of building safety but also of health, energy economy, environment protection, and other important aspects for the public interest, such as durability [ISO/FDIS 15686-1:2000] [New Zealand 1992]. As a matter of fact, durability is one of the requirements for quality during the whole building life cycle: such a quality level establishes the total building quality together with the preliminary quality [Di Giulio 1991].

The Italian Ministry of Productive Activities substantiated the importance of durability through the criteria for the Italian application of the harmonized standards on building components according to the 89/106/EEC directive. Criteria are explained within detailed tags arranged for each national implementation standard. Each tag (according to appendix ZA) considers durability among the harmonized issues, but durability is not actually a binding rule: nevertheless some Technical Groups of UNI (Italian National Standards Body) strongly suggested its introduction [Galeotto 2004] [Circolare August 5th, 2004].

2 DURABILITY: THE SEVENTH "HIDDEN" ESSENTIAL REQUIREMENT

Durability, seventh essential requirement “hidden” in the 89/106/EEC directive, regards the capability of a construction work to maintain its performances for a specified time span and use [Chown 1999]. Basically, durability is not a material property, but it’s related to the ability of a material, component or system to maintain specified features and/or performances, according to a fixed complexity level, for a specified time span and under established exposure and use conditions [Blachère 1971] [Costantini & Cassaro 2003]; nevertheless nowadays no unanimously shared definition of durability is available. However, anyone is the definition, it is important to recognize parts of definitions of durability that may not be completely congruent with architectural design thinking [Kesik 2002], with particular reference to adaptability and energy conservation requirements, where conflicts were identified [Langford *et al.* 2000].

Some case studies are presented in the follows, with reference to incoherence in durability definition and implementation. As an instance, in Canada, the recently revised CSA S478-95 (R2001) - Guideline on Durability in Buildings - provides more specific terminology to designers: nevertheless such guideline remains entirely voluntary and as such is seldom adopted in practice [Kesik 2002]. Moreover, different design solutions may provide the same service life, but one may provide better adaptability than another: for example, roof structure and cover may be made of the same building materials, but roof shape and structural design may be different, and one may be easier to adapt than the other. A design solution which is identified as good regarding its durability may be a hindrance to adaptability. On the contrary, a design solution which is identified as poor for durability, may be poor as well in relation to adaptability: for example, a complex and not easily accessible roof. Durable windows may be characterized by an inadequate U-value [Langford *et al.* 2000]. Finally, a review of international research generally indicates that, with exception to structural elements, all components require varying levels of maintenance, repair and replacement during the life cycle of the building. The related extent and intensity energy demand vary significantly, depending on how appropriately the materials, assemblies and systems are harmonized in terms of durability, and how accessible they are for periodic maintenance, repair and replacement [Kesik 2002].

However, it is not possible to provide prescriptive guidelines for conflict reconciliation between durability and other building requirements because of the combination of variable factors in building design (start-off and in-progress buildings functions, required service life, required level of adaptability, changes in energy requirements, for instance among them). Prescriptive guidelines may carry too many qualifications and limitations, which are not applicable to all cases [Langford *et al.* 2000].

In order to address critical issues about durability, two important concepts may be explored: differential durability and functional obsolescence [Kesik 2002]. Differential durability describes how useful service life of building components (structure, envelope, finishes and services) may vary. The term may also be used to describe the whole building system by comparing the service life of the building and its functional obsolescence [Kesik 2002]. Differential durability may be referred to the following issues:

- trouble-free replacement of parts and components, in order to carry out maintenance actions aimed also to improve performance issues;
- easy reorganization of internal spaces in order to satisfy changes in customer requirements.

3 THE CASE STUDY: A HOUSING BUILDING FOR IMMIGRANTS BASED ON STRUCTURE/ENVELOPE TECHNOLOGIES

The case study concerns socially subsidized housing for immigrated workers. The concept of social housing, according to the French bylaw DM 26/94, refers to dwellings arranged by local governments, for a short time span, for citizens not possessing resources for acquiring or renting their own dwellings. The aim of social housing is to assure economic and social embedding, in order to come back to their traditional dwellings: therefore social housing is intended as a switch solution, useful for vanquishing social segregation.

The case study will be dealt with reference to French strategies, as France is long time involved in immigration matter, and therefore related housing requirements are particularly stressed; in fact the migratory stream is managed also through well-established housing strategies. We found data about 71 social residences built, with approximately 2,700 individual accommodations. The Administration, both customer and manager, possesses 400 furnished residences, spread over 53 provinces and 260 municipalities, corresponding to 70,000 individual accommodations subdivided as follows: 28% French, 56% Maghrebian and 9% Sub-Saharan Africans. At present, 58% of occupants gain 300 - 600 euros per months. Moreover, the consequence of the economic crisis is carrying the situation worse: only 38% is at work, whereas 22% is unemployed, 9% are students and 22% are retired employees. Moreover a strong ageing phenomenon is in evidence: 35% is over 55 years old and 15% of them is 56-60 years old, 15% is 61-70 years old and 5% is over 70 years old. Besides, considering the consequences of ward quarters, France shows a different approach towards new immigration phenomena: instead of addressing socially weak people towards marginal zones, the choice is integration within the city and close to central quarters too, promoting the principle of *mixité social*. Such a principle was introduced through the bylaw SRU (*Solidarité et Renouvellement Urbains*), establishing that within 20 years each city will have the 20% of social residences (see art. 55). Through such a specific disposition, the construction of approximately 22,000 social dwellings per year is to be expected within 20 years, reaching 450,000 total dwellings.

The design of the building under investigation took into account customers' distinctiveness: such a building had to be adaptable to ageing problems of residents and to problems related to accommodation of people with different cultural perspective and with different familiar situations. The average age is high and national migratory politics, with restricted incomes, will make this value to increase, giving the possibility to determine the deadline of the building, with reference to inhabitants, that are mainly retired workers. More specifically, in 25-30 years, the number of residents will tend to zero, and new perspectives will be opened: the Administration will decide whether to assign the dwellings to new immigrated people or to the newborns and to the more and more numerous disadvantaged French citizens.

Generally speaking, with reference to technical prescriptions for social residences (circ. n.74-202 of December 5th, 1974), the building internal distribution should promote dwelling habitability, preserve people' privacy, ease maintenance processes and limit construction costs. More specifically, maintainability and durability are the most important requirements: critical issues that will occur during the building life cycle have to be previewed, analyzed and solved during the design phase, in order to assure a high level of maintainability, that is to say a low level of refusal to control, during substitution, restoration and cleaning operations. With reference to maintenance operations the following aspects were taken into account:

- easy inspection of facilities in order to favour integration, control, repair and substitution, without disturbing people' privacy (technical inspection holes reachable from outside, with bathrooms and kitchens as close as possible to inspection holes designed for two dwellings for construction cost limitation);
- easy replacement of components in internal elements in order to satisfy the impending requirement of flexibility;
- easy displacement of external wall components in order to assure fast and economic substitution in case of breakdown, as well as to favour materials recycling in case of building dismission.

As durability is concerned, designers at first clearly planned the building life cycle, in terms of distributed obsolescence; as a matter of fact, after 30 years, according to variation in customer typology, inhabitants' requirements vary too, and French inhabitants might refuse in the future the minimal standards adopted today. As a consequence, the whole internal distribution was rethought, making existing surfaces larger and creating little dwellings with more than one room.

4 STRUCTURE/ENVELOPE TECHNOLOGIES FOR MANAGING BUILDINGS WITH PLANNED OBSOLESCENCE OF DIFFERENT COMPONENTS [Imperadori 1999] [Imperadori & Poli 2001] [Zambelli *et al.* 1998]

The chosen solution in the case study was a drywall system with plasterboard elements; such system may be considered in the category of "structure/envelope" (Str/En) systems, justified whenever taking into account durability with reference to maintainability.

The Str/En technologies are based on the absolute separation between structure elements and other technical elements (envelope, horizontal and vertical partitions, ...), and on the definition of elements as an assembling of specialized layers that jointly define the global performance [Masera 2004]. The walls are defined after the structure elements: the walls mold an internal box that is developed within an external box. Such technology allows the usage of drywall systems, which are speedy to assemble and independent from the previous structural phase; moreover easy adjustment should be made also after the conclusion of the works. Drywall systems make reference to spreading technologies that generally exclude the water usage, with the curious oddity of slabs completely dried, completely mortarless. Typical materials in this kind of construction are steel or wood profiles, plasterboard systems and insulation panel systems, assembled with high thickness.

The drywall technology is characterized by a strong reversibility: the juxtaposition, even when mechanically intricate, is equivalent to dismantling, with exception of bonded parts. The assembling mechanisms concur to remove small portions of the building system – those interested by deterioration - without removing extended elements related to the morphology of the building. Reversibility corresponds also to maximum ease and speed during the connection. That means therefore considering "continuous" and equivalent the process of establishing, maintaining and adapting (modernization, increment, variation, etc) a prefixed group of requirements during life cycle time. The drywall assembling processes, compared to other assembling procedures, represent the best solution for making maintenance easy in order to keep up the initial characteristics of the building during the whole life cycle. As a matter of fact building components and drywall assembling methods are intrinsically disposed for maintenance process, not only due to their workability, but also due to facilitated cleaning, inspections and reviews.

Therefore the drywall technology, beyond confirming performance advantages, is coherent with the concept of continuously upgraded building, emphasizing durability and maintainability aspects with particular reference to different obsolescence. Moreover such systems concur to catch up a higher comfort level in respect of buildings realized with traditional technologies. Finally, such building paradigm favours materials optimization, shows obvious advantages on the building site, enhances performances, takes greater benefits from quality control, and promotes improvements in building procedures.

As far as materials optimization is concerned, the adoption of innovative technologies involves both the awareness and judgment of technical and operational characteristics of a huge and diversified range of materials and components, and the capability to catch a glimpse of new usage fields through analysing their potentialities. Such behaviour is essential in order to solve structural issues, to give shape and technical contents to the wall systems, to provide a proposal for distribution and organization problems of internal spaces and to rationalize energy management. The main concept of these technology solutions is based on lightness-oriented concepts: less materials means lower weight of structural and not structural elements, wider structure-free spaces, and reduction of stress in structural parts.

As far as advantages on the building site are concerned, the drywall systems concur to get ready construction procedures based on assembling accuracy of different components, including the employment of specialized operators fit to assure high quality levels during the construction phase. Construction sites based on drywall technologies concur to cut time and constraints of the traditional construction site, overcoming some limits related to materials characteristic, being based on procedures and materials that facilitate the construction development according to rationality and continuity, with reduced or no meteorological conditioning, cutting time of assembly and displacement of provisional elements, cutting down waiting time for constructed elements as structural parts are allowed to work immediately after they are built.

As performance and quality control are concerned, the use of industrially produced components, assembled within the construction site according to well-defined procedures, concurs to predetermine qualitative and performance aspects with reference both to the single element and to the whole building. Drywall technologies are based on packages made of layers of different materials, each one with specific functions that can be separately controlled and maintained. As an instance, separating the wall system and the structure, it is possible to act on critical points where thermal bridges are formed; moreover facades can be arranged according to different geometries and shapes, with reference to internal solutions and to climatic issues according to building orientation.

The accurate estimation of building times and procedures avoids cumbersome and dangerous stocking of materials on the construction site: as a matter of fact different components are delivered according to programs previously defined, taking into account their immediate and appropriate positioning within the building. Among the other advantages, drywall technologies make independent the different activities for realizing the building. As a matter of fact, while structural elements are being completed at higher floors, at lower floors the external envelope and the internal partitions can be mounted, and at ground floor technical facilities can be installed together along with internal finishing. Finally the rationalization of the building processes is based on simple procedures of junction of steel materials and plasterboard. Such connections are gas emission free: they don't consider setting and curing times typical of materials where a water content is expected to evaporate. Such aspects make also safer the construction site. Moreover drywall processes allow remarkable savings thanks to the closing down of provisional elements (such as formworks and props), that are expensive both for manpower related to assembling and disassembling procedures, and for the material costs, subdue to degradation and wear, with consequent limited numbers of reusing times.

Finally, drywall technologies favour better acoustic isolation through the internal plasterboard coating, supported by systems of steel profiles, all rested on a resilient layer, in order to obtain a complete separation [AA. VV. 1995] [AA. VV. 2000]. Such a technical solution covers not only hygrometric issues, such as thermal and acoustic isolation but also a strong fire protection (usually higher than requested by standards), thanks to fireproof characteristics of plasterboards.

5 A COMPARISON BETWEEN ENVELOPE/STRUCTURE TECHNOLOGIES AND TRADITIONAL TECHNOLOGIES

Following the above exposed considerations, the economic advantages achievable while realizing buildings with drywall technologies are now considered, adapted to a planned social housing project in the East-Northern part of Italy. The results were obtained through the implementation of Value Analysis principles [Miles 1989] with the aim to reduce not only construction costs but also the global cost.

As a matter of fact, according to informal definition of durability by Sérgio Lopes (member of CIB Working Commission W094 – Design for Durability), durability represents the ability of a structure to meet the requirements of serviceability, strength and stability throughout its intended service life, without significant loss of utility or excessive unforeseen maintenance; moreover Canadian Standards Association Document CSA A478 Guideline for Durability in Buildings describes durability as the actual period of time during which the building or any of its components performs without unforeseen costs or disruption for maintenance and repair. Consequenlty, advantages with respect to durability might be estimated first of all in terms of maintenance and repair costs; therefore, adavntages might be considered with reference to the global cost, that is to say with reference to the trade-off among:

- start-up costs (sum of investment, production and construction costs);
- managing costs (sum of costs related to usage and maintenance);
- decommissioning costs (sum of demolition costs, transferring operations costs, etc, including recovered sums from materials recycling and land value).

In this case study, after a first implementation of the Value Analysis [Weiller 1995], an higher parametric construction cost for drywall technologies resulted with reference to traditional technologies: the gap between traditional and drywall technologies amounts to 38.93 €/m^3, a gap that might induce to judge the investment uneconomical if referred to construction costs only. However, the higher construction costs are distributed in a shorter period (12 months) in respect of traditional technologies (24 months). Making a comparison between the distribution of the cost gap (38.93 €/m^3) for one year time span (the period during which the gap is reported) and distributing (in case of traditional technology), construction costs (241.70 €/m^3) on the second year (the period in which the financial exposure is to be considered), the subsequent concerns might be considered. Using traditional technologies, the building company would take care of a lower fee for a longer time (two years); opting for drywall technologies, the same company would take care of a higher fee for a shorter time and with a smaller amount of interests. For the first year a higher burden of 1.95 €/m^3 is obtained, whereas in the second year a saving equal to 12.08 €/m^3 is obtained.

Moreover, managing costs are lower for drywall technologies in respect of traditional technologies. The measured managing costs includes only energy consumption cost, because the latter represents the most significant issue in comparison to traditional building systems. Comparing hygrometric features of external walls realized with drywall technologies and external walls realized with traditional technology, with reference to the transmittance difference, the following result was obtained: drywall technologies show 0.203 W/m^{2}K for transmittance, whereas traditional technology shows 1.047 W/m^{2}K. Moreover, drywall technology allows better control in heat flow and therefore reduces fuel consumption for winter heating. The energetic parameters considered are VD (Volume Coefficient), S.E.R. (Standardized Energy Requirements) and season fuel consumption. Consequently a 58% energy saving percentage is assured.

Finally, as maintenance costs are concerned, because the drywall technology building satisfies the maintainability requirement, the repairs costs are the same in the traditional and in the drywall system: nevertheless, the drywall system favours the substitution without demolishing other components with higher duration.

6 THE PERFORMANCE LIMIT STATE ACCORDING TO PREFIXED DURABILITY PARAMETERS

The authors, bearing in mind the above considerations, as complementary aim, tentatively established the performance limit state according to prefixed durability parameters [Costantini & Norsa 1996]. The performance limit state, generally speaking, is determined as an integration of different parameters, with particular reference to different cost fractions, which are influenced by technology solutions, as analyzed in the previous paragraphs.
The performance limits are as follows:

- the duration of building life is bound to a minimum 30 years for convenient investment;
- the construction cost, higher in respect of traditional technology, must be spread on half time than needed for traditional buildings;
- the external envelope technology should assure a value of total thermal transmittance lower than 0.15 W/m^{2}K, value established by the Passivhaus Institut as an essential requirement for energy certification;
- the building features should privilege the "open systems" principle, which allows to act without substantially altering the architectonic substance of the building, in order to assure maximum internal flexibility and maintainability.

7 CONCLUSION

The paper discusses the benefits of dry wall concrete and mortar-free technologies from a maintainability point of view when compared to traditional technologies. Such advantages are shown in terms of trade-off among the different building releting costs.

Such an approach is intended to promote the usage of dry wall technologies among those who disregard such tecnologies for their higher construction costs when compared to traditional technologies; more specifically such an approach is intended to show that dry wall technologies assures low service life costs (maintenance, repair and management costs), with respect both to energy performance and to durability.

Moreover, the investigated case studies and related reported results seem to confirm dry wall concrete & mortar-free technology as an absolutely useful tool in order to manage and to assure durability in association with satisfying economics. More specifically, specific advantages are shown whenever proper selection of building elements during the design phase is made on the basis of differentiate obsolescence, while great importance should be concurrently attributed to space / usage flexibility.

8 REFERENCES

AA. VV., 1995, *L'acustica con Knauf: soluzioni tecniche per l'edilizia civile*, Technical Sheet, Knauf, Milano, ordering form available online (http://www.knauf.it).

AA. VV., 2000, *Guida all'isolamento acustico*, Technical Sheet, BPB Italia S.p.a., Milano, ordering form available online (http://www.bpbitalia.it).

Blachère, G. 1971, *Saper costruire: abitabilità, durabilità, economia degli edifici*, Editore Ulrico Hoepli, Milano.

Chown, G. A. 1999, 'Requirements for Durability and on-going Performance in Canada's Objective-based construction codes', Proc. 8th International Conference on Durability of Building Materials and Components, Vancouver, Canada, 30 May-3 June 1999.

Council Directive 89/106/EEC of 21 December 1988 on the approximation of laws, regulations and administrative provisions of the Member States relating to construction products.

Costantini, M. & Norsa, A. 1996, *Prospettive di politica tecnica in edilizia – Produzione e qualità,* FrancoAngeli, Milano.

Costantini, M. & Cassaro, F., 2003, "Business Process Reengineering and Value Analysis: tools for Durability?", Proc. La gestione della Durabilità nel Processo Edilizio - Management of Durability in the Building Process, Milano, 25-26 June, Politecnico di Milano, organizzata dal CIB.

CSA S478-95 (R2001) Guideline on Durability in Buildings

Di Giulio 1991, *Qualità edilizia programmata – Strumenti e procedure per la gestione della Qualità nel Ciclo di vita utile degli edifici*, Editore Ulrico Hoepli, Milano.

Galeotto, A., 2004, "Marchio Ce, dall'Uni la mappatura dei requisiti essenziali dei prodotti", Edilizia & Territorio: norme e documenti n° 37, 27 settembre - 2 ottobre 2004, pp. 26-27.

Imperadori, M., 1999, *Le procedure Struttura / Rivestimento per l'edilizia sostenibile*, Rimini, Maggioli.

Imperadori, M. & Poli, T. 2001, *Dubosc&Landowski: Architettura e Industria*, Brescia, Grafo.

ISO/FDIS 15686-1 : 2000, *Building and constructed assets – Service life planning – Part 1: General principles*, ISO/TC/59/SC14.

Kesik, T., 2002, *Enclosure Durability, Faculty of Architecture Landscape and Design*, University of Toronto.

Langford, D., MacLeod, I., Maver, T., Dimitrijevic, B., Zhang, X. Q., McElroy L., 2000, *Research Outcomes: Durability, Adaptability and Energy Conservation of Buildings, Final Report*, University of Strathclyde, Glasgow.

Masera, G. 2004, *Residenze e risparmio energetico*, Il Sole 24Ore, Milano.

Miles, L. D. 1989, *Techniques of Value Analisys and Engineering*, Eleanor Miles Walker Executive Director Lawrence D. Miles Value Foundation, United States of America.

Ministero delle Attività Produttive, Circolare - 5 agosto 2004, Norme armonizzate in applicazione della direttiva 89/106/CE sui materiali da costruzione - Appendice Za.

New Zealand 1992, *The Building Regulations 1992*, Wellington.

Zambelli, E., Vanoncini, P.A. & Imperadori, M. 1998, *La costruzione stratificata a secco*, Rimini, Maggioli.

Weiller, G. 1995, *L' analisi del valore e l'analisi function-needs : riduzione sistematica dei costi e conferimento al prodotto, al massimo livello, delle caratteristiche apprezzate dal cliente : guida teorico-pratica d'applicazione*, FrancoAngeli, Milano.

Uncertainty Analysis in Using Markov Chain Model to Predict Roof Life Cycle Performance

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ABSTRACT

Making decisions on building maintenance policies is an important topic in facility management. To evaluate different maintenance policies and make rational selection, both performance and maintenance cost of building components need to be of concern. For roofing sytem Markov Chain model has been developed to simulate the stochastic degrading process to evaluate the life cycle perfornance and cost. [Van Winden and Dekker 1998; Lounis et al. 1999] Taking value in a discrete state space, this model is especially appropriate when scaled rating regular inspections and related mainteance policies are implemented in large organizations. [Van Winden and Dekker 1998]

However, many parameters in this Markov Chain model are associated with variance of significant magnitude. The propagation of these variances through the model will result in uncertainties in predicted life cycle performance and cost results. Without a solid uncertainty analysis on the simulation, decisions based on these simulation results can be unrealiable. In this paper we provide methods to estimate the range of parameter values and represent them in a probabilistic framwork. Monte Carlo method is used to analyze simulation output (life cycle cost and performance) variance propagated from these parameters through the model. These probablisitc informnation can be used to make better informed decisions.

An example is provided to illustrate the Markov Chain model development, parameter identification method, Monte-Carlo uncertainty assessment and decision making with probabilistic information. It is shown that the uncertainty propagating through this process is not negligible and may significantly influence or even change the final decision

KEYWORDS

Uncertainty Assessment, Markov Chain Model, Life Cycle Performance, Life Cycle Cost, Monte Carlo Method.

1 INTRODUCTION

Making decisions on building maintenance policies is an important topic in facility management. To rationalize selections among different maintenance strategies, two difficult aspects should be concerned. First, simulating the degradation process is a complex but necessary task. This process is probabilistic in nature due to the uncertain environmental factors in the service life duration and the variability among each individual. Hence it is desirable to simulate and predict this process in the framework of stochastic models. However, the validation and parameter identification of a stochastic model depends on the availability and format of data, coming either from controlled experiments or field tests. Since the service life interval is tens-of-years in duration, the standard lab test way is to conduct accelerated degradation experiments and make inference on the real service life through the lab testing results.[Masters 1989; Martin et al. 1996] The accessibility to field data is quite limited due to the extended time period. Therefore it is necessary to discuss how to make inference based on limited informaiton with different data format and assess its uceratinty. Secondly, since most of field data is collected by inspectors, the reliability of inspection methods needs to be tested to ensure the constistence and objectivity of inspection process. Executable and reliable methods are under development. [Saunders et al. 1998]. This paper will focus on the first aspect and assume the reliable inspection when field data is used.

In the field of roofing system degradation, there are systematical methods implemented by large scale facility organizations to record the condition of unit in an ordinal scale. In corresponding to the ordinal data inspected at regular time interval, Markov Chain modeling technique is developed to represent and predict the degrading behavior of roofing system. [Van Winden and Dekker 1998; Lounis et al. 1999]. Through the data collected from Dutch GSA, Van Winden and Dekker [1998] justify the feasibility of this model in getting insight in the relation between the maintenance budget and the overall performance, and therefore guiding the budget distribution decisions. In BELCAM project, Lounis el al. [1999] integrate Markov Chain model to optimize maintenance priority assignment for a large roofing system network. It is shown through these researches that Markov Chain model is appropriate in making predicitons and supporting decision making. However, in both cases the parameters (the coefficients in transition matrix) are provided by authors, and there are few discussions on the method to determine their values. If we have sufficient information from the inspection records to determine the transition matrix, the parameter range should be studied due to the statistical nature of data format and the variance among individual units. Furthermore, since the inspection system has not worked for sufficient time period, we do not have enough data to directly infer the parameters. Therefore the method to infer parameters under incomplete or unknown field data needs to be discussed. Besides the parameter identification problem, the uncertainty assessment of model prediction is another unaddressed topic. Van Winden and Dekka [1998] are aware of the fact that the prediction involves too many uncertainties, and thus cautiously point out that the prediction serves as a high level guideline in comparing different policies. In Lounis et al' paper [1999], since the goal is to establish a formal method to assign relative maintenance priority among a number of roofing systems, the uncertainty involved in the Markov chain model would not weaken the algorithm. In other words, although the predictions based on Markov model may be sensitive to the deviation caused by parameter variance, the decision is not too much influenced by this deviation. However, in a more general background, as will be shown in the further exposition through an example, prediction uncertainties can be influential to rational decision makings and therefore should be of concern.

2. MARKOV CHAIN MODEL DESCRIPTION

2.1 Basics

The following discrete scale value for roofing system will be used in this paper.

Condition Rating	Condition/State Description	Damage
1	Excellent	0-10%
2	Very Good	11-25%
3	Good	26-40%
4	Fair	41-55%
5	Poor	56-70%
6	Very Poor	71-85%
7	Failed	>85%

Table 1. Condition Assessment Scales (Modified from Lounis et al. 1998)

Based on such a scale system, if we observe the roofing system at fixed time interval, a random variable S_n can be used to represent the system state at nth observation. S_n takes value from 1 to 7 with certain probability, and the collection of all the random variables $\{S_1, S_2, \ldots, S_n\}$ constitutes a stochastic process. The applicability of Markov chain to represent and predict the roofing system degrading process has been discussed by previous literature. [Van Winden and Dekker 1998; Lounis et al. 1999] Such a Markov process can be described through the following formula: [Ross 2000]

$$S_n = r\,P^{(n)} = rP^n \tag{1}$$

$$E(S) = \sum_{i=1}^{n} i * S_n(i) \tag{2}$$

S_n – State vector at time step n
$S_n(i)$ – the ith component in vector S_n, meaning the probability for the system to take value i.
P – Transition matrix, where P_{ij} represents the probability of process going from state i to j.
r – Initial state vector.
$E(S_n)$ – Expected value of system state. It will be used to represent the predicted system state as a result presented to decision makers

In this article, we assume the transition matrix P takes the following form with non zero items p_i (i=1 to n, n is the number of states): $p_{ij}=p_i$, when $j=i+1$; $p_{ij}=1-p_i$, when $j=i$; $p_n=1$;p_{ij}=0 otherwise. The implication is that it is only possible for this system to staty in the current state or go to the adjacent next state for the next step. To justify this statement, it is important to assume that the inspection interval to be small enough so that the state transition will not exceed more than 1 rating class between intervals. This is realistic in practice since the inspection interval is about 2-3 years. It is to be pointed out that this assumption is for illustrating simplicity and is not a restrictive requirement for applying the method developed in this paper.

2.2 Maintenance policy representation

The maintenance policy situated in such a scaling system can always be described as "When the system hit state i, recover it back to state j". This action can be represented in matrix format. For example, M in (3) represents the following policy: whenever the state reaches 4 or upper, restore it back to state 2. With a maintanence policy matrix M the system performance can be predicted with (4). [Augenbroe and Park 2002]

$$M = \begin{bmatrix} 1 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 1 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 1 & 0 & 0 & 0 & 0 \\ 0 & 1 & 0 & 0 & 0 & 0 & 0 \\ 0 & 1 & 0 & 0 & 0 & 0 & 0 \\ 0 & 1 & 0 & 0 & 0 & 0 & 0 \\ 0 & 1 & 0 & 0 & 0 & 0 & 0 \end{bmatrix} \tag{3}$$

$$s_n = r*(M*P)^n \tag{4}$$

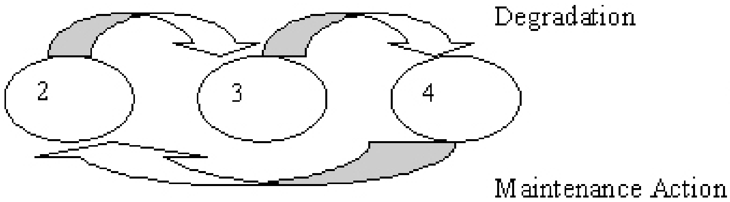
The assumption in this model is that the maintenance takes place right after the inspection. In a realistic world, maintenance action may not be conducted immediately. For example, if maintenance is taken m time periods later after the inspection, the transition matrix can be written as $P^m * M$.

2.3 Identify the Period of the Markov Chain Model

If there is no maintenance, then system will deteriorate toward the "fail" state eventually. However, with appropriate maintence interventions the system behaves periodically in the long run. For example, with the maintanence policy *M* in (3) and the transition matrix *P* shown in Fig. 1, the system will behave as following:

1->2->3->4->2->3->4->2->….

The system will not hit states higher than 4 because whenever it hits 4, it is restored back to state 2. As shown in fig1, "2->3->4" becomes a period in the long run.



$$P = \begin{pmatrix} 0.95 & 0.05 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0.93 & 0.07 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0.91 & 0.09 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0.88 & 0.12 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0.83 & 0.17 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0.75 & 0.25 \\ 0 & 0 & 0 & 0 & 0 & 0 & 1 \end{pmatrix}$$

Figure 1. Periodic Behavior of Aging Process

2.4 Calculating the life cycle performance and cost

Once the period of system is identified, the whole life cycle process can be devided to two :

1) From initial state to the first periodic state. For the example in Fig. 1, it is from 1 to 2.
2) Periodic behavior thereafter. For example in Fig. 1, it is 2->3->4.

The expected time duration of phase 1) is the time for $E(S_n)$ to reach the periodic state. For phase 2), the stationary distribution of the process is established by the following: [Ross 2000]

$$\pi = \pi * (M * P) \qquad (5)$$

We can consider π_j as the proportion of time the system stay in state *j* after the periodic behavior starts. Suppose we consider the life cycle behavior of *N* years, and it takes N_1 years to reach periodic phase, then the system will be in the periodic phase for $N\text{-}N_1$ years. For any periodic state *i*, the system will be in it for $(N\text{-}N_1) * \pi_j$ years.

Given the information on how many years the system are expected to spend in each state, it is easy to calculate the life cycle performance and cost by the following formula:

$$LCP = \frac{\sum_{i=1}^{n} i * Ti}{N}, LCC = \frac{\sum_{i=1}^{n} Ci * Ti}{N} \qquad (6)$$

T_i – years in state *i*;
N – concerned years;
C_i- The cost associated with the state i to be restored to certain goal prescribed by M. If there is no action associated with this state, it is 0.
LCP – Expected life cycle performance
LCC – Expected life cycle maintenance cost per year ($/Year)

3. ESTIMATE COEFFICIENT IN P MATRIX

If we have sufficient scale rated in-field data, it is straightforward to compute the coefficients in P matrix:

$$p_i = \frac{\sum_{t=1}^{m} p_i^t}{t}, \; s_i^2 = \frac{\sum_{t=1}^{m} (p_i^t - p_i)^2}{n+1} \qquad (7)$$

However this method subjects to the following constraints: since p_i^t reflects the overall probability of the roofing systems, the observed roofing systems should be of the same kind and expose to similar circumstances. This requirement is too restrictive to make it a realistic method. Another difficulty is that inspected data under certain maintenance policy may never reach certain states. For example, roofing systems shown in Fig. 1 will never reach state 5, 6 and 7, and the data collection work from this organization will not directly yield useful data to estimate p_5 and p_6.

It is more practical to infer parameters from a service life curve, coming either from lab test or expert opinion. When it comes to expert opinion, intuiation is to let expert directly estimate the coefficient in Markov chain model. However, this estimation requires the expert to be farmiliar with both the assumptions of Markov chain model and the specific transition behavior. Therefore it is more reasonable to let experts serve the information similar with lab test result: how does a certain kind of roofing system behave over time? The expert or lab tester should be asked to supply a curve similar to "expert judgement" curve shown in Fig. 2.

Given this curve coefficients in P can be derived based on (8) [Abraham and Wirahadikusumah 1999; Ractutanu and Sundquist 2002]. The idea is to find the parameters set so that the prediction based on this set best matches the observations.

$$Min(\sum |Y(t) - E[S(t,P]|) \qquad (8)$$

Y(t)- estimated condition state at time *t*, provided by expert.
E[S(t,P)] - Expected value of roofing condition at time t as predicted by the Markov Chain model with probability matrix *P*, compute from (1) and (2).

To illustrate the idea, given the expert judgment on service life condition under natural degradation as the sequence of points shown in Fig. 2, we can optimize (8) and derive the P matrix as shown in (9). The comparison of expert judgement and model prediction is demonstrated in Fig. 2.

$$P = \begin{pmatrix} 0.95 & 0.05 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0.93 & 0.07 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0.91 & 0.09 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0.88 & 0.12 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0.83 & 0.17 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0.75 & 0.25 \\ 0 & 0 & 0 & 0 & 0 & 0 & 1 \end{pmatrix} \qquad (9)$$

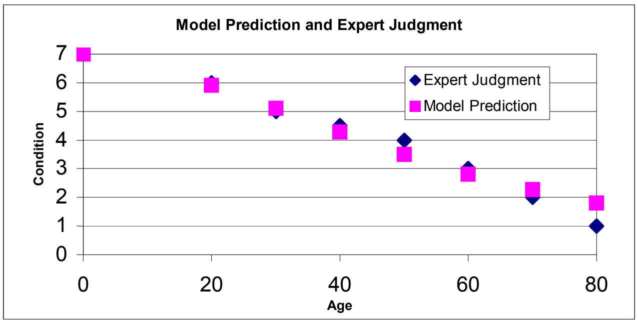


Figure 2. Expert Judgment Data and Model Prediction

4. Uncertainty Assessment through Monte-Carlo Method

This section is to evaluate the uncertaitnies propogated through the coefficients in P and C matrix. These parameters can not be accurately measured with negligible variance; rather, they are estimated or inferred from estimations. They will not take the exact estimated value but take values around it with some probablisitc distributions. The most commonly used technique in evaluating the variance of simulation results casued by parameter uncertainties is Monte-Carlo method: first associate all parameters with certain probability distributions; then their values are randomly generated for simulation. It is mathematically proved that the prediction result approaches the normal distribution no matter how the assumed parameter distributions look like. Practically, it is suggested that the minimum number of simulation required is 60-80 times. [Lomas and Eppel 1992]. In this paper we will estimate the varianace of *LCC* and *LCP* propogated by matrix *C* and *P*.

First we need to estimate the input parameter distributions. If the service life curve comes from lab tese, its variance should be supplied. If it comes from the expert estimation, the same expert should be asked to also provide the 95% confidence interval boundary. It is more natural for an expert to give the boundary corresponding to the certain state, that is to say, he will normally judge that it will take 20 years for the system to fall to "Very good" status, and he is 95% sure that it will take 18 to 22 years for this transition. To represent it, we would denote the estimation point as (i, n) where i is state and n is the time interval to reach it. Therefore the expert judgment is in fact a sequence of points. Suppose for every i with 95% probability it takes $0.9n$ to 1.1n years to reach this state, then the probabilistic distribution for each point would be $(i, N(n, (0.0561n)^2))$.

To estimate the variance of C_{ij} depends also on the expert opinion when statistical data is not available. If C_{ij} reasonably represents the budget cost from state I to state j, then the probabilistic distribution of C_{ij} represents the actual cost from state I to state j. For example, if we use the scale provided by Lounis and the material is evaluated as being in state 3, the damage extent can be any where between 56%-70%. The actual cost to improve the material from state 3 to state 4 (which is defined as 41%-55%) will vary from case to case. Suppose the expert judges that with 95% probability that the cost will fall into the interval of $(0.8C_{ij}, 1.2C_{ij})$, then we can calculate that $C_{ij} = N(\overline{C_{ij}}, (0.0612\overline{C_{ij}})^2)$ by elementary probability knowledge.

Monte Carlo simulation can be performed based on the given information. After the simulation *LCC* and *LCA* cen be represented in a probabilistic distribution. An example will be demonstrated later.

5. DECISION MAKING

To illustrate the previous algorithm and how the probabilistic information influence the decision making, a system with an estimated service life as Fig. 1 is studied. Its corresponding *P* matrix has been inferred earlier. Its maintenance cost matrix is as (10), and the coefficients are the percentage of the cost of new roofing system. [Van Winden and Dekker 1998]

$$\overline{C} = \begin{pmatrix} 2 & 0 & 0 & 0 & 0 & 0 & 0 \\ 4 & 2 & 0 & 0 & 0 & 0 & 0 \\ 15 & 4 & 2 & 0 & 0 & 0 & 0 \\ 20 & 15 & 10 & 10 & 0 & 0 & 0 \\ 50 & 40 & 25 & 15 & 10 & 0 & 0 \\ 65 & 55 & 37 & 23 & 15 & 10 & 0 \\ 80 & 70 & 50 & 30 & 20 & 15 & 10 \end{pmatrix} \qquad (10)$$

Suppose the uncertainty assessment by the expert is conducted and the sequence of service life is estimated as $(i, N(n, (0.0561n)^2)$, while the cost is estimated as $C_{ij} = N(\overline{C_{ij}}, (0.0612\overline{C_{ij}})^2)$. Suppose the concerned time period is 60 years, and the decision maker wants to develop some understanding on the cost and performance between the following two policies: 1) *Policy 1*: Do nothing; 2) *Policy 2:* maintenance policy represented by *M* in (3).

For the first policy, the life cycle cost is the fixed as A. $LCC=A/60$. The performance index of the system can be estimated directly through the service life estimation without resort to Markov Chain model. The result is shown as following Fig.3. $LCP=N(5.17,0.31^2)$.

For the second policy, the simulation results are shown in Fig. 4 and 5. They can be represented as $LCP=N(5.53,0.34)$ and $LCC=N(11.17,1.33)$.

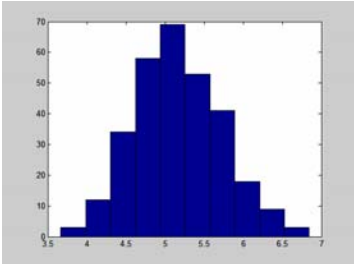


Figure 3. LCP for Policy 1

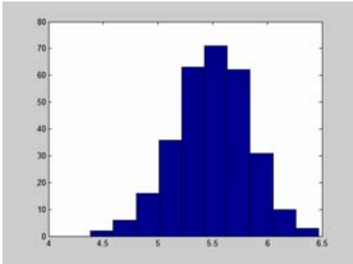


Figure 4. LCP for Policy 2

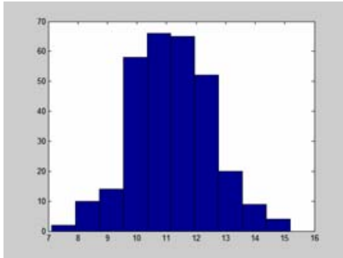


Figure 5. LCC for Policy 2

In the following it is supposed that the expected budget is 10/year and the required performance is 5, we will illustrate how the additional probabilistic information will affect the decision making in selecting the policies. Therefore the concerned consequence of each maintenance policy can be written as: *{{C_1; not C_1}; {C_2 ;not C_2}}* where C_1 represent the event *{LCP>5}* and C_2 represent the event *{LCC<10}*. Table 2 and 3 show the results both from probablistic and deterministic method.

	Deterministic		Probabilistic	
	LCP	*LCC*	*LCP*	*LCC*
Policy 1	*5.17*	*0*	$N(5.17,0.31^2)$	*0*
Policy 2	*5.53*	*11.17*	$N(5.53,0.34^2)$	$N(11.17,1.33^2)$

Table2. Result from deterministic and probablistic method

	P{C1: (LCP>5)}	*P{C2:(LCC<10)}*
Policy1	*0.7083*	*1*
Policy2	*0.9405*	*0.8105*

Table 3. Inference from probabilistic information

Without uncertainty analysis we will reject policy 2 immediately because its *LCC* value exceeds required value. However, by looking into the probablisitc information in table 3 decisions will be supported from utility theory. There are systematic methods to construct the utility value but in this paper we consider them as given. [De Wit 2001] In this simple example, we assign the same utility value, 1 when the criterion is satisfied; and 0, when it is not satisfied. Informally speaking, the utility values represent the relative importance decision maker assign for the satisfaction of different criterion.

With the quantified utility value, we can formulate our problem as a simple decision making problem: the action space is *{Policy1, Policy2}*; the consequence of the action is C_1 or C_2, shown in Table 3; and the utility value of each consequence is given as $U(C_1)=U(C_2)=1$, $U(\text{not } C_1)=U(\text{not } C_2)=0$. Thus the utility value can be computed as following:

$EU(Policy)=(U(C_1)*P(C_1)+U(C_2)*P(C_2)+U(\text{not } C_1)*P(\text{not } C_1)+U(\text{not } C_2)*P(\text{not } C_2)$
Where *EU(Policy)* : expected utility value of certain action.

$EU(Policy1)=0.7083*1+1*1=1.7083$
$EU(Policy2)=0.9405+0.8105=1.7510$

Therefore the rational decision should be *Policy2* because it results in larger expected value. Through this example we show that the Monte Carlo analysis can introduce further information and influence our decision. Under certain conditions such as in this example, the rational decision will differ from the decision induced from the deterministic information.

7. Conclusion

This paper is analyzing the uncertainties propogated through Markov chain model in predicting the performance and maintance cost of roofing system. It is demonstrated that thte magnitude of uncrtainty has significant impact in the selection of maintenance policies. Therefore it suggests that uncertainty analysis is neessary for rational decision making in this field. This paper focuses on the method under the context of roofing system, but it can be easily extended to the fileds of other building components.

For the future work, the method and analysis presented in this paper need to be supported by practical data. The realistic probabilistic distributions of service life field data, maintenance cost data are all crucial information to estimate the variance of simulation results.

8. References

Abraham, D. M. and R. Wirahadikusumah (1999). Development of Prediction Models for Sewer Deterioration. Durability of Building Materials and Components 8, Vancouver, Canada.

Augenbroe, G. and C.-s. Park (2002). Towards a Maintenance Performance Toolkit for GSA. Atlanta, Georgia Tech, GSA Funded Report.

De Wit, S. (2001). Uncertainty in predictions of thermal comfort in buildings, Technische Universiteit Delft.

Lomas, K. J. and H. Eppel (1992). "Sensitivity Analysis Techniques for Building Thermal Simulation Program." Energy and Building(19): 21-44.

Lounis, Z., M. A. Lacasse, et al. (1999). "Towards standardization of service life prediction of roofing membranes." ASTM Special Technical Publication Proceedings of the 1998 Symposium on Roofing Research and Standards Development: Fourth Volume, Dec 6-Dec 6 1998(1349): 3-18.

Lounis, Z., D. J. Vanier, et al. (1998). Effective Decision Making Tools for Roofing Maintenance Management. Proceedings of the First International Conference on New Information Technologies for Decision Making in Construction, Montreal.

Martin, J. W., S. C. Saunders, et al. (1996). Methodologies for Predicting the Service Lives of Coating Systems, Federation of Societis for Coating Technology.

Masters, L. W. (1989). "Systematic methodology for service life prediction of building materials and components." Building materials and structures(22): 385-392.

Moser (1999). Towards the Practical Evaluation of Service Life - illustrative application of the probabilisitc approach. 8th Durability of Building Materials and Components, Vancouver.

Ractutanu, A. A. G. and H. Sundquist (2002). A Markov Approach in Estimating the Service Life of Bridge Elements in Sweden. 9th Durability of Building Material and Components, Brisbane.

Ross, S. M. (2000). Introduction to Probability model, Harcourt/Academic Press.

Van Winden, C. and R. Dekker (1998). "Rationalisation of building maintenance by Markov Decision Models: A pilot study." Journal of Operational Research Society **49**: 928-935.

Service life of building elements & installations in European apartment buildings

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ABSTRACT

It is generally acknowledged that the quality, or reliability of performance, will decrease with time and that the imposed loads and the level of maintenance affect the rate of decline. Service life prediction is rather difficult, since it depends on various factors. The predicted service life is an estimate of service life derived from previous experience, extrapolation from short term or accelerated test results. Deterioration of building parts is a normal consequence of the aging process. Once the degradation processes are understood and described it is possible to predict the performance of a building component at any point in time; make relative comparisons of the performance of construction options; examine the impact of maintenance scenarios upon asset life-cycle and value. A total of 349 residential building audits were performed in seven European countries to collect data on the degradation of building parts (envelope, installations etc). The data was collected based on a standardized methodology for building audits. Follow up analysis revealed the most important influencing factors on the building deterioration and the correlation between these factors and the deterioration process. This paper provides a brief overview of the software and building audits, followed by the findings on the actual building deterioration, the influencing factors and the tool for the prediction of service lives.

KEYWORDS

Residential buildings; Deterioration; Influencing factors; Service life; INVESTIMMO.

1 INTRODUCTION

According to Eurostat, the number of households in European Union has significantly increased over the past decades. In 1961, the fifteen Member States of the European Union (EU-15) had 92 million households with an average of 3.3 persons per household. By 1995 the figure had risen to 148 million households with an average of 2.5 persons per household. The existing building stock is estimated at 164 million (193 million in EU-25), whereas only around 2 million are built every year. Between 1990 and 2000, the number of households increased by 1.1% per annum (pa) in the EU-15 (0.89% pa in EU-25) [Mantzos et al. 2003]. Technical installations are also becoming obsolete and consequently may fail to properly serve the occupants and energy related equipment consume more thermal or electrical energy compared to new and more efficient equipment.

The deterioration of building structural parts and installations is a consequence of the ageing process. The service life of a building component is defined as the period of time after installation or construction during which all properties meet or exceed the minimum acceptable performance when routinely maintained. For any building element, the anticipated service life depends on the environment, material properties, operations and maintenance. Considering that building operational costs will grow with time and that problems get worse unless some actions are taken, the need for consistent maintenance and timely renovation (repairs and restorations to good condition) or refurbishment (upgrading to better condition) actions, becomes evident. Such actions mainly have direct effect on the physical and functional building parts, as well as energy consumption and indoor environment quality. About half of the expenditure in the construction industry in Europe is spent on repair, maintenance and remediation.

INVESTIMMO is a new method and software to assess residential building renovation and refurbishment processes, for selecting long-term financial investment strategies and setting priorities for a large building stock. INVESTIMMO also includes a model able to predict the future deterioration of all building parts [Bauer et al. 2004]. The user can create and evaluate several retrofit scenarios and perform a cost analysis, taking into account: building physical and functional state of deterioration, future deterioration of building components, occupants' quality of life, energy and water consumption as well as the environmental impact from building's operation and retrofit actions, reduction of operating costs and the overall time effectiveness of the investment.

The building's physical and functional state diagnosis is based on EPIQR [Jaggs & Palmer 2000], a standardized methodology for short walk-through building audits [Balaras et al. 2000] and software [Flourentzos et al. 2000a], developed in a previous European project. Each building is divided into 50 different structural and functional parts, defined as Elements. Depending on the nature of a particular Element, several Types have to be considered. For example, a Type can refer to different construction or materials of a given Element.

The overall condition of a building is evaluated on an Element-by-Element basis. For each defined Element, the most representative problems and deteriorations have been gathered and grouped in collective categories. The diagnosis is made through visual inspection and is defined using up to four deterioration codes (designated as "a", "b", "c" or "d"), which correspond to the observed stage of degredation and are associated to specific actions/works. For example, a code "a" corresponds to the best possible condition (no actions or works required), while a code "d" corresponds to the worst condition (element is deteriorated or obsolete and needs to be replaced). The other codes ("b" or "c") correspond to intermediate stages of deterioration that require limited repair works (i.e. replace 20% of windows) or limited replacement (i.e. replace specific system components). Similar decision support software for sustainable building refurbishment has also been developed for office buildings [Caccavelli & Gugerli 2002] and hotels [Dascalaki & Balaras 2004]. The following sections present a brief overview of the building audits, followed by the findings on the actual building deterioration, the influencing factors and the tool for the prediction of service lives.

2 BUILDING AUDITS

A total of 349 residential building audits were performed in seven European countries including Denmark, France, Germany, Hellas, Italy, Poland and Switzerland to collect data on the degradation of building constructional parts and installations. The buildings cover typical architectural typologies, sizes, constructions and installations, at different states of deterioration.

The age (construction date) of the great majority of the audited buildings was 16-45 years (about 68% of the audited building stock). Only 4% were new buildings (less than 15 years) and 16% of the buildings were old buildings (over 75 years), since the main objective of this investigation was to select old buildings in order to study the degradation process. The detailed data from the audited are available in a multimedia CD-ROM, the European Residential Building Audits Database [Balaras et al. 2004]. The breakdown of deterioration codes for all building Elements in each participating country is presented in Fig. 1.

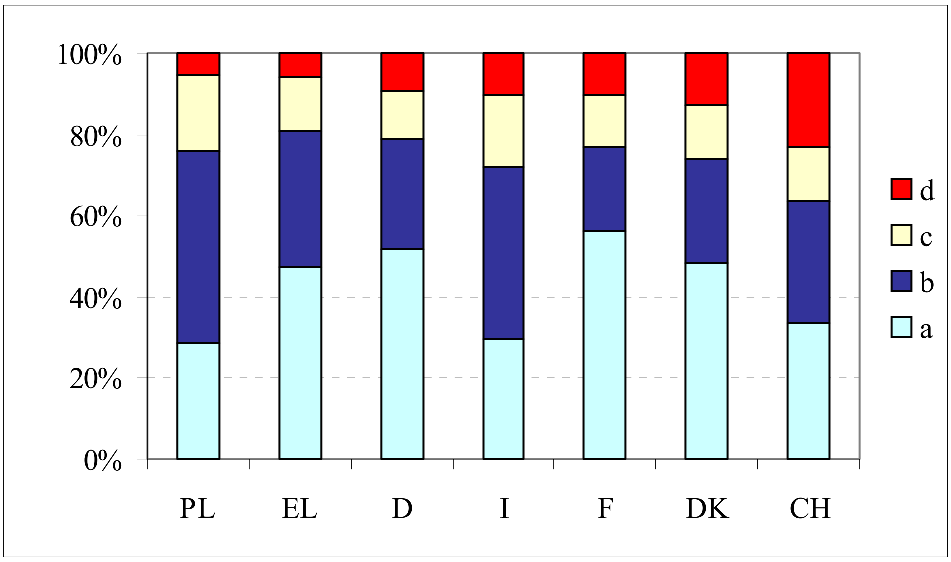


Figure 1. Breakdown of deterioration codes for building Elements in Denmark (DK), France (F), Germany (D), Hellas (EL), Italy (I), Poland (PL) and Switzerland (CH). Results are ranked in descending order of "d" codes.

Building Elements can be divided into two main categories; architectural (including all building construction components) and installation (including all systems, networks, equipment, services and electromechanical installations). From the statistical analysis of the available data, installation Elements have higher percentages of condition code "a" than architectural Elements. An overview of the overall degradation of architectural and installation Elements according to the EPIQR deterioration codes (a-d) is presented in Fig 2.

In practically all cases, at least 60% of all Elements are identified in codes (b-d) and need some kind of renovation work. The architectural Elements found in good condition (requiring minor or no repairs) are: Carpentry (92% at codes a and b), Interior joinery (92%) and Stairways and landings (91%). The installation Elements found in good condition are: Gas distribution (96%), Central sanitary services, gas installations, connections (91%) and Heat Production (88%). The architectural Elements found in worst condition (requiring major or total replacement or are missing and should be added) are: Basement-ground floor thermal insulation (66% at codes c and d), Attics (common rooms) (60%) and Roofing insulation (59%). The installation Elements found in worst condition are: Electrical Installation in apartment (54%), Low voltage installations (38%) and High voltage: common installations (34%).

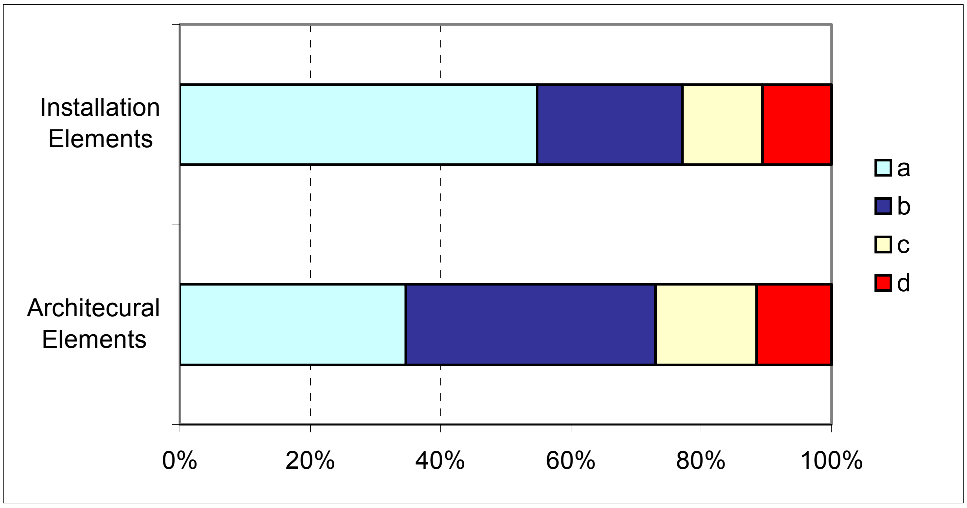


Figure 2. Breakdown of deterioration codes for all architectural and installation Elements.

A total of 34 critical factors that may influence the deterioration process of each building Type, were defined during the first stage of the data analysis. The initial list included general factors (location, weather conditions, pollution, age, building occupancy, etc.) influencing the entire building, as well as more specific factors influencing individual Elements and Types (materials, quality, maintenance, and other Element/type specific characteristics). As a first step, this was considered an exhaustive list of factors and it was further elaborated and screened as it is described in the following discussion.

The individual influence of all the factors on the deterioration of a specific Element was studied using the statistical method of classification trees [Breiman et al. 1984]; [Loh & Shih 1997]. For all the building Types with sufficient data available in the audits database, the Discriminant-based univariate splits were performed, considering the deterioration code as the dependent variable and all the proposed influencing factors as categorical variables. The method did not provide results for sample of less than 15 buildings; samples with unsuitable distribution of data in categorical variables, no variation or small variation of the deterioration code. In many cases, the sample of data for specific (national) Types was not sufficient and the distribution of the values in Types and parameters was not uniform. For these reasons, the resulting conclusions and statistical information are only indicative.

The results of the classification trees analysis include the factors ranked on a 0-100 scale, in terms of their potential importance on the dependent variable (deterioration code). From the analysis of the available data, some of the proposed critical factors were finally excluded since they were not found to have any variation in the data sample, did not influence the Elements initially considered, or their potential importance is lower than 50%. Accordingly, from the 34 initially proposed critical factors, the important factors were limited to 25. The factors excluded from further consideration are: altitude of location, type of ground (i.e. stable, soft), underground activity of area (i.e. high seismic activity, underground trains), percentage of apartments with pets, excessive vegetation near the building envelope, pipes location (i.e. shaft), window type (i.e. sliding, hinged), carpentry protection and network complexity (i.e. complex, simple). From the 25 important factors only 10 influence more than 5 Types. The factors excluded are: atmospheric pollution, environmental humidity levels, underground water, windy area, percentage of apartments with families with children, unoccupied apartments for over a year, occupants move in frequently (once a year), regular cutting of vegetation, external wall type, capillary humidity, ventilation, safety, installation inspection, roof water resistance layer and slope of pitched roof. The important factors for the building, ranked according to their average relative importance for all influenced Elements, are: Building ownership status (91%), Element quality (89%), Age of construction or last refurbishment action (87%), Type of paint (86%), Responsible for building maintenance (80%), Type of material (79%), Percentage of rented apartments (76%), Building location (75%), Cold Water (75%), Element maintenance (71%).

3 SERVICE LIFE

A predictive model describing the deterioration process of each building Element was developed. As an example, Figure 3 shows the possible time evolution of deterioration for an external coating. The y-axis represents the deterioration state measured with a discrete code (a-d). A "low quality" coating will have a short life (zone in red), and a "high quality" coating will have a long life (zone in green). These zones depend clearly not only on the manufacturing and the materials of the Element (i.e. synthetic or mineral coating) but also on maintenance (i.e. cleaning, painting) and environmental conditions (i.e. facade exposed to wind and rain or atmospheric pollution).

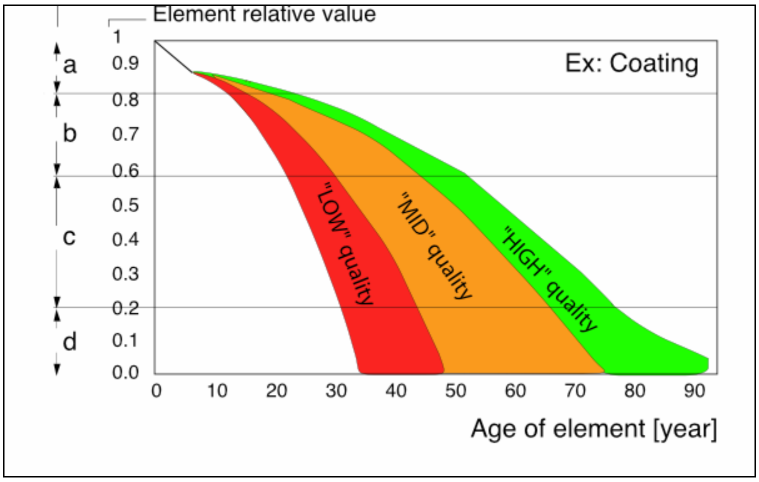


Figure 3. Illustration of the future deterioration of a building Element.

A probabilistic model was created using the survey data (about 300 couples of code and age examples for each building Element). Figure 4a (curves built using INVESTIMMO surveyed buildings) reveals that an Element can be in deterioration code "a" after 20 years with a probability of 0.6 and in deterioration code "b" with a probability of 0.4. After 80 years, the probability to be in "d" code is almost 1.

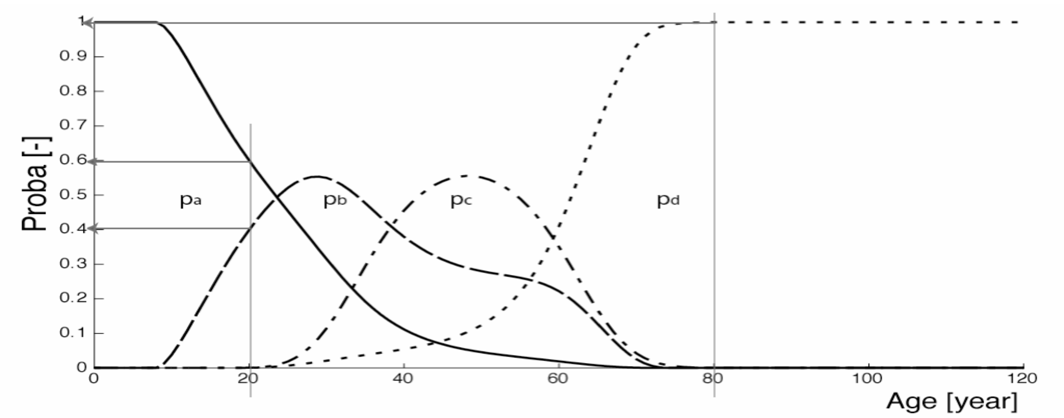


Figure 4a. Probability model curves for each EPIQR deterioration code.

These probability curves can also be plotted on a single cumulative diagram (Fig. 4b). The y-axis of the diagram can be interpreted as the building quality space (0≤q≤1). If q=0, the Element is high quality, with a long life span. It will be in d code after 75 years (beginning of the red zone when y=0). If q=1, the Element is low quality, and will be in d code after 30 years, for this example. The model makes the assumption that the quality of the Element remains constant during time. The knowledge of the quality of an Element or at least an interval of quality defines its future evolution of deterioration.

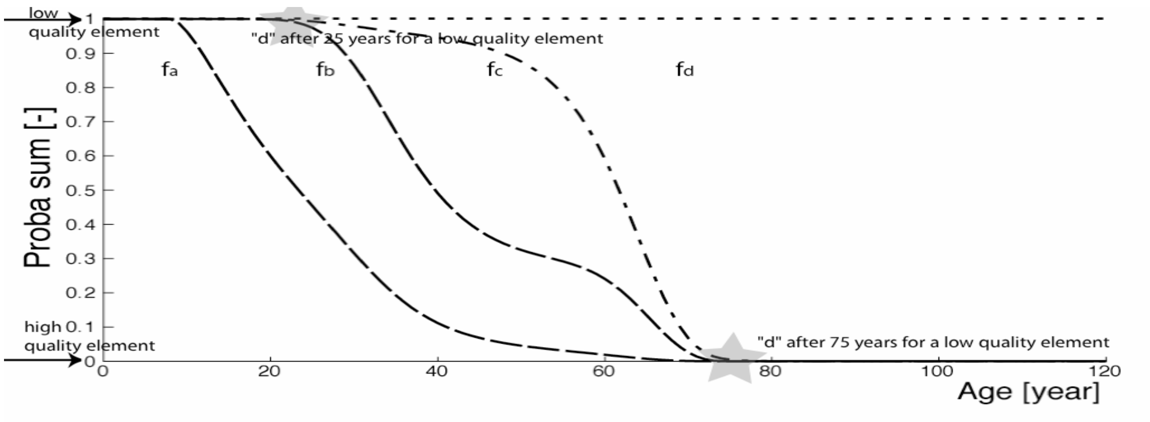
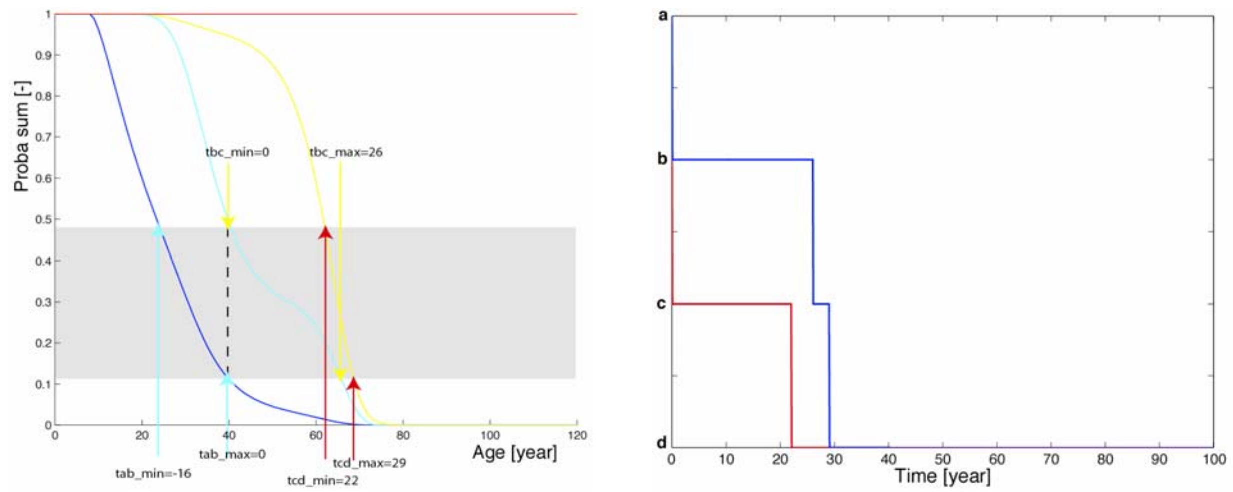


Figure 4b. Cumulative model curves for each EPIQR deterioration code.

Using the cumulative curves diagram and the deterioration code and age of an Element (duration from last refurbishment action) from the surveyed data, the predictive degradation model of the corresponding Element was developed. The changes between states are expressed in terms of intervals. These intervals represent the uncertainty of the prediction as illustrated, for example, in Figure 5 for the facade Element.



(a) (b)

Figure 5. (a) Time passage estimation for the facade Element. (b) Prediction of future deterioration states (blue and red lines define the uncertainty prediction interval).

The time passage from a to b, from b to c and from c to d can then be estimated using Figure 5(a). The obtained results are plotted in Fig. 5(b); the time is set to zero at the observation. In a more detailed and probabilistic approach, one can calculate the conditional probability to be at a current state at any time, knowing the deterioration code at a certain time [Flourentzou et al. 2000b].

When developing the degradation model only with the survey data, there is a chance that the model might be incomplete or give awkward results, due to the small data population of the statistical sample of buildings. Additional information coming from the literature or from experts as a last resort, has been included in the model by means of additional databases. For this reason, a correspondence between the definition of EPIQR Elements and the Elements for which there is literature information was also built. A "fusion procedure", based on a methodology developed by CSTB [Lair 2000], intended to extract a consensual information from all available literature data was used. The fusion software gives a cumulative probability of failure as well as optimistic and pessimistic predictions (Fig. 5). Additionally a search for trends was carried out using the survey data in order to increase the prediction accuracy. The correlations between the environmental conditions (weather impact, use and maintenance conditions, etc.) and the quality of Elements have been analysed. Finally, the "country" was kept as the major influencing parameter in the model.

The software inputs are: country; Element and type; age and deterioration code. The software provides various output graphics (distribution curves of the chosen source, calculated probability curves, calculated probability cumulative curves, future deterioration state evolution) as well as output transition times (with the minimum and maximum possible values).

4 CONCLUSIONS

A total of 349 residential building surveys were performed in seven European countries. Among the objectives of data analysis were the investigation on the factors influencing the deterioration of building Elements and the estimation of service lives. By understanding why buildings age and deteriorate, and in what time scale, building professionals can better treat these buildings and help avoid future ills when designing new structures.

The construction date of the audited buildings ranged from relatively new constructions to older buildings upto 154 years, with 67% of the audited buildings in the range of 16-45 years. The building Elements found in best condition (over 75% at code a) are: Load bearing structure (80%), Central sanitary services, Gas installations & connections (79%) and Oil storage (75%). Elements found in worst condition (over 40% at code d) that require replacement or are missing and should be added, are: Roofing insulation (47%) and Facade thermal insulation (43%).

The most important factors influencing the majority of the analysed building Elements-Types are building ownership (i.e. private, public, small or large company), quality (in terms of materials used, installation and construction) and age (time originally installed or refurbished). With regard to the building ownership status, there is not a clear trend for the breakdown of the deterioration codes. Overall, for public buildings and small private companies the ownership status appears to have a relatively negative impact with more "c+d" degradation codes, which means more serious degradation. For quality, there is a clear trend of higher codes "a" for high quality that decrease for average and low quality. As one would expect, when the age increases, the deterioration codes "a" decrease and the deterioration codes "d" increase.

The previous analysis provided some insight on the influence of individual factors on the deterioration of building structural parts and installations. However, it is realistic to expect that Element deterioration will be influenced by various factors combined impact. For a more detailed multi criteria analysis on the building deterioration, it would be necessary to collect additional data for all building Elements and Types, which was not possible during this work.

A model able to predict the future deterioration of all building Elements has been developed. This tool is very efficient to elaborate optimal refurbishment strategies in time. It is based both on a large European building survey and literature data. The model has been implemented and tested on a software platform. Its structure allows a simple future adaptation to other building data.

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6 REFERENCES

Balaras, C.A., Droutsa, K., Argiriou, A.A. & Asimakopoulos, D.N. 2000, 'EPIQR surveys of apartment buildings in Europe', *Energy & Buildings*, **31**[2], 111-128.

Balaras, C.A., Dascalaki, E., Droutsa, K. & Kontoyiannidis, S. 2004, 'European Residential Building Audits Database – ERBAD' Multimedia CD-ROM, Group Energy Conservation, IERSD, National Observatory of Athens, (ISBN 960-87905-5-7), Athens, March.

Bauer, M., Lair, J. & Wetzel, C. 2004, 'WP2 Final Technical Report: Predictive model for future deterioration', INVESTIMMO Project, European Commission.

Breiman, L., Friedman, J.H., Olshen, R.A. & Stone, C.J. 1984. *Classification and regression trees*, Chapman & Hall, New York.

Caccavelli, D. & Gugerli, H. 2002. 'TOBUS – A European diagnosis and decision making tool for office building upgrading', *Energy & Buildings*, **34**[2], 113-119.

Dascalaki, E. & Balaras, C.A. 2004. 'XENIOS – A methodology for assessing refurbishment scenarios and the potential of application of RES and RUE in hotels', *Energy & Buildings*, **36**[11], 1091-1105.

Flourentzou, F., Droutsa, K. & Wittchen, K.B. 2000a, 'EPIQR Software', *Energy & Buildings*, **31**[2], 129-136.

Flourentzou, F., Brandt, E. & Wetzel, C. 2000b, 'MEDIC. A method for predicting residual service life and refurbishment investment budgets', *Energy & Buildings*, **31**[2], 167-170.

Jaggs, M. & Palmer, J. 2000, 'Energy performance indoor environmental quality retrofit – a European diagnosis and decision making method for building refurbishment', *Energy & Buildings*, **31**[2], 97-101.

Lair, J. 2000, 'Evaluation de la durabilité des systèmes constructifs du bâtiment', Doctorat d'Université. Spécialité : Génie Civil. Université Blaise Pascal (Clermont-Ferrand) n°DU : 1222, n°ED : 219.

Loh, W.Y. & Shih, Y.S. 1997, 'Split selection methods for classification trees', *Statistica Sinica*, 7, 815-840.

Mantzos, L., Capros, P., Kouvaritakis, N. & Zeka-Paschou, M. 2003, *European Energy & Transport – Trends to 2030*, Directorate General Energy & Transport, Brussels, (ISBN 92-894-4444-4), January.

Quality and Service Life of Buildings

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ABSTRACT

ISO 15686-1 describes a method for estimating the service life of building components and assemblies based on reference service life and modifying factors (the Factor method). The Norwegian Building Research Institute (NBI) has published reference service life data for building components and assemblies in the NBI Building Research Design Sheet series since 1990. These data are mainly based on analysis of approximately 6000 individual building damage assessments carried out by NBI over the last 50 years. Two recently completed projects carried out as part of an evaluation of the changes in procedural rules in the Norwegian Planning and Building Act (PBA) in 1997, and partly as part of the NBI research & development programme "Climate 2000", will improve the basis for deciding reference service life and modifying factors in Norway.

The first project has evaluated possible quantitative changes in the number and severity of building damages as a result of the amended PBA. This study also includes a detailed review of about 700 NBI building damage assessment reports. In this study the most critical building components and assemblies are revealed. The time of damage relative to the time of construction, and the cause of the damage, has also been determined. The second project, which focused on the design process, showed that the control of the design has been improved. However, in many building projects it is still not apparent who (i.e. which designer/engineer/architect) is accountable, for the vital parts of the design. Furthermore, design of details crucial to durability and service life of building components and assemblies is often omitted, i.e. the details are left to the contractor to solve on site.

The overall conclusion of the two projects is that the amended PBA appears to be a move towards improved quality of construction. However, as the results also show, it is still a long way to go.

KEYWORDS

Service life, reference service life, building defect, building failure, building damage, factor method, design process, work execution

1 INTRODUCTION

Investigations carried out by The Norwegian Building Research Institute (NBI) [Ingvaldsen 1994, Ingvaldsen 2001] have estimated the repair cost after completion due to building defects, flaws, and damages in Norway amounts to approximately 5 % of the annual capital invested in new construction. Repair work during construction is estimated to be another 5 %. With an annual investment in new construction of NOK 100 billion, it is therefore reasonable to estimate that about NOK 10 billion is spent on repairing damage to buildings every year. The investigations carried out by Ingvaldsen also concluded that more than 60 % of the building damages registered for new buildings was due to faulty design and other decisions done prior to construction. According to the Norwegian building authorities, this was due to [Bjørkmann 1999]:

- Insufficient knowledge of the Building Act and Building Regulations in the building industry.
- Incomplete, insufficient or incorrect design as basis for construction works
- Lack of systematic documentation of conformity with regulatory requirements.
- Public control was primarily concerned with site inspection, not design control.
- Primarily the client, secondly the contractors but not the designer being accountable to the building authorities.

Full compliance with the building code should eliminate virtually all building defects. Ingvaldsen's study demonstrated that this was clearly not the case. Thus, improving the design process was determined to be a necessity in order to increase the quality and service life of the Norwegian building stock. This prompted the decision by the government to amend the Norwegian Planning and Building Act (PBA) in 1997.

The preliminary investigation done prior to amending the PBA, and the introduction of new regulations, determined the factors expected to be critical in order to increase the overall quality of the design and construction process. In the end, the revision included provisions for a new building quality control system and also new and improved requirements making the various participants in the building process (including the designers) directly accountable for mistakes made. In addition, a professional approval system for designers and contractors was introduced, thus, establishing minimum competence requirements for various building projects.

The Norwegian authorities have funded a 5 year R&D programme to evaluate the main effects of the amended PBA. The two projects briefly presented in this paper have been evaluating the effects of the amended PBA and the new regulations introduced in 1997 on the number and severity of process related building damage and on the design process as a whole.

2 BUILDING DEFECTS, FLAWS AND DAMAGE

2.1 Introduction

The main objective of the first project [Mehus et. al 2004] has been to evaluate the possible quantitative changes in the number and severity of building damage as a result of the amended PBA. The study is limited to process related building damages that are likely to be influenced by the amended PBA. The study made use of changes in the number and severity of building damage as a measure of changes in the quality of buildings. The results verify that the effect of the revised PBA has been positive; i.e. there has been a relative reduction in the number of building damage has been reduced. However, the study has also provided data of interest in deciding critical building components and assemblies and reference service life. The paper will focus on this part of the study.

Existing sources of information that has been used in the quantitative inquiry are:

- Building defects, flaws and damages identified in written reports from inspections carried out, both at the time of project completion and also in the time period of which the guarantee applies. Written reports from three major Norwegian building owners were collected and reviewed.

- Written reports from NBI's project archives

In addition, this information has been complemented with data from insurance companies, and by a survey answered by different participants in the construction- and real estate market.

2.2 Building defects, flaws, and damage under warranty

A total of 78 building projects have been examined, and together with basic information on each project, approximately 30.000 registrations of documented building defects, flaws and damages has been registered in a database developed specifically by NBI for the purpose of analysing these data. To distinguish between minor flaws on one hand and more severe damage on the other end, each registration has been classified according to a grade of consequence, related to estimated cost of repair work.

This data source is complex and the 78 buildings included in the investigation are all quite different with significant variation in for example size and types of structures. Most of the buildings included in the investigation are less than 10.000 square meters. The study shows great variations in inspection procedures both at the time of project completion, and in the time period of which the guarantee applies. For most projects, an initial inspection is carried out when the building is turned over from the contractor to the owner. However, for the remaining part of the warranty the documentation is generally less satisfactory. At one year after completion, inspections with written reports were only carried out for about 50 % of the buildings. While at two and three years after completion inspection rounds with written reports were only carried out for 10 – 15 % of the projects.

Table 1 presents a summary of the most critical building components and assemblies before and after the PBA amendment. The building component codes refer to the Norwegian Standard *NS 3451 Table of Building Elements*. The results in Table 1 shows that for building projects from after the PBA amendment, there is a tendency of more registrations of damage on *equipment*, and less on primary constructions.

Table 1
The top five most critical building components and assemblies (codes according to *NS 3451 Table of building elements*), before and after the PBA amendment [Mehus et. al 2004].

	Before PBA amendment		After PBA amendment	
	Building component/assembly	Number of registrations	Building component/assembly	Number of registrations
1	24 – Internal walls	2844	24 – Internal walls	3400
2	25 – Floors	2189	25 – Floors	1760
3	23 – Exterior walls	1627	23 – Exterior walls	1278
4	26 – Roofs	286	31 – Sanitary equipment	500
5	28 - Stairs, balconies etc.	221	26 – Roofs	139

2.3 NBI's project archives
NBI's project archives, together with the NBI Building Research Series, represents one of Norway's most important sources of knowledge on types of building damage and related causes. Altogether NBI has approximately 6,000 building damage assignment reports in its project archives from the period 1963 – 2001. The data are stored in a Microsoft SQL-server database. Information on the following key data is registered electronically in the archives: Client, project number, project leader, report date, built (year), building address, construction method, keywords and summary. The summary provides

overall information on the type, location, scope and cause of the damage. Paper copies of each damage assignment reports are easily accessible in the institute's central archives. Detailed information on each assignment is obtained from the paper copies.

The analysis has included all building damage assessment reports in the NBI Project Archive where:
- The building has completion date in the period 1992 – 2000
- The building damage is considered to be process related (i.e. damages that are likely to be influenced by the amended PBA)

This gave a total of 698 building damage assessment reports which were included in the analysis.

Figure 1 presents four vintages of buildings from the NBI Project Archive (completion year 1993 – 1996), and the distribution of damage registered with respect to time for the first seven years after completion. As Figure 1 shows, the distribution is relatively similar for each of the four vintages with approximately 60 % of the damage occurring within the 3 first years of operation.

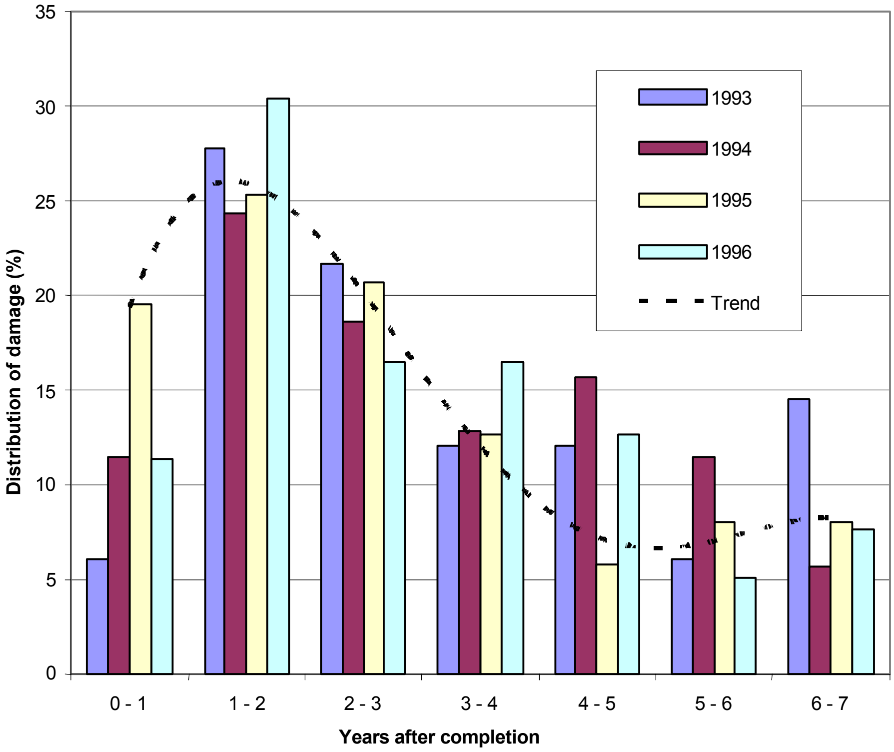

Figure 1
NBI's project archive: Time of damage during the 7 first years after commissioning expressed in percentages, for buildings finished during the period 1993 – 1996 [Mehus et. al 2004].

2.3.1 Critical building components and assemblies

An interesting aspect is how the process related building damage reported during the current period of time, is related to critical building components and assemblies. Figure 2 illustrates this distribution. As shown in the figure, roofs and external walls above the ground level (i.e. the building enclosure) are the most frequently damaged building components, representing about 47 % of all assignment reports in the NBI project archive included in this analysis. However, there is a considerable variation in building damage and critical building components and assemblies. This can be illustrated by the category "other building components and assemblies" in the figure, covering 35 % of all causes of damage.

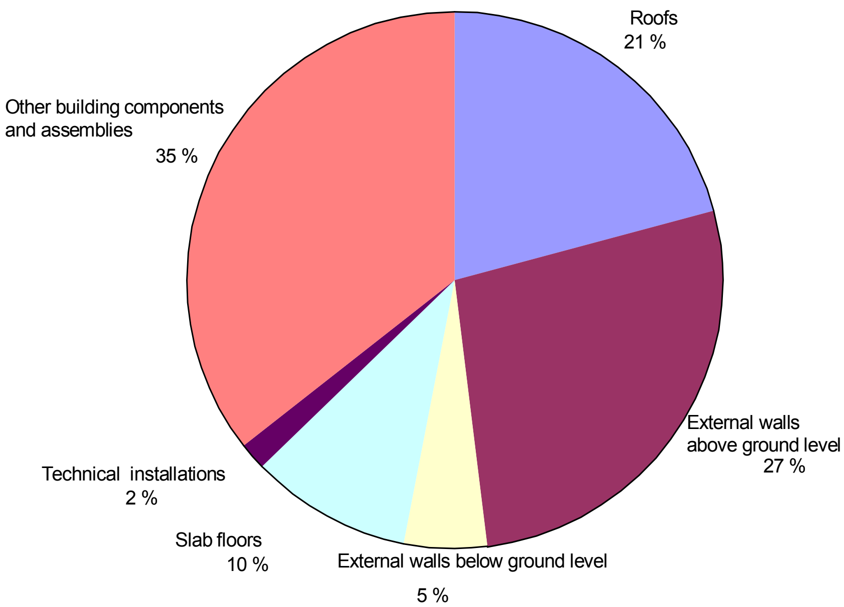

Figure 2
NBI's project archive: Distribution of registered cases on critical building components and assemblies, for buildings with year of construction within the period 1993 – 2002 [Mehus et. al 2004].

2.3.2 Moisture related building damage

Moisture related building damage was chosen for further analyses because previous inquiries have proved this to be a frequent and important type of damage. An ongoing investigation of building damage cases in NBI's project archives indicates that more than three-quarters of the total number of buildings examined in the period 1988 – 2002 had suffered water and/or moisture damage [Lisø et al. 2004]. In addition, this type of damage often has a relatively short "incubation time", and is therefore quickly identified. This makes this type of damage relevant considering relatively new buildings constructed according to the amended PBA.

For damage related to moisture, an additional inquiry was made to determine the cause of damage (i.e. where did the moisture come from?). This is illustrated in figure 3. The rest of the building damages were not analysed in detail to determine neither the type of damage nor the cause of the damage.

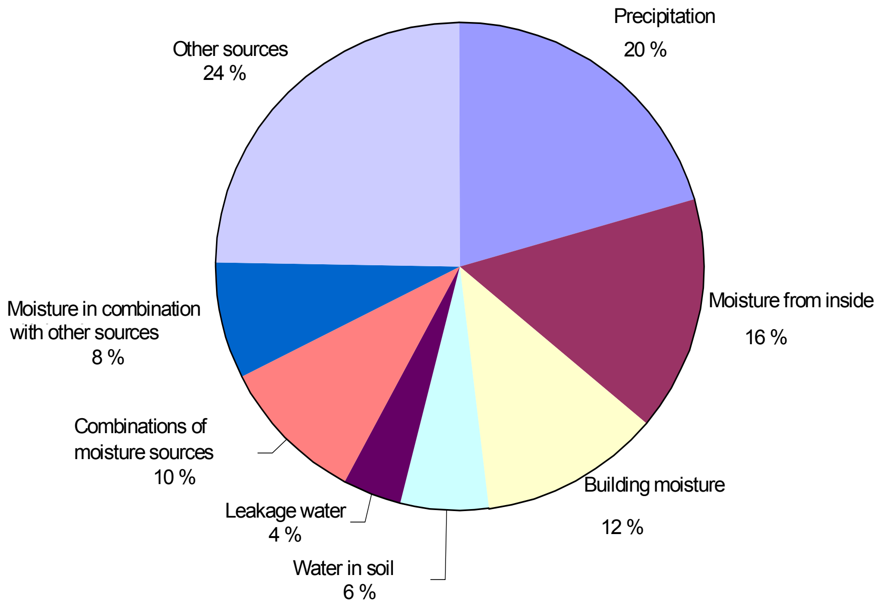

Figure 3
NBI's project archive: Registrations distributed by source of damage, for buildings with year of construction within the period 1993 – 2002 [Mehus et. al 2004].

2.4 Changes caused by the PBA amendment

An assessment of the results of this evaluation shows that the number of building damage is most likely reduced as a result of the PBA amendment. The amendment is therefore also likely to have caused a positive change in building quality, measured by the number of process related building damages. However, uncertainties and limitations in the sources of data make it impossible to quantify the relative reduction.

The analyses made in this project are designed for the possibility of repetition. The tools, methods and basis of data developed can be repeated, improved and expanded. The database for registered building damage in the time period of which the guarantee applies can be developed into a tool for monitoring of changes in the level of building damage in Norway. The analysis of data from the NBI Project Archive also illustrates the potential for this database to be utilized in further in depth investigations of building damage related issues. A further development with registration of new building projects together with extended analyses of the historical material would provide important data and results for the Norwegian building and construction industry. In addition, this would also generate important data for the determination of reference service life and modifying factors for the Norwegian building stock.

3 EVALUATING THE DESIGN PROCESS

3.1 Introduction

In the second project [Stenstad and Rolstad 2004] the evaluation is based on the intentions described in the preliminary investigation done prior to amending the PBA, the amended PBA, and also new regulations and guidelines most of which were published in 1997. The new regulations represented a clear break with the previous trend of de-regulation and put the emphasis on quality of buildings.

The most important measures introduced in 1997 supposed to increase the quality of the design process included:

- New regulations and requirements with respect to accountability and responsibility between the parties involved in the building process (i.e. architects, engineers, contractors etc.) and local authorities
- New requirements for documentation when applying for building permit, and verification of compliance with performance based codes
- New system with improved requirements for supervision and control of building works
- New and improved regulations providing a separation of the responsibilities of the local authorities on one hand and the client on the other hand

Accountability, competence/qualifications and control activities are factors influencing the quality and service life of building components and assemblies.

3.2 Accountability

The evaluation shows that the measures introduced in 1997 only partly have been followed up by the designers. In many building projects it is still not apparent who is accountable for fulfilling the requirements of the building regulations. If the accountability is clear, very often the actual design work has been carried through by another company. This means that one company is accountable and another company has the competence and has done the work. This is not according to the intension of the new regulations. Besides, in many cases there is no written contract between the company accountable and the company carrying through the work.

3.3 Competence

The aim of the approval system introduced as part of the amended PBA was to ensure adequate qualifications of the companies responsible for design and construction. However, the study of actual building projects shows that it may be questioned if this really is the case. In many complex projects adequate competence seems not to be used. An example is the design process of 14 school projects. In these projects, despite massive problems concerning indoor climate in schools in the recent years, special competence in this field has not been provided in the project.

3.4 Detailed engineering

The study does not provide obvious conclusions concerning the extent and quality of detailed engineering. However, the investigation indicates that the changes of 1997 to a certain extent have led to improved basis (drawings and specifications) for execution. The extent of detailed engineering in the projects examined in the study is varying quite a lot. However, the extent seems to be lower in large, complex projects than in small projects (single family houses). This is a paradox because the needs of detailed engineering normally will be higher in large, complex projects because in these projects the use of "standardised" solutions is less frequent. The study also shows that architectural or aesthetic considerations very often are dominating in choosing solutions. This means that sometimes a solution is chosen although the designer knows it will imply increased damage risk.

3.5 Control

The changes in the building regulations in 1997 have to a great extent led to the implementation of quality systems in the designing/engineering (and construction) companies. Furthermore, this has led to increases focus, and more time spent, on control. So, the control seems to be improved due to the reform. However, the control very often is limited to "self control" – which means that the one doing a piece of work is controlling himself/herself. Besides, some of the extra time spent is used on paperwork. Despite this, the investigation is indicating an increase in the number of damage detected and corrected during the design phase. The number of complaints towards the engineering (and construction) companies is reduced. The extent of independent (third party) control is low. In the projects studied in the evaluation, third party control has been carried through only in the field of fire safety engineering.

4 CONCLUSIONS

The results of the two projects presented in this paper show that the number of process related building damages in Norwegian building projects in fact have been reduced, and new quality control requirements in the amended PBA are assumed to be the main reason for the reduction observed. Thus, in evaluating the influence of the design process on service life of building components and buildings, the quality control system of the companies accountable for the design is of great importance.

However, even if the amended Norwegian PBA appears to be a move towards improved quality of construction, defects, flaws, and premature damages are still flourishing in new construction, incurring huge costs to the building and construction industry, the building owners and society itself. In many building projects it is still not apparent who (i.e. which designer/engineer/architect) is accountable for the vital parts of the design. It may also be questioned if the qualifications of the design/engineering companies are sufficient.

The analysis of data from the archive at NBI demonstrates lack of basic understanding and knowledge about critical design and construction requirements and recommendations. Furthermore, design of details crucial to durability and service life of buildings is often omitted, i.e. the details are left to the contractor to solve on site.

5 ACKNOWLEDGEMENTS

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6 REFERENCES

Bjørkmann, Wiran R. (1999) Experience in introducing performance based fire regulations, Interflam 1999, Edinburgh.

Ingvaldsen, T. (1994) *Byggskadeomfanget i Norge*. NBI Prosjektrapport 163, Norwegian Building Research Institute, Oslo.

Ingvaldsen, T. (2001) *Skader på bygg. Grunnlag for systematisk måling*. NBI Prosjektrapport 308, Norwegian Building Research Institute, Oslo.

Lisø, K.R, Kvande, T. and Thue, J.V. (2004). High-performance weather protective flashings, *Building Research & Information* (accepted).

Mehus, J., Rolstad, A.N., Nordvik, V. and Stenstad, V. (2004) Endring i byggekvalitet – Kvantitativ registrering av byggskadeomfang, NBI Prosjektrapport 379, Norwegian Building Research Institute, Oslo.

Stenstad, V. and Rolstad, Anna N; (2004) Klarere ansvarsforhold og nye kontrollprosedyrer - effekt i forhold til feil og mangler ved prosjektering, NBI Prosjektrapport 375 og 376, Norwegian Building Research Institute, Oslo.

Standards and Tools for the Guarantee of the Reliability of the Intermediate Product in Building

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ABSTRACT

The purpose of the research is to define methodologies and instruments in order to reduce the technical risk in design and construction and to increase the borders of reliability in management of technical documentation about the different phases of the building process. In particular, here, a study for the reliability of building intermediate products is proposed, relatively to the reciprocal interaction relation between single materials, in the different phases of design, of realization and building management.
The problem is put in that more general of reliability of executive design, because on this design level is concentrated the request of technical appropriateness, of conformity and sustainability of construction solutions. It is at this level that the requirement of validations of the project before the execution, is refered, since in it the conditions of major risks are concentrated, in relation to the possibility of realizing a predeterminate global quality of the structure. At the same time we have the aim of introducing these verification in design praxis, in the most efficient way.
The problem is acting so that evalutative protocols of durability be part of the elements usually considered necessary for a "complete technic and economic definition of the object of contract".
This has been translated in the definition of an assessment and validation instrument by the application, in experimental way of the "Factor Method".
The work has been developed through out research steps finalized to define, for technological units and critical knots, the quality-quantitative and problematic aspects, suggested by Factor Method. The FM, according to the ISO 15686, is structured in seven factors, to each of those, in relation to the specific use conditions, is given a weight. On the basis of the aims expressed the activity of improvement has concerned in particular the Factor A - Quality of Components for what a first level of structure of basis knowledge of the behaviour of single materials used of technological solutions, has been constructed.
To this archive a software (BenchMat) has been applied, it manages different data-bases and develops, for each material and technological solutions, various levels of verification, to which it is possible to connect a scale of values to be used in the application of the Factor Method. On this first result, in the prototype version, a program of application and experimentation is going to be initiated.

KEYWORDS
Reliability, Service Life, Factor Method.

1 INTRODUCTION

Over the last few decades, what has emerged within the building field is a growing awareness of the decisive role which is to be played by temporal variable (in order to fix standards for the architectonic project). It is evident consistently with what the most recent positions regarding the matter of sustainability are, the necessity to define a project for the durability of materials and components.
As for what intermediate product is concerned, it has been registered a greater interest in the quality characteristics: take into consideration the importance given to trademarks, even if only for selfcertification and under the spur of mere commercial motivations. Besides a large set of product rules is available (UNI, CEI, ISO, CEN).
Despite that, from the contents of the different product specification few certain data emerge on the behaviour of the materials and on their interactions.
The problem of compatibility among products of different nature and origin is often unsolved and so there is the necessity to know the reliability, durability and degradation of the materials and components, from the starting phase. A very serious matter, considering that the technical circuit board of the materials remains the central element of the conventions and of the contractual relationship between the production operators and those of the design, realization and control.
It is really the crux of the matter, highlighted from the difficulties the answer of certified quality expressed, in Italy, from the law 109/'94 is dealed with.

2 BACKGROUND

References for this work are:
- The CIB activity that, in teamwork with international research bodies, organizes periodically meetings on the *Durability of Building Materials and Components* theme.
- The work of the several CIB committees which inquire on the problem since a long time. Particularly the W080 committee, which collaborates with the Technical Committe (TC) of the International Association for Building Materials and Structures (RILEM). The coordinate group CIB W80/RILEM 175- SLM *Service Life Methodologies*, has pubblished in several documents recommendations for the Service Life evaluation of components and building systems.
- The standard regulations and the recommendation from ISO.
- The research procedures of the technical body TC 59 Building Construction, which inquires the theme of Service Life in building, and particularly of one of its sub-committe (SC14 "Design life"), whose work has converged in the Series ISO 15686 *Buildings and constructed assets – Service life planning regulating the prevision and the planning of Service Life*.
- The state of the art on application and experimentation of Factor Method, which was pubblished in February 2002, from Task Group Performance Based Methods for Service Life Prediction of the committee CIB W80/ RILEM 175- SLM.
- The English rule BS 7543, which configurates as a general policy law about the hard wearing over time of the building product, supporting the description of the specification of durability, its estimate and the project of buildings and of their parts, subjected to certain environmental conditions and to the consequent chemical deterioration of the materials they are made of.
- The European Directive about building materials (89/106), in which are labeled six primary requirements that must be guaranted during the service life of a building.
- The explanatory documents of the CEE 89/106, whose main target is to define the relations with the mandate the Committee gave to the standardization bodies to establish the guidelines of work for the EOTA.
- The four guidelines turned out from the EOTA in 1999, in which are described the data basis to evaluate the service life of a product connected to his durability.
- *Guidance Papers E* and *F* published from the European Community, one of which set the specific aim of putting the durability theme in connection with the CEE 89/ 106.

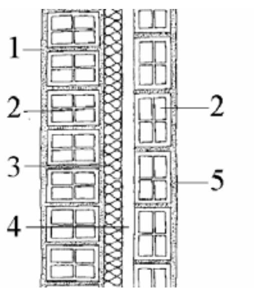
3 AIMS

In the last years knowledges about the performance of building materials during their Service Life have increased remarkably, and the progress regarding the definition of working methods for a reliable prevision of ESL have been remarkable. Despite that, much has still to be done in the direction to improve and to implement these methods so that they can be applied in a concrete way. That represents the challenge and the commitment from research for years to come.
In this context, the research group from Reggio Calabria has produced a study for the guarantee of the reliability of the intermediate product in building, relating to the mutual relationship of interaction among every single material, in the phases of design, realization and managment of the building.
The problem is situated in the more general one of the reliability of executive design, because it is on this level of design that the request of technical appropriateness is concentrated, in the conformity and sustainability of building solutions. It is at this level that validation of the project before its execution referes to, since the conditions of major risk are concentrated in it, in relation to the possibility of realizing a predeterminate global quality of the building.
The problem is acting so that the evalutative protocols of durability become part of the elements usually considered necessary for a complete tecnical economic definition of the object of the contract: from the requirement of acceptability of materials and components, to specification of performance with the relative proof test methods, to the documents that form the plan of maintenance and that of quality. All that with the operative aim to activate mechanisms of information and selfcontrol in alternative to the once usually used in the managment of tecnical information and in its translation in the form of explanation for the operatores. This has allowed the definition of an assessment and validation instrument propaedeutic to the application of the “Factor Method” in an experimental way.
More specifically the aim as been reached by implementing and finalizing operative instruments (the software BenchMat), already produced in their prototypic form by the researchers of the work group at the Laboratory of Materials for Architecture (LA.M.A) working inside the DASTEC at the *Mediterranea* University of Reggio Calabria.

4 THE RESEARCH FIELD

The research, overcoming the traditional principles of the evaluation of the durability of simple technical elements, has inquired more complex building systems: flat roofs and walls. In this phase the necessary technical knowledges have been taken from the literature, starting from the construction of a classification of provable “critical connectiones” of the project.
The most recurrent localizations around, characterized by the arise of significant signs of damage have been located, defining a possible statistic structured on the basis of the three main classes of identified *critical connectiones*:

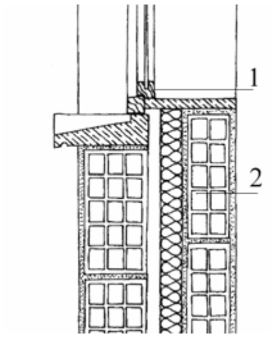
- 1st level *critical connection*. Malfunction or performances collapse affecting one or more Functional Layers of a Technical Element (Fig. 1).



1. Cladding layer
2. Bearing layer
3. Thermal insulating layer
4. Air space
5. Fining coat

Figure 1 – Example of critical connection of 1st level: individuation of the Functional Layers.

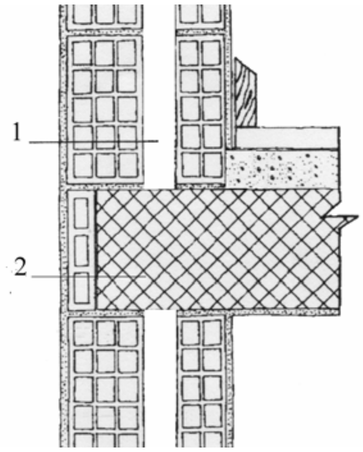
- 2nd level *critical connection.* Malfunction or performances collapse affecting Technical Elements which are relating to each other and belonging to the same Technical Elements Class (Fig. 2)



1. External Transparent Vertical Wall Elements
2. External Opaque Vertical Wall Elements

Figure 2 – Example of critical connection of 2nd level: individuation of the Technical Elements belonging to the same Technical Elements Class.

- 3rd level critical connection. Malfunction or performances collapse affecting Technical Elements which are relating to each other but belonging to different Classes of Technical Elements, though more or less belonging to the same Technological Unit. (Fig. 3)



1. External Opaque Vertical Wall Elements
2. Intermediate Floor

Figure 3 – Example of critical connection of 3th level: individuation of the Technical Elements belonging to different Technical Elements Class.

Such a hypothesis is confirmed by considering how a breakdown can be determined by states of crisis concerning not merely the Technical Elements of each Technological Units but possibly at the same time the points of correlation between different Technological Units. All this, taking into consideration the specificity of the matter and its context, so as to understand the synergetic action of external agents, the materials change and degradation, the way in which such degrading affects the technological system. Such a process, proceeds from the interaction of many distinguished elements coinciding with the seven factors of the Factorial Method.

5 REALIZING AN INFORMATIC MODEL

The project has progressed through research aiming to define the qualitative, quantitative and problematic aspects suggested by the Factor Method for each Technological Unit and for each Critic Connection. The investigation was mainly concerned with the Factor A – Quality of Components, for which it was realised a first structuring level of basis knowledges about the behaviour of the materials employed in the realization walls and flat roofs. One of the results is a " products archive" made of data provided by the producers themselves.
The materials taken into consideration are so identified: commercial identification, specification item, certifications, technical characteristics and a 1st level evaluation of the item suitability for the different technical solutions of the Technological Unit which is under consideration. The archive has been associated with a software (BenchMat) managing the various data-bases and developing different

assessment levels for each material and Technological Solution. The outcome of such assessment levels can be related to a value scale to be used when employing the Factor Method.

The software components are:

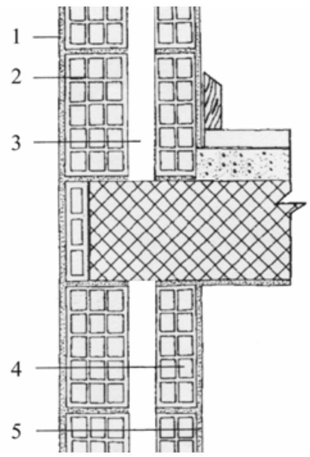
- Management criteria for the acquisition of the technical data provided by the producers in order to equalize the products characteristics and performances, so as to make it possible to evaluate them and to compare such evaluation outcomes.
- Ranking products, materials, mill shapes and components according to their physical and mechanical characteristics.
- Plain and composite materials evaluating procedure, according to the cumulative performance levels and by defining the merit indices related to the 20 “relevant” characteristics with which each performance is to be compared. Precisely: 5 mechanical characteristics; 5 physical characteristics, 5 technical characteristics and 5 technological characteristics.
- Realising a case record of Technical Solutions which are frequent within the investigated Technological Units (walls and flat roofs).
- Converting the evaluations into weights to be associated with the Factor A. The operativeness of the whole program is based on the main Menu which is employed to manage different tasks and to create comparative diagrams between the different materials and or components.

6 FACTOR A. APPLICATION OF THE SOFTWARE MODEL TO A CASE STUDY

In order to apply our evaluation model:

- Description of the case study: the case study includes the analysis of the project for the realization of a residential building to be placed in Reggio Calabria. The environmental conditions of the area are characterised by an high seismic level, a medium level of atmospheric pollution and a high level of noise pollution. The area chosen for the project is a developing one, characterised by 3 or 4 floors buildings and it set in the southern part of the town. The project consists of a 5 floors building made of reinforced concrete according to the traditional technologies. As for its planimetric configuration and locating, the building has south-west exposure. It is characterized by wide glazing panels shielded all around with deep juts.

- A 3rd level critical connection example: the following example represents the technical solution adopted for the connection between the , intermediate floor and the opaque non load-bearing wall with thermo-insulated cavity (Fig. 4)



1. Cladding layer: Outside cement-lime plaster (mm.20)
2. Bearing layer: Hollow clay block (mm.120)
3. Air space
4. Bearing layer: Hollow clay block (mm.80)
5. Fining coat: Inside cement-lime plaster (mm.15)

Figure 4 - Individuation of Functional Layers

- Materials the connection is made up with: the materials for the project technical solution, included in the products archive, have been evaluated according to the Benchmat evaluative model.
- Evaluation: in the specific case, the application of the BenchMat software allows to evaluate the project Technical Solution according to the related background and to the materials to be employed.
- Association with a weight: this phase provides for the assessment for the attribution of weights to be employed during the application of the Factor Method.

7 SUITABILITY OF FACTOR METHOD TO THE CASE STUDY

The experimentation relating to the Factor Method suitability, in an initial phase, concerned the Factor A. Afterwards, specific investigations aimed to constitute a body of knowledges for the other factors, to be employed within the ESL estimating process. Precisely:

Factor B – Design Level

The Standardized assessment procedures which have been used for many years, have been introduced in routine relating to three classes of requirements: Control, Safety and Protection. There have been produced synthetic schedules that give definition, description, evaluation criteria and the normative references. The evaluation is expressed basing on the results of the several assessments.

Factor C – Work execution Level

The definition of the range of values about Factor C follows the assessment of the presence/absence of certifications inside a system of quality. The considered variable are: qualification of the process and of the operators (ISO 9000), making responsible and control during execution (presence/absence of Quality Plan) and at least, real reaching of the performances settled in the design phase.

Factor D – Indoor Environment

The possibility to determine methods, facilities and instruments to check in a methodical way if, and to what degree, the qualitative levels considered optimal correspond to real performance supplied from spaces when they are used, is being studied. The aim is to realize check – lists on the project which express a sequence of minimum performances which the building must have. The identified microclimatic factors are: control of ventilation, of relative humidity, of temperature. The fulfilment of such performances affects the weight to associate with Factor D.

Factor E – Outdoor Environment

The action of the aggressive agents always happens in a different combined way. As a matter of fact deterioration does not depend on a single action, but on a synergy of aggressive actions.

Starting from the background on the subject, the evaluation takes place on the basis of the assessed sensitivity of materials and components either to various disturbance factors or to aggressive agents on the one hand, on the basis of presence/absence of a determinate inconvenience factor in the specified context of the intervention on the other hand. The suggested evaluation method BenchMat, sets out, with the exception of very specific cases, an application of the value "1" to this factor as the context scenery has already been evaluated with the procedure used for Factor A.

Factor F – In – use Condition

Defining the range of the values relative to this Factor depends on a careful analysis of the implications connected with the use condition and with flexibility. More specifically the located variabilities are: the use as regards the use destination, the qualification of the users as to the use capacity, the use flexibility. Conditions functional to the maintenances during Service Life.

Factor G – Maintenance Level

The location of the value range of Factor G depends on a well – definite planning of maintenance operations, on the qualitative levels, on the feasibility evaluations, on the technical compatibilities of the estimated operations and on the happening of unexpected conditions regarding the planned maintenance of the building. Variabilities interfaced with the national issues linked with the application of the normative of the actuation of the general policy law referred to public works (DPR 554/99), which, as it is known, provides a specific document directed to the managment activities.

RSL – Reference Service Life

To define RSL, we have proceeded, getting data from literature and experimental activities, building up a data–base of theorical durability organized for the classes of Technical Elements taken into consideration by the study work (walls and flat roofs). Data bases to be used in the experimentation for the defining of RSL to be inserted in the Factor Method equation.

8. CONCLUSION

The aim of this first phase of the research has been to associate some weights to Factor A.

Such weights are expressed in function of the general knowledge in the field of inquiry and application (walls and flat roofs) structured in the course of the research and implemented in function of the application to other Factors. More specifically, as it is known, for the progressive thinning of the weights to be attributed to the single factors (modifying factors) it will have to be taken into consideration the implementation of the knowledge regarding the effects of the environmental actions on the different materials and also refer to the collection of the data relative to the consequences of an inefficient executive and maintenance level. At the same time, as pointed out by ISO 15686, there is the necessity of involvment and collaboration with the world of production, which we have already activated, in order that it gives, wherever possible, some information that can be expressed through a quantity translatable it a precise weight for that given factor. It follows the importance to manage in a correct way both the gathering data, and their translation into numerical figures (weight).
It is just towards this direction that we inted to give with this work a contribution to experimentation in general of Factor Method and in particular to the refining of the knowledges referred to Factor A.

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REFERENCES

CEE 89/106, (1989), *Construction Products Directive*, European Commission, Bruxelles
BS 7543, (1992), *Guide to Durability of Buildings and Building Elements, Products and Components.* British Standards Institution, Londra
Interpretative Documents of Council Directive 89/106/EEC, (1994), *Official Journal of the European Communities*, C62. Vol.37
European Organization for Technical Approvals (EOTA) (1999) *Assumption of working life of constructional products in guidelines for European Technical Approval, European Technical Approvals and harmonized standards.* EOTA Guidance Document 002, Bruxelles
European Organization for Technical Approvals (EOTA) (1999) *Assessment of working life of products.* EOTA Guidance Document 003, Bruxelles
European Organization for Technical Approvals (EOTA) (1999) *The provision of data for assessments leading to ETA.* EOTA Guidance Document 004, Bruxelles
European Organization for Technical Approvals (EOTA), (1999), *Reference material for ETAGs Comprehension documents for ETAGs Updating process of ETAGs Guidance.* EOTA Guidance Document 005, Bruxelles
European Commission, DG III, (1999) *Levels and classes in the Construction Products Directive. Concerning the Construction Products Directive, CEE 89/106.* Guidance Paper E, Bruxelles
European Commission, DG III, (1999) *Durability and the Construction Products Directive.* Guidance Paper F, Bruxelles
ISO 15686 - 1 (2000) Buildings and constructed assets - Service life planning - General principles
ISO 15686 - 2 (2001) Buildings and constructed assets - Service life planning - Service life prediction procedures
Hovde, P. J. (2002) *Factor Methods for service life prediction. A state of the art.* Final draft report

Product Declarations with respect to Durability – A progress report

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ABSTRACT
The scope of ISO/TC59/SC14 "Design Life of Buildings" is to produce standards on the steps to be taken at various stages of the building cycle to ensure that the resulting constructed facility, will last for its intended life without incurring large unexpected expenditures of money or resources. The standards for design life of buildings also identify a guiding concept on durability of building products of help in implementing the European Construction Products Directive, CPD. Four parts of the standard series, ISO 15686 Buildings and Constructed Assets – Service Life Planning, have been published. Another 4 parts of the 15686 series are on the way to be approved, one being the Part 8 "Reference service life and service life estimation". Of particular importance is the concept of Reference Service Life (RSL; the expected service life in a well-defined set of in-use conditions), the procedures for service life prediction (Part 2) and the Factor Method (FM) for estimation of service life in specific projects (Part 1 and 8). The FM is used to modify an RSL to obtain an estimated service life (ESL) of the components of a design object, while considering the difference between the project-specific and the reference in-use conditions. This methodology receives much interest from the international R&D community. A challenge is to establish databases on RSL and factor distributions. This presupposes the involvement of the industry and other stakeholders in the work. RSL is also essential in providing environmental information on whole life cycle of building products. According to ISO/DIS 21930 Buildings and Constructed Assets – Sustainability in Building Construction – Environmental Declarations of Building Products, it is necessary to have RSL data of the product to provide scenarios for environmental impacts of the use stage of the product. The Part 9 "Service life declaration in Product standards" is developed in parallel with work in CEN to establish guidance documents for inclusion of durability declarations in product standards. For innovative products EOTA (European Organisation for Technical Approvals) are issuing European Technical Approvals, where the durability evaluation is performed according to EOTA guidance developed on the basis of the service life prediction concepts as expressed in ISO 15686-2.

With the rapid development of IFC based standards for digital object oriented models of building products there is a huge need for property sets, such as durability and service life data, which can be linked directly to the building elements.

KEYWORDS

Service Life, Product Standards, Durability Declarations

1 INTRODUCTION

In 1993, the standardisation work in the field of service life planning started when ISO/TC59/SC14/WG9, Design Life of Buildings, was launched at a meeting in Atlanta. The purpose of the activity is to document steps to be taken at various stages of the building cycle to ensure that the resulting building, or other constructed facility, will last for its intended life, the design life, without incurring large unexpected expenditures. The group was initiated on a significant European initiative based on the need of supporting standards when implementing the European Construction Products Directive, CPD [CEC 1988]. The establishment was based on the Vienna Agreement on co-operation between CEN and ISO. Some years later the working group was elevated to ISO/TC59/SC14. The background to the standardisation work and specifically its relation to CPD but also its relevance to similar motives and needs internationally have been thoroughly reported by Sjöström et al [2002].

The scope of this article is to report and discuss the further progress of the standardisation, with a specific focus on product standards and the principles for inclusion of durability and service life declarations in these. The significant importance of service life data in all aspects of sustainable construction, e.g. environmental product declarations, is addressed, as is the opportunities offered by the development of digital object oriented models of building products and the IFC based standards.

2 ISO 15686 STATE-OF-WORK

The four parts ISO 15686-1 “General principles” [ISO 2000], ISO 15686-2 “Service life prediction principles” [ISO 2001], ISO 15686-3 “Performance audits and review” [ISO 2002] and ISO “Procedure for considering environmental impacts” [ISO 2004a] have already been approved as full ISO standards.

Further parts in the series are under way.

Table 1: Further parts on ISO 15686.

Parts on ISO 15686 under way
Part 4: Data requirements
Part 5: Whole life costing
Part 7: Performance evaluation for feedback of service life data from practice
Part 8: Reference service life and service life estimation
Part 9: Guide on the inclusion of requirements of service life assessment and service life declarations in product standards
Part 10: Description of the data required in estimating service life
ISO 21933-1 Performance standards in buildings – Levels of functional requirements and levels of serviceability – Part 1: Principles

Part 4 is decided to become a Technical Report. Part 5 is the subject of a current DIS voting, while part 7 and 8 already have been approved as DIS. The development of part 9 and 10 has recently started. The further development of ISO 21933-1 has been transferred to ISO/TC59/SC14 due to the closure of ISO/TC59/SC3. In addition, a revision of part 1 “General principles” has commenced.

In Europe, an approval process by CEN of ISO 15686 as EN standards has started, subsequently making them mandatory within EU. By no doubt, this will generate demands on both further RTD and subsequent standardisation, particularly at the product level. For this purpose, CEN has established CEN/TG Durability, having the task to produce a Short-term Guidance document and a Long-term Guidance document on the inclusion of working life in product standards. The European Organisation for Technical Approvals – EOTA has in principle the readiness to issue CE labelling of products. The significance of this would be that the product meets the requirements of the CPD, which can be demonstrated based either on standards or technical approvals. As a consequence, the Service Life Methodology has also formed the basis of an EOTA document for the assessment of working life of products [EOTA 1999]. This issue of implementation of the CPD via the ISO 15686 standards is also the subject of a recent paper by Sjöström et al. [2002].

3 THE FACTOR METHOD AND THE REFERENCE SERVICE LIFE CONCEPT

Service life planning of a design object involves the estimation of service life of its components. However, even if there are certain service life data of a component available from various sources, in general such reference service life data cannot be used satisfactorily as found. This is because it is very unlikely that the in-use conditions specific to the design object will be the same as the reference in-use conditions, i.e., the in-use conditions under which the reference service life data are valid.

Accordingly, in order to achieve an appropriate estimated service life, there is a need of modifying a reference service life available by taking the differences between the object-specific in-use conditions and the reference in-use conditions into account. The Factor method [ISO 2000 and ISO 2004b] provides a systematic way of carrying out such a modification.

The Factor method can be applied at different levels of sophistication, from working as a simple check-list to complex calculations. The level should be selected in dependence on conditions such as the actual purpose of the estimation, type and quality of available data and models, skill level and type of expertise of the user(s) making the estimation, and resources and time available for the calculation.

One straightforward way of applying the Factor method is to multiply a reference service life *RSL* by a number of modifying factors *A* to *G*, each of which reflecting a difference between the object-specific and reference in-use conditions within a particular factor class, see Table 1, thus obtaining an estimated service life *ESL*:

$$ESL = RSL \times A \times B \times C \times D \times E \times F \times G. \quad \textbf{(1)}$$

Table 2: Factor classes.

Factor class	Designation
A	inherent performance level
B	design level
C	work execution level
D	indoor environment
E	outdoor environment
F	usage conditions
G	maintenance level

For the application of the Factor method, except from knowledge of *RSL* itself, of course also information of the reference in-use conditions as well as the object-specific in-use conditions must be available in order to allow estimation of the modification. Thus, the reference in-use conditions should be provided together with the *RSL*, while the object-specific in-use conditions are determined from the knowledge of the design object and site.

In ISO [2004b], it is described how an *RSL* and appurtenant reference in-use conditions together with additional required or useful information concerning the *RSL* should be formatted into an RSL data record.

4 SERVICE LIFE DECLARARTIONS IN PRODUCT STANDARDS

There is a need of standard guidance on the inclusion of requirements in product standards of how service life assessment and service life data should be derived and declared to enable compliance with the existing ISO 15686 standards, specifically parts 1, 2 and 8 [ISO 2000, ISO 2001, ISO 2004b].

Hence, a new work item in the ISO 15686 series is part 9: " Service life declarations in product standards". This standard is intended to give guidance to standard writers how to take into account and include requirements of declarations of service life assessment and service life data in product standards. This ISO work item is closely interrelated to the scope and objectives of a CEN Task Force on Durability, which are to give guidance to CEN technical committees in their work to establish harmonised European product standards. The CEN TF has produced a short term guide aimed at supporting those CEN TC´s still working on the first generation harmonised product standards. This guide give directions on how to meet the durability requirements of CPD, and is included in an ongoing revision of the CPD Guidance Paper F "Durability and the CPD" [CEC 2001]. The second task of the CEN TF is to produce a long term guide for second generation of harmonised product standards considering the concepts established in ISO 15686, and subsequently this task goes hand in hand with the work by ISO.

It is of crucial importance to form a document that the manufacturers will consider as a useful tool when marketing their products and not as an obstacle. That is, within a standardised frame and without compromising with the credibility, as large a number of freedoms as possible to evaluate and declare service life data should be allowed. Of course, subsequently the manufactures themselves may impose stricter requirements at the product level via their participation in respective product standardisation committees. In other words, this standard should provide a minimum level for the evaluation and declaration of service life data.

It can be foreseen that it would not be feasible to require that service life declarations have to be given for every bolt or nail. What products that could be excepted from declaration provisions are up to the standardisation organisation issuing the product standards to decide. However, it is important to offer every producer the possibility to declare service lives, if so on a voluntary ground, to improve his/her competitiveness.

A declared estimated service life is not to be interpreted as a guarantee given by the manufacturer, but are regarded only as a means for choosing the right products in service life planning according to ISO 15686-1 [ISO 2000].

Preferably, service life data should be obtained in accordance with the provisions of 15686-2 [ISO 2001], that is, by long-term exposure and/or short-term exposure, and subsequent evaluation, or feedback from practice [ISO 2000c].

Nonetheless, other test methods should be allowed. Such tests, however, have eventually to result in explicit statements of the estimated service life. The minimum requirement on a statement could be just a lower limit of the estimated service life. This type of minimum statement could be particularly suitable when a test result or a property just is evaluated against a simple pass/fail requirement, which in some respect has to be based on experience of the service life becoming long enough upon passing, say at least 100 years. Finally, if the concept of "normal" service life is used as the lower limit, "normal" has to be quantified in years!

In every instant, all in-use conditions or measures required to reach the estimated service life should be provided. Particularly, as for outdoor environment an appropriate range of conditions should be assumed, resulting in a range of service lives. A definition of environment classes or use categories may be done.

Provision of data should be made according to ISO 15686-8 [ISO 2004b].

5 REFERENCE SERVICE LIFE IN SUSTAINABLE CONSTRUCTION

Predominantly a client sets the performance requirements for the building in terms of building performance and functionality. The key elements that need to be developed for the successful linkage of performance-based building with service life planning are thoroughly discussed in Trinius, Sjöström [2004]. The basis for the requirements is usually about technical performance. But during its life cycle, from raw material supply of building products to the final disposal of building components, the building has environmental and economic impacts as well as impacts on the health & comfort of the users. These impacts can be analysed in the assessment of environmental performance, economic performance and health & comfort performance of building.

According to the scope of the becoming European framework standard for the assessment of integrated building performance there are three dimensions for sustainability of the building: environmental, economic and health & comfort. Assessment of overall sustainability of buildings can be done, but it is necessary to assess those three dimensions of sustainability at the same time, but separately. It is impossible to evaluate and integrate indicators of environmental, economic and health & comfort performance into one indicator and one value. Nevertheless, it is clear that the assessment of sustainability of buildings must be done with life cycle approach.

5.1 Environmental performance of buildings

There is a very important bond between durability of building products, service life planning and assessment of integrated building performance. Life Cycle Assessment (LCA) is used as a tool for assessing environmental impacts of a building during its whole life cycle, from cradle to grave. It is impossible to assess environmental performance of a building without having scenarios on the use and durability of building products, i.e. information on the service life of building products in the intended conditions of use.

The International draft standard, ISO/DIS 21930 Buildings and Constructed Assets – Sustainability in Building Construction – Environmental Declarations of Building Products, requires that in order to declare environmental information on the whole life cycle or on the use stage and on the required maintenance of the building product, it is mandatory to declare the reference service life of a building product together with the reference in-use conditions. Of course, it is not necessary to declare all the detailed service life information in the Environmental Product Declaration (EPD), because in the EPD it is logical to have a reference to the service life declaration of the building product. The becoming CEN standard on the EPD of the building products is going to be totally in line with the ISO/DIS 21930.

The main purpose of the EPD is to communicate the environmental information and related additional information to the building level for assessment of environmental performance or health & comfort performance. The related additional information can be on the technical properties or e.g. the release of hazardous substances into the indoor air during the use stage. The essential fact is that the manufacturer of the building product is a sole owner of the environmental and technical information of the product. It means that the information in a generic database is not always updated.

5.2 Economic performance of buildings

In order to be consistent in the assessment of the economic dimension of sustainability of buildings, the term “life cycle” must refer to the whole life cycle of the building, from cradle to grave. Again, as is the case with environmental assessment, it is impossible to assess life cycle costs of a building without having information on the service life of building products in the intended conditions of use.

6 IFC BASED STANDARDS AND DURABILITY DATA

With the rapid development of IFC based standards for digital object oriented models of building products there is a huge need for property sets, such as durability and service life data, which can be linked directly to the building elements. On the other hand the IFC development opens up and facilitates for a much quicker than anticipated implementation of ISO 15686.

The ISO standard ISO16739, known mostly as Industrial Foundation Classes (IFC) has been developed by the IAI (International Alliance for Interoperability) for the building sector based on the generic Standard for Exhange of Product model data (STEP), ISO 10303. IAI is an alliance of organizations within the construction and facilities management industries dedicated to improving processes within the industry through defining the use and sharing of information. Organizations within the alliance include architects, engineers, contractors, building owners, facility managers, manufacturers, software vendors, information providers, government agencies, research laboratories, universities and more.

EXPRESS is the data modelling language allowing for implementation of the IFC and STEP standard. International Framework for Dictionaries (IFD), or the ISO 1200-3/PAS, extends IFC to include unique and specific definitions of concepts used in the building industry. A simplified analogy would be to describe IFC as a communication method, just as speaking is a communication method commonly chosen in certain situations, as opposed to sign language used at other times. IFD is then the language used while speaking. IFD provides the dictionary, the definitions of concepts, the common understanding, necessary for the communication to flow smoothly. Work is going on in many countries to develop national dictionaries to ease the trans - national communication flow (Bell et al, 2004).

The IFC standard has already reached an implementation level where it now provides real value for users in projects. The US General Services Administration requires drawings and plans to be delivered on IFC format from 2006 on. IFC allows, among other things, two or more unrelated applications in the building process to exchange information about the underlying model the project wishes to realize. It thus supports the move away from drawings and onto real 3D models, allowing for a continously enrichment of the model with relevant information throughout the building process.

The IAI developers has already defined and included concepts and properties related to a whole range of facility management aspects, such as life cycle costing, environmental impacts and service life, relying and referring to international standards, such as ISO 15686.

Within the IAI framework several pilot and –development projects are paving the way for implementation of IFC/IFD based models within the sector. One such project is the development of IFC based links to geographical information (IFG) [Wix et al, 2005]. This will be essential for example for facilitating a seamless zoning and building plan permit. It will also be essential in linking the appropriate exposure data to the building and the algoritms of the service life. In Norway Bjørkhaug et al [2005] has developed a pilot that describes the implementation of the ISO 15686 SLP process on an IFC/IFD/IFG platform by link to existing knowledge bases at Byggforsk and elsewhere [Bjørkhaug et al, 2005].

6 DISCUSSION AND CONCLUDING REMARKS

The ISO 15686 series of standards on design life of buildings and constructed assets are today implemented as national standards in a number countries. The concepts for service life planning presented in the standards are being met with interest, but there is still far from a situation where the standards are commonly used in practice. Education and information campaignes are being organised in certain countries, e.g., Sweden, and the standards with design concepts are also in focus by European network projects.

This article has a prime focus on the implementation of the ISO 15686 concepts in product standards, as a means to meet the requirement on, e.g., harmonised European product standards, to declare durability of the product in its intended use situation. The concept of Reference Service Life of products is described in ISO 15686 Part 8. The relevance of the RSL concept also in planning for sustainability and in, e.g., EPD is discussed. It is anticipated the RSLs predominantly will be offered by the materials and products producers. The reasons for this being at least twofold: firstly, the industry knows their products and the RSLs are – even if as far as possible objective data in accordance with the standard requirements – also their arguments on the market, and secondly, only in a few cases there will be resources available for authorities or R&D-organisations to provide RSL-data.

The rapid development of IFC based standards for digital object oriented models of building products fuels the need for property sets, such as durability and service life data, which can be linked directly to the building elements, and on the other hand the development opens up and facilitates for a much quicker than anticipated implementation of ISO 15686.

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8 REFERENCES

Ch. Sjöström, P. Jernberg, P. Caluwaerts , S. Kelly , S. Haagenrud, J.L. Chevalier (2002) Implementation of the European Construction Products Directive via the ISO 15686 standards, *9th Conference of Durability of Building Materials and Components, Brisbane 2002*

The Council of the European Communities 1988, *Council directive of 21 December 1988 on the approximation of laws, regulations and administrative provisions of the Member States relating to construction products*, The Council of the European Communities, Brussels. 89/106/EEC.

The European Commission 2001, EC Guidance Paper F: Durability and the CPD, http://europa.eu.int/comm/enterprise/construction/internal/guidpap/f.htm, update 11/06/2001.

EOTA (European Organisation for Technical approvals) (1999) *Assessments of working life of products.* PT3 Durability (TB97/24/9.3.1)

ISO (International Organisation for Standardisation) (1984) Performance standards in buildings – Principles for their preparation and factors to be considered. Geneve (ISO 6241-1984).

ISO (International Organisation for Standardisation) (1992) Corrosion of metals and alloys– Corrosivity of atmospheres- Classification. Geneve (ISO 9223:1992).

ISO (International Organisation for Standardisation) (2000) *Buildings and constructed assets – Service life planning – Part 1: General principles*. ISO, Geneve. ISO 15686-1.

ISO (International Organisation for Standardisation) (2001) *Buildings and constructed assets – Service life planning – Part 2: Service life prediction principles*. ISO, Geneve. ISO 15686-2.

ISO (International Organisation for Standardisation) (2002) *Buildings and constructed assets – Service life planning – Part 3: Performance audits and review*. ISO, Geneve. ISO 15686-3.

ISO (International Organisation for Standardisation) (2004) *Buildings and constructed assets – Service life planning – Part 6: Procedure for considering environmental impacts*. ISO, Geneve. ISO 15686-6.

ISO (International Organisation for Standardisation) (2004 b) *Buildings and constructed assets – Service life planning – Part 8: Reference service life and service life estimation*. ISO, Geneve. ISO/DIS 15686-8.

ISO (International Organisation for Standardisation) (2004 c) *Buildings and constructed assets – Service life planning – Part 7: Performance evaluation for feedback of service life data from practice.* ISO, Geneve. ISO/DIS 15686-7.

ISO/WD 15686-4 Buildings – Service Life Data Sets – Part 4 – Service Life prediction data Requirements (2002).

W. Trinius, Ch. Sjöström (2004) Service Life Planning and Performance Requirements, in print, Building Research and Information

L. Bjørkhaug, H. Bell; G. Krigsvoll, S. E. Haagenrud (2005) Providing Life Cycle Planning services on IFC/IFD/IFG platform – a practical example. *10th Conference of Durability of Building Materials and Components, Lyon, 2005*

J. Wix, L. Bjørkhaug; Ø. Rooth, J. Sjögren (2005) Industry Foundation Classes – Facilitating a seamless zoning and building plan permission. *10th Conference of Durability of Building Materials and Components, Lyon, 2005*

H. Bell, L. Bjørkhaug, J. Rønning (2004) ICT- platform for Object Oriented Knowledge in the Building and Construction Industry. B4E Conference "Building for a European Future", Maastricht, 14. Oct 2004

Durability Design of External Thermal Insulation Composite Systems with Rendering

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ABSTRACT

The paper deals with a study process aimed at building up a design method of durability of external thermal insulation composite systems with rendering.
Based on the proceeding set by ISO 15686 – service life planning and referring to ETAG 004 (Guideline for European Technical Approval of External Thermal Insulation Composite Systems with rendering), the study is intended to assess the definition phase (user needs, building context, performance requirements and criteria, material characterisation) and the preparation phase (identify possible degradation mechanisms, degradation factors, degradation indicators and suggest ageing tests) in order to achieve an exposure testing programme meant to evaluate the degradation intensity and the service life of component.
A second part of the project will look for the basis of a factorial method, peculiar to ETICS with rendering, avoiding subjectivity in factors assessment. It will split up factors in sub-factors, the weight of which is to be estimated with short-term exposure tests and references to International Standards and literature. The result of the factorial method will be related to the reliability evaluation (according to the proceeding proposed by Politecnico di Milano).

KEYWORDS

Service life estimation, factor method, ETICS, External thermal insulation

1 INTRODUCTION

Since the ISO 15686 was edited, many studies dealing with Service life predictions have been published, as portrayed in the W080 State of the Art Report. Three main methodologies of Service life estimation are possible: stochastic methods (nowadays economically achievable only for concrete), engineering methods and factor methods. Mainly the Factor Method and its features have been evaluated: simple application, subjectivity, not reliability. This study purposes a process aimed at avoiding subjectivity in factors estimations of Factor Method. First it needs to acquire a perfect knowledge of the building component being evaluated (phase 1 of ISO 15686 procedure), then to draw up the agents complex, degradation factors and indicators (phase 2), last to suggest ageing test designed in order to experimentally confirm a Factor Method on demand specifically for ETICS.

2 DURABILITY DESIGN METHOD

In order to rule the durability performance of a building component, it is possible to look up at a design approach according to UNI 10723, which subdivides the phases of the building process in decisional process, distinguished in meta-design (when the objectives of the project are analysed and the requirements and their fulfilment strategies are identified) and design, executive process and management process. In this way, the phase 1 and phase 2 of ISO 15686 could offer a guideline in the meta-design of construction process (concerning in detail Service Life design).

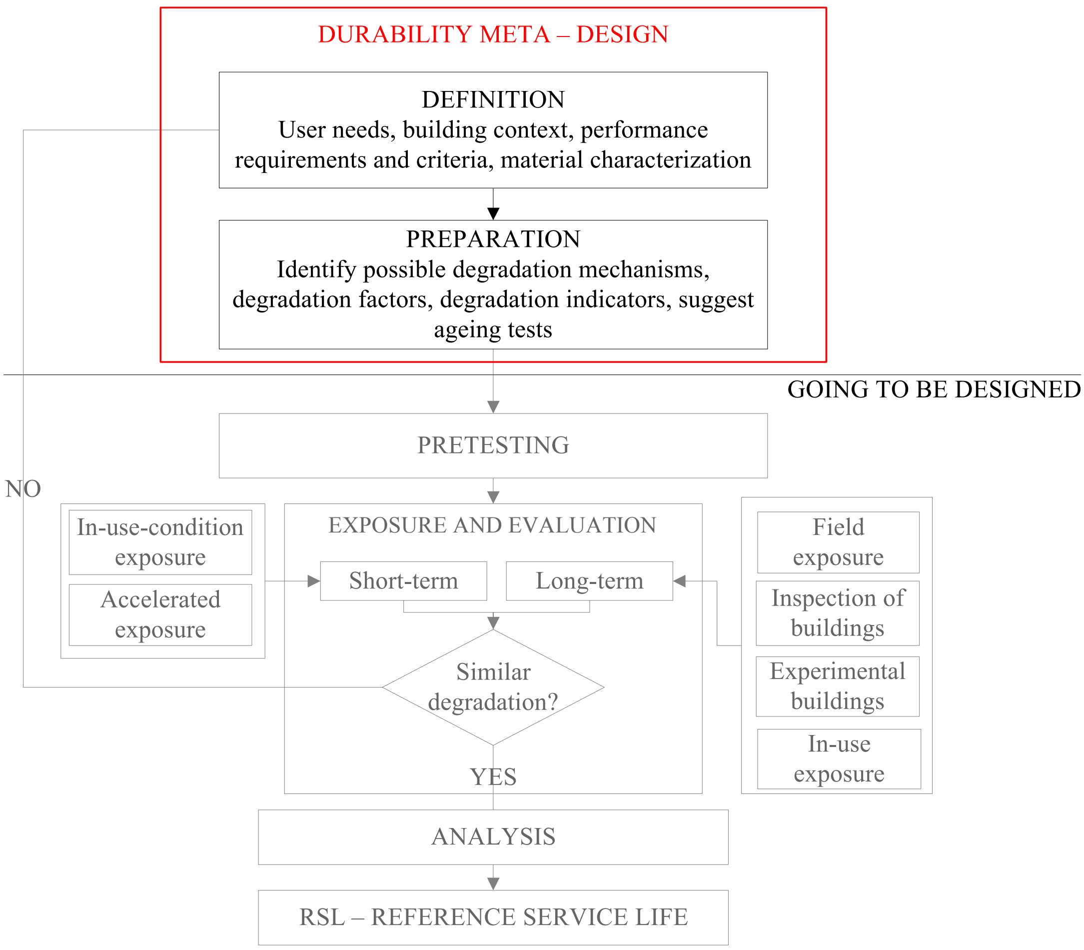

Figure 1. Flow diagram of the ISO 15686 Service Life Estimation Process

2.1 Phase 1: Definition

2.1.1 User needs
References to users' requirements standards: ISO 6242, UNI 8289, UNI 8290-2.
While the approach of ISO 6242 focuses on environmental requirements (thermal, air purity, acoustical, lighting), the UNI 8289 and UNI 8290-2 analyse the technological and performance aspects (e.g.: water tightness, thermal insulation, thaw resistance, etc.).

2.1.2 Building context
It needs to identify the main contexts where ETICS are applied, in order to build up a corresponding ageing test. It possible to subdivide Europe in macro – climatic areas like in the Annex A of EOTA Guidance Document 003 – Assessment of working life of products, Dec 1999.
So even the production context will be identified and it will be possible to configure tests for a main group of systems mainly adopted in a macro – area of Europe (e.g. systems with mineral wool are used in the Scandinavian area, while in France and Italy is preferred polystyrene).

2.1.3 Performance requirements and criteria
It is not reasonable to express the requirement for a building component in thermal transmittance, due to the fact that, in different countries, different laws impose different minimal performances and mainly it is not specified a superior limit on U [W/m^{2}K], but it is required a performance of the whole building. In many studies about thermal behaviour of buildings, the main unit of measurement is the annual requirement of energy per square meter: Q [kWh/m^{2}y], according to EN 832.
Due to this fact, it is possible to build up an artifice: to assess the thermal performance of a building component it can be used a "reference building". This building should be 10 m x 10 m x 20 m (dimensions internally measured) and it should respect the following features expressed in Table 1.

Fixed features of the "Reference building"	*Variable features of the "Reference building"*
dimensions and geometry	building components
number, dimensions and position of the windows	macro-climatic and meso-climatic context
thermal energy providing system	model of use
	duration performance

Table 1. Features (fixed and variable) of the "Reference building" for comparison of thermal performance of building components

It is also possible to link the building component performance to environmental requirements. In detail, e.g., the ETICS performances could be assessed estimating the parameters (and their environmental gradient) which evaluate the users' thermal requirements analysed in ISO 6242-1:
- t_B dry – bulb air temperature;
- t_r mean radiant temperature
- v_a air velocity
- t_o operative temperature
- t_{nw} wet – bulb air temperature

Another performance requirement could be identified in the duration: it should be reasonable to design classes of Working Life for ETICS (not only 25 years indicated by Guidance paper F) or classes of propensity of duration (classes of acceptable risk).

2.1.4 Material characterisation
In order to limit the research programme (as indicated above), it could be considerable a subdivision in macro – classes of ETICS systems (e.g. identified by insulator) for macro – climatic areas of Europe. It is also important that the support must not be an insulator by itself, or it could influence the percentage thermal performance of the external system.

2.2 Phase 2: Preparation

2.2.1 Degradation factors and degradation mechanisms

It has been developed a degradation factors – actions – reactions analysis specifically for ETICS in order to estimate the main degradation mechanisms. The structure of the analysis has been based on EOTA Guidance Document 003, ISO 6241, UNI 8290.

The degradation factors to be studied have been chosen in reference to information provided by the Building Pathology: State of the Art reports, literature, maintenance handbooks [1999 Di Giulio et Croce 1999, Pizzi *et al.* 2000].

In detail, have been studied the thermal shock, the mould attack, and the wind action. In Table 2 an example of agent – action – effects analysis is portrayed about thermal shock in summer conditions.

	Agent	Action	Effects
DESCRIPTION	Thermal shock in summer condition (solar radiation and summer storm)	1. Heating (caused by solar radiation) → rendering expansion 2. sudden cooling (caused by rain) → rendering contraction	1. Rendering cracking mainly associated with joints between insulation panels 2. Loss of water tightness 3. Capillary wetting of rendering → differential dirtying of rendering (associated with insulation panels joints) → local increase of solar absorbance α → progressive increasing of the sensitivity of the system in following thermal shock cycles (increased action of sun radiation → increased surface temperature → increased ΔT between action 1 and action 2) 4. Further decay of mechanical characteristics of rendering and development of linear thermal bridges associated with cracking (close to panels joints) 5. Fully developed decay of thermal performance
PARAMETERS	Air – Sun temperature $T_{as} = T_{ext} + \alpha * G/h_{ext}$ G [W/m^2] total incident solar radiation α [0-1] solar absorbance h_{ext} [W/m^{2}K] external air-film conductance T_S [°C] Surface temperature	σ, τ, ε	- w [mm] cracking width - water absorption (according to capillarity test § 5.1.3.1 ETAG 004) - colour variation [surface colour measurement according to UNI 8941-3 and UNI 10623] λ [W/mK] - Ψ [W/mK] according to ISO 14683 - mechanical characteristics of rendering (such as tensile strength, resistance to hard body impact, etc.)

Table 2. Thermal shock in summer conditions

2.2.3 Suggest ageing tests and degradation indicators

Nowadays the experimental tests of accelerated exposure do not consider (or they partially do) the simultaneous effect of agents and the chain of failure. This means, e.g., that the rain – sun – freeze – thaw cycle is executed on one sample, while the effects of wind are observed on another sample. In this way all the information are not related, because of the fact that a superposition of effects cannot

be assumed for degradation factors (it is not a first order problem). A better approach could be the one proposed by the FMEA (Failure Modes and Effects Analysis).
Another important aspect is the shape of the samples: nowadays the ageing accelerated tests are executed on section, but most of the failures occur in building detail. So it could be significant to define some standard building details' shapes to test the ETICS. Two standard building details could be the masonry corner and fixture – wall detail.
In the ETAG 004 are not considered the interactions between the ETICS and the support and the whole building. These aspects instead must be assessed because the strains of the structure often cause cracking in the rendering.
It is also remarkable that the ETAG 004 does not identify a time re – scaling towards Service Life (comparison of degradations after short – term and long – term exposure and achievement of a time scale) of the proposed tests and does not determine a scale or a unit of measurement for most of considered degradations.

3 FACTOR METHOD FOR ETICS AIMED AT AVOIDING SUBJECTIVITY

This research project is looking for a factor method on demand for ETICS (it cannot be used to estimate the Service Life of other building components) based on the Agents – Actions – Effects Analysis. Its main features are:

- objectivity (achieved by introducing multiple choices and not a designer's free assessment);
- references to international standards and calculation procedures;
- guided procedure;
- experimental confirmation.

It has been considered to maintain the factor method structure owing to the fact that accelerated exposure tests do not offer exhaustive information about the duration performance of the building component: they only portray the effects of the climatic agents. In this way, the factor method could offer a useful structure to combine the information concerning the effects of climatic agents (measured by experimental tests), design level, work execution level and all items involved in duration performance.

3.1.1 Factor Method check – procedure

An ETICS system, whose duration performances are known, could be used to build up and verify the factor method, e.g. an ETICS technology used in past, which has been carried out without constructive pathologies, and has reached the end of Service Life. On samples of the considered unit of measure (the ETICS system whose duration data's are known) could be executed accelerated pre-tests (e.g. external exposition 45° inclined, South direction or experimental short – term exposure) in order to achieve the re – scaling factor. The duration of the ETICS assumed as unity will be assumed as Reference Service Life for ETICS systems and the factors (from A up to G) will be considered equal to one, as first step (in a second time it could be possible to translate the RSL values in order to assume unitary the configuration of a medium ETICS which is currently made).
A pre-dimension of the factors (and sub-factors) will be got through the assessment of an equip of expert designers. Then the same tests carried out on the "unitary ETICS" will be executed on n systems (n = sum of all sub-factors or macro – factors), with only one factor (or sub-factor) different from one and in this way will be performed a sensivity analysis. In order to asses all possible configurations it will be not possible to carry out tests on all possible ETICS: only a few tests will be accomplished to get the function analysis and predict the other values.
Some factors, such as factor C (work execution level) or G (maintenance level), will not be estimated through experimental accelerated exposure, but must be evaluated by means of references to standards (e.g. ISO 9001 and standards concerning the thermal properties such as in order to define macro – classes of thermal resistance), state of the art, etc.
In the last few years some studies have been published about the use of Monte Carlo Method in handling factors or sub-factors values of Factor Method. The most of proposed methods suggests a use of triangular distribution of data (easy to treat) to approximate a Gaussian distribution. Concerning

ETICS, only some aspects of factor A (quality of component), such as quality of materials, could be estimated with Monte Carlo Method (using Gaussian functions), because of the fact that the most part of working is made on yard.

In Figure 2 is portrayed a proposal of subdivision in assessment subjects of the Factor Method and in Table 3 is proposed a possible development of subdivision a factor (e.g. factor A) in sub-factors.

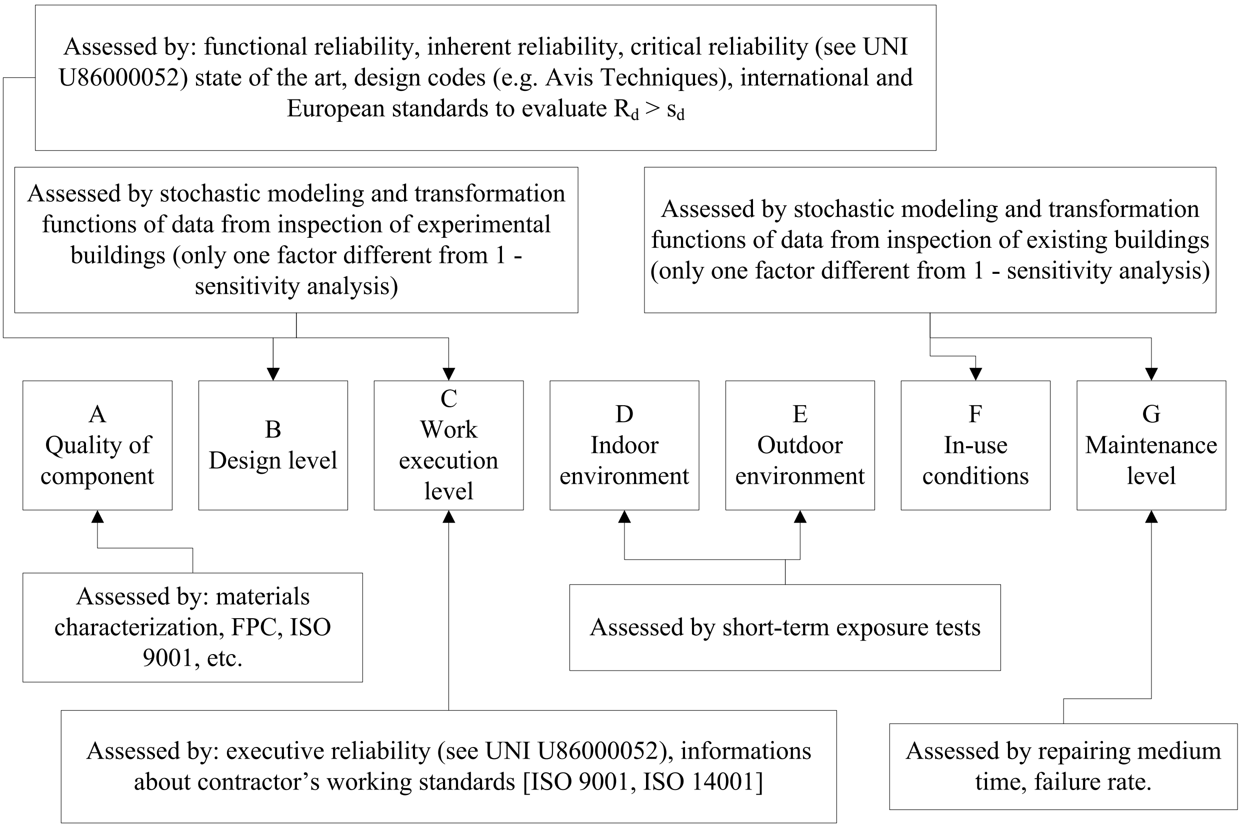

Figure 2. Use of Factor Method Structure in assessing Service Life of ETICS (this scheme is to be intended as purely approximate of a possible subdivisions in study items of factors)

	Factor A - Quality of component	Value
A_1	Quality of insulation product	
	No production certification	0,8
	ISO 9001, 14000	1
	ISO 9001, 14000 + FPC	1,1
…	…	
A_n	Thermal insulation product class TIPC = (R [m^2K/W]) / (Min T_{ext} [K])	
	TIPC < 0,0045	0,7
	0,0045 < TIPC < 0,005	0,8
	0,0050 < TIPC < 0,0055	0,85
	0,0055 < TIPC < 0,0060	0,95
	0,0060 < TIPC < 0,0070	1
	0,0070 < TIPC < 0,0080	1,05
	0,0080 < TIPC < 0,0085	1,1
	0,0085 < TIPC < 0,0090	1,15
	0,0090 < TIPC	1,2

Table 3. Example of subdivision of factors in subfactors

(factors estimation values aren't test confirmed yet)

5 CONCLUSIONS

Some important conditions should be verified to achieve the Factor Method proposed:
- the ETAG 004 procedure should be modified and its test linked between themselves to get a real ageing programme (not only a certification procedure);
- the results of the new ETAG 004 procedure should be available (privacy secured) to research institutes and universities so that researchers may improve and sharpen, in an economically sustainable way, the Factor Method (the results of the new ageing tests could be used to assess factor D and E).
- a suitable software should be implemented to manage the data (and its complexity) got by ageing tests and estimation of factors;

An objective Factor Method (not affected by subjectivity of evaluation of factors and confirmed by experimental tests) could forecast the duration of ETICS systems, allow a correct design of maintenance and so remarkably extend the Service Life. On the other hand the ageing tests could offer the opportunity of improving knowledge about modes of failure of ETICS.

6 REFERENCES

UNI U86000051 Progetto di norma sulla "Valutazione della durabilità dei componenti edilizi" – Parte 1: La durabilità dei componenti edilizi. Terminologia e definizione dei parametri di valutazione.
UNI U86000052 Progetto di norma sulla "Valutazione della durabilità dei componenti edilizi" – Parte 2: Il metodo per la valutazione della propensione all'affidabilità.
UNI U86000053 Progetto di norma sulla "Valutazione della durabilità dei componenti edilizi" – Parte 3: Metodologia per la valutazione della durata (Vita Utile).
EOTA Guidance Document 003 – Assessment of working life of products, Dec 1999
Guidance Paper F – Durability and the Construction Products Directive 89/106/EEC. EC DG III Industry, Construction 22/7/99
ETAG 004: March 2000 – Guideline for European Technical Approval of External Thermal Insulation Composite Systems with Rendering
EN 832: 2001 – Thermal performance of building – Calculation of energy use for heating – Residential buildings
ISO 6241: 1984 – Performance standards in building – Principles for their preparation and factors to be considered
ISO 6242-1: 1992 – Building construction – Expression of users' requirements. Part 1: Thermal requirements
UNI 8941-3: 1987 – Coloured surfaces – Colorimetry – Calculation of colour differences
UNI 8289: 1981 – Building – Functional requirements of final users – Classification
UNI 8290/2: 1987 – Residential building – Building elements – Analysis of requirements
UNI 8290/3: 1987 – Residential building – Building elements – Agents list
UNI 10623: 1998 – Colorimetry – Gloss difference compensation for surface colour measurement
UNI 10723: 1998 – Construction process – Classification and definition of the process steps for new constructions
ISO 14683: 1999 – Thermal bridges in building construction – Linear thermal transmittance – Simplified methods and default values
ISO 15686: 2000. Building and constructed assets. Service life planning. Part 1. General Principles
ISO 15686: 2000. Building and constructed assets. Service life planning. Part 2. Service life prediction principles
J. Hovde, K. Moser 2004, *Performance Based Methods for Service Life Prediction, State of the Art Reports*, CIB Report: Publication 294, CIB W080 / RILEM 175-SLM Service Life Methodologies Prediction of Service Life for Buildings and Components
Roberto Di Giulio, Sergio Croce 1999, *Manuale di manutenzione edilizia*, Maggioli editore, Rimini

Enrico De Angelis, Manuela Grecchi, Luigi Paolino, Emilio Pizzi 2000, *La facciata – Diagnosi del degrado ed interventi di ripristino*, Maggioli editore, Rimini

Building Life Cycle and Tacit Knowledge Base of Best Practice

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ABSTRACT

An enormous volume of knowledge is generated during the phases of brief, design, planning, construction, maintenance, facilities management and demolition of a facility. Throughout the whole life cycle of a building, organizations rely on their experiences, professional intuition, and/or other forms of tacit knowledge to accomplish satisfactory work. Usually, professionals find it difficult to reuse core experts' knowledge for highly knowledge-intensive building life cycle activities. This situation calls for better disseminating tacit knowledge from experts' brains to achieve higher quality building life cycle. In order to apply experts' tacit knowledge better the multiple criteria decision support systems have been developed for a building life cycle and its stages.

KEYWORDS

Building life cycle, tacit knowledge base, best practice, decision support system.

1 INTRODUCTION

Tacit knowledge base of best practice consists of informal and unrecorded procedures, practices, and skills. This "how-to" knowledge is essential because it defines the competencies of employees. Knowledge management systems is of value to organizations to the extent that it can codify “best practices” in a building life cycle, store them, and disseminate them through-out the enterprise as needed. It makes the company less susceptible to disruptive employee turnover. It makes tacit knowledge explicit. Tacit knowledge is highly personal, context-specific, and therefore hard to formalize and communicate. Tacit knowledge is knowledge housed in the human brain, such as expertise, understanding, or professional insight formed as a result of experience. Because of the orientation toward unique projects, much knowledge in the building life cycle is experience-based and tacit. The knowledge needs are dynamic, depending on the task to be performed or the problem to be solved. Nevertheless, the typical strategy for knowledge management is to make knowledge explicit and store it as computer software and databases. Organizations have been successful at collecting and storing explicit information in their databases, but they are poor at knowledge retrieval and exchange.

Tacit knowledge is extremely important to the organizations because, once a project is completed, professionals tend to forget it and start something new. Therefore, knowledge utilization is a key factor in productively executing a building life cycle.

In order to develop of tacit knowledge base of best practice its economic, technical, qualitative, technological, social, legislative, infrastructural and other aspects analysis is needed. The diversity of aspects being assessed should follow the diversity of ways of presenting data needed for decision making. Therefore, the necessary data may be presented in numerical, textual, graphical (schemes, graphs, charts), formula, videotape and other forms.

The paper is structured as follows. Following this introduction, Section 2 outlines explicit and tacit description of a building life cycle. In Section 3 we describe handling of best practice by organisations. The development of a decision support system for building life cycle is introduced in Section 4.

2 EXPLICIT AND TACIT DESCRIPTION OF A BUILDING LIFE CYCLE

A building life cycle consists of six closely interrelated stages: brief, design, construction, maintenance, facilities management and demolition. A building life cycle may have a lot of alternative versions. These variants are based on the alternative brief, design, construction, maintenance, facilities management and demolition processes and their constituent parts. The above solutions and processes may be further considered in more detail. For instance, the alternative building variants may be developed by varying its three-dimensional planning, as well as structural and engineering solutions. Thus, dozens of thousands of building life cycle alternative versions can be obtained. The diversity of solutions available contributes to more accurate evaluation of climatic conditions, risk exposure, maintenance services, as well as making the project cheaper and better satisfying a client’s architectural, comfortability, technological and other requirements. This also leads to better satisfaction of the needs of all parties involved in the project design and realization.

Various interested parties (clients, users, architects, designers, utilities engineers, economists, contractors, maintenance engineers, building material manufacturers, suppliers, contractors, finansing institutions, local government, state and state institutions) are involved in the life cycle of a building, trying to satisfy their needs and affecting its efficiency. The above needs or objectives embrace the expected cost of a building, maintenance costs, living space, number of floors as well as the requirements to its architecture, aesthetics, comfortability, functionality, proportions, materials, sound insulation of partition walls, taxes and allowances, interest rates, etc. Besides, the environment of the site, its ecology, sound level and local infrastructure are also taken into consideration. This list may be continued.

There are two essential branches of knowledge management in a building life cycle. According to one of them, knowledge can be *explicit.* Knowledge systems facilitate the storage, registration, organization, filtration, analysis, collection and distribution of *explicit knowledge.* Explicit knowledge is comprised of the documents and data that are stored within the memory of computers. This information must be easily accessible, so that an organization could get all the necessary knowledge without disturbances. The knowledge system is valuable for business as long as its present possibilities are practically used. One of the main management factors of such knowledge is information technologies.

The other branch, the main organizational knowledge is *tacit*. Knowledge does not belong to the group of the direct resources of a company (raw material, equipment, labour, finances). This is the integrated sum of physically intangible resources, the bigger part of which is tacit: skills, competence, experience, organizational culture, informal organizational communication networks and intellectual capital of an organization. It is impossible to manage such knowledge with conventional methods.

To sum up, it is necessary to stress that, nevertheless, the newest tendencies of knowledge management evolve from explicit to tacit knowledge management.

New knowledge is created during the interaction between people, when individuals having different types of knowledge (explicit of tacit) communicate with each other. Explicit knowledge, i.e. information is widely used in information technologies. The main organizational knowledge is tacit. It is impossible to manage it using conventional methods. The creation and distribution of tacit knowledge require creativity and competence.

Explicit and tacit description provides the information about various aspects of a building life cycle (i.e. economical, technical, technological, infrastructural, qualitative (architectural, aesthetic, comfortability), legislative, social ones, etc.). Explicit information is based on the criteria systems and subsystems, units of measure, values and initial significances as well as the data on the alternative projects development.

When drawing up the system of criteria fully describing the life cycle of a building, it is worth-while to take into account the suggestions of other authors. This is explained by the fact that the goals pursued by the interested parties and the system of criteria describing the projects in a certain sense are rather subjective. Therefore, in order to increase the degree of objectivity, we shall rely on the suggestions of specialists working in this field when drawing up the system of criteria describing the projects.

Tacit description of a building life cycle presents textual, graphical (schemes, graphs, diagrams, drawings), visual (videotapes) information about the projects and the criteria used for their definition, as well as giving the reason for the choice of this particular system of criteria, their values and significances. This part also includes information about the possible ways of multivariant design..

3 HANDLING OF BEST PRACTICE BY ORGANISATIONS

Perception, that much more attention has to be paid to the knowledge creation and spread in the form of the best knowledge bases and databases, have been recently set in a building life cycle field. Governments of the most progressive countries stimulate construction and real estate sector to store and spread knowledge of the best practice. It is tried to find it more quickly, describe in quantifiable and qualitative form, classification and spread. Advanced construction and real estate companies often point out, that they are applying the best practice of their own field. Storage, management and improvement of the best practice, and the best practice knowledge bases and databases created on their basis, is one of the main and justified priorities of advanced companies.

Comparative analyses of the best practice are becoming more popular in the world. They are based on the analysis of the best examples of a building life cycle. On the basis of this analysis, certain recommendations are formed, indicating how to provide services of higher quality and better serve needs of clients. They provide a possibility to quickly and efficiently understand and apply the methods, which could help to achieve the quality of client service of world-class. Further, in short usually occurring pieces of advice of comparative analysis of the best practice are provided: create favourable atmosphere for the employees of a company to constantly take interest in the best practice; pay constant attention for the search and practical use of the best practice; relate the best practice with the implementation strategy of a company; create systems of determination and recognition of the best practice; create information, expert, knowledge and decision support systems of transmission and spread of the best practice in a building life cycle.

Construction and real estate sector organisations, applying the information of the best practice stored in the databases and knowledge bases, could assume the following advantages: preserve vitally important experience when employees leave an organisation; install more perfect mechanism of decision making; be able to adapt more flexibly to shifting micro and macro environment; more perceptively determine client needs and satisfy them more accurately; increase efficiency of enterprise activity; decrease total expenditure on a building life cycle.

Databases and knowledge bases of the best practice are knowledge-obtaining tools, which allow to save a lot of time, provide information on the best construction and real estate sector business practice in different forms (criteria systems, slide presentations, structural schemes, text, video and audio material, etc.).

Interested groups most often are trying to achieve different economic, technical, technological, ethical, social and other aims. Different means could be used to achieve them. Some aims are not so easy to be achieved, others might require more expenses. The best practice allows not limiting oneself only to the implementation of economic aims; it creates conditions to better achieve integrated qualitative objectives and understand, from what perspective this practice was named as the best one. The main problem of many best practices is the way of their presentation: they are suggested, not taking into account certain situation. Let us say, auto-mechanics from construction mechanization organisation find for excavator-dragline the best excavate system, the best transmission mechanism, the best driving system, the most economical fuel system, etc. Let us say, that after evaluation it becomes clear (and it is most likely to appear) that such systems acknowledged (driving system, etc.) are the systems of different excavator-dragline producers. It is most likely, that the best systems of the best excavator-draglines joined together would not work. Every the best subsystem of an excavator-dragline is designed in the way, that it fits other specifically designed parts of a certain excavator-dragline. All an excavator-dragline is studied as a solid, very integrated mechanism. Often others could not physically substitute some parts or their substitution is not worth to be provided due to economical and efficiency approach. However, very often the best practice is presented as if it was applied ignoring certain situation.

The best practice is obtained in different ways: fundamental and applied researches, wisdom and experience stored by practicians, experience of clients and other interested groups, opinion of experts, models recurring in different spheres, etc.

Building life cycle knowledge is in design and contract documents, software, etc., e.g. spreadsheets, presentations, e-mail messages; and this information is not exchanged.

Providing all search results of the best practice, summary of each document, evaluation of search criteria match, and document source is presented.

If an employee leaves a company or if he is shifted to other position, his knowledge is easily lost. Possibility to efficiently spread and repeatedly use collective organisation knowledge is the basis, and with reference to it a company could obtain competitive advantage in different spheres of its activity.

Comparative analysis systems of the best practice help companies to determine directions of priority for the increase of activity efficiency and ways of determination of achieved progress, which allow to compare the performed construction and real estate management processes with existing ones in other organisations; as well as, determine the spheres lacking behind and suggest tools to eliminate these gaps. Modern companies know how to use possibilities of comparative analysis, and therefore decrease their expenditure, increase productivity and competitiveness.

In order to efficiently distribute existing resources, comparative analysis is performed in the spheres, in which better results could be achieved with the help of minimum analysis. Mostly principle of cost-benefit analysis is applied. Successful comparative analysis helps to distribute existing resources in a right way; afterwards, a company could implement its strategic aims. For projects and activities meeting the main company aims, high priority is given, when distributing the resources, time planning, accumulating efforts and applying experience. In reports of the best practice detailed information on successfully used strategy and tactics in the organisation is often presented, which are used by the most progressive companies when solving the basic objectives of their activity.

Knowledge management helps to solve issues of organisation adaptation, existence and competence, which become vitally important due to constant increase of discontinuity and inconsistency of micro and macro environment changes. This tendency stimulates synergetic coordination of information collection and its process capacities, creative and innovation abilities of employees.
Managers, trying to more efficiently apply the best practice in their organisation should pay attention to the following pieces of advices:

- Based on possibilities, create good conditions for organisation employees to use information, expert, knowledge and decision support systems, yet, guaranteeing, that in case of change of micro or macro environment conditions, necessary decisions could quickly be taken.
- In a company implement new, flexible organisational forms, which would create conditions for the establishment and better functioning of local and external interrelated activity communities. Communities of such activity are activity groups, established by non-official and half-official employees and persons not working in an organisation, on the basis of common troubles and interest.
- Stress your approval to tacit knowledge and human aspects related to them, such as ideals, values or emotions, in order to more efficiently formulate more comprehensive conception of knowledge management.
- Perceive organisation as community of people, which, using technological systems, could provide the received information with different meaning. Do not follow traditional approach, when most of attention is paid to orders and control.
- Try to refer the approach, stating that " it should be done because it was usually done like this" as less as possible. Then the best practice could be constantly and in different aspects evaluated and applied to dynamically changing micro and macro environment.
- Stimulate formulation of different approaches and do not make consensus too early, if it is necessary to perform more detailed analysis of assumptions and reasons. Often approaches of persons, having different qualification and experience of life might differ, and this would create a possibility to approach the issues discussed more widely. It is vitally important in order to understand the essence of the main issues, especially, when micro and macro environment quickly changes; and it is necessary to overview that was called as standard or the best practice the day before.
- Stimulate employees to more actively and initiatively use their experience, fantasy and creativity, because it will help to achieve inner organisation completeness, which matches variety of environment.

4 DECISION SUPPORT SYSTEM FOR BUILDING LIFE CYCLE

Based on above ideas multiple criteria decision support systems developed by the author and colleques for a building life cycle and its stages as follows [Zavadskas & Kaklauskas *et al.* 1994, 1995, 1996, 1998, 2001, 2002]:

- multiple criteria analysis of a building life cycle and its stages,
- multivariant design and multiple criteria analysis of refurbishment of residential houses,
- multiple criteria analysis of construction projects,
- multivariant design and multiple criteria analysis of in-situ buildings,
- project total quality analysis,
- multivariant design and multiple criteria analysis of one-family houses,
- multivariant design and multiple criteria analysis of foundations of agricultural production buildings of 1.810.2 series based on three-joint reinforced concrete frames,
- multivariant design and multiple criteria analysis of foundations of various types,
- multiple criteria analysis and multivariant design of agricultural production buildings.

In order to demonstrate the above systems a Decision Support System for Building Life Cycle will be considered below as a sample.

Based on the analysis of existing information, expert, knowledge and decision support systems and in order to determine most efficient versions of building life cycle a Decision Support System for Building Life Cycle consisting of explicit and tacit knowledge database, database management system, model-base, model-base management system and user interface was developed.

In order to perform a complete study of a building life cycle a complex evaluation of its economic, technical, qualitative (i.e. architectural, aesthetic, comfortability), technological, social, legislative, infrastructural and other aspects is needed. The diversity of aspects being assessed should follow the diversity of ways of presenting data needed for decision making. Therefore, the necessary data may be presented in numerical, textual, graphical (schemes, graphs, charts), formula, videotape and other forms.

Explicit and tacit description of a building life cycle and its stages made is used as a basis for developing a integrated databases containing overall information about it and allowing to carry out its multivariant design and multiple criteria analysis. Since the efficiency of any constituent part of the project depends on a particular party in its execution only integrated design of a building life cycle involving close cooperation of all interested parties can yield good results.

Alternative building life cycle versions include different cost of a plot and a building, maintenance costs as well as various architectural, aesthetic, space-planning, comfortability characteristics, infrastructure and environment pollution. Particular interested parties often have their own preferential rating of these criteria, also giving different values to qualitative characteristics. Besides, designing of a building life cycle allows for the development of plenty of the alternative versions of its particular stages. This causes a lot of problems in determining the most efficient project. To overcome these difficulties some integrated databases were developed. They contain a complex description of the alternative versions available in explicit and tacit forms. These data taken together can describe the object to be considered in more detail. The application of integrated databases described allows to better satisfy the needs of the parties involved as well as helping to choose an efficient project.

Integrated databases consists of the following parts:

- Initial databases. These contain the initial data provided by various interested parties allowing to carry out a complex design of the whole project or its parts.
- Evaluation databases, containing integrated explicit and tacit information provided by interested parties allowing to get a full description of the alternative variants. Based on the evaluation databases multiple criteria analysis of a building life cycle and its stages is carried out.

- Multivariant design databases consisting of integrated explicit and tacit information about possible combinations of the alternative variants available.

These integrated databases can contain data on theoretical and practical experience of the interested parties, some additional facts as well as the recommendations as to how to avoid previous mistakes.

For example, a decision support system using these databases can help compare the project being designed or executed with the alternative or already realized projects in order to find its disadvantages and provide recommendations as to how to increase its efficiency.

In this way, the use of integrated databases enables the user to take into account experts (including building owners and users, financing organisations, architects, engineers, manufacturers of building materials, contractors, state and its institutions, local governments, etc.) knowledge in various fields and the previous experience gained in developing similar projects applying them to currently developed project. For getting more efficient projects this information should be used at an early stage when the first meeting with a client takes place, which could save from repeating prior mistakes as well as leading to a more advanced and efficient project. In making a complex building life cycle design architects, designers, utility engineers, economists, contractors, suppliers, users can more efficiently solve common problems. This results in lower project cost and building time, as well as increasing its quality.

Interacting with the databases the user can get more detailed or integral information on the object considered. Given this opportunity and using the data from integrated databases as well as being provided with a decision support system, the user can find an effective project variant in a comparatively short time. In this way, a project best satisfying the needs of the client may be found saving the time of the client and designers.

In order to design a number of alternative building life cycle versions as well as determine the utility degree of the alternatives obtained and set the priorities new multiple criteria analysis methods […] were developed.

It is quite obvious that to develop and analyse thousands of the alternative variants based on dozens of criteria having each specific values and significances would be hardly possible without the use of computers. Only development of decision support systems could help solve this complicated problem. Therefore, to achieve the above-mentioned aims a multiple criteria decision support system consisting of databases, database management system, a model base, model base management system and the user's interface were created to be used for a building life cycle design and multiple criteria analysis.

5 REFERENCES

Zavadskas, E., Kaklauskas, A., Kvederyte, N. 2001, *Multivariant design and multiple criteria analysis of a building's life cycle*, Technika, Vilnius, 380 p.

Zavadskas, E., Peldschus, F., Kaklauskas, A. 1994, *Multiple Criteria Evaluation of Projects in Construction*, Technika, Vilnius, 276 p.

Kaklauskas, A., Zavadskas, E. 2002, *Web-based decision support*, Technika, Vilnius, 296 p.

Zavadskas, E., Simanauskas, L., Kaklauskas, A. 1998, *Decision support systems in construction*, Technika, Vilnius, 205 p.

Zavadskas, E., Kaklauskas, A. 1996, *Multiple criteria analysis of projects in construction*, Technika, Vilnius, 276 p.

Zavadskas, E., Kaplinski. O., Kaklauskas, A., Brzezinski, J. 1995, *Expert systems in construction. Trends, potential & applications*, Technika, Vilnius, 180 p.

ISO 15686-6 – Procedure for Considering Environmental Impacts

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ABSTRACT

Within the ISO series on Service Life Planning of Buildings and Constructed assets, ISO/TC59/SC14 has developed part 6, which focuses on the interrelationship between Environmental Life Cycle Assessment (LCA) and Service Life Planning (SLP). It has recently been published and establishes a procedure for the inclusion of environmental impacts into the framework of service life planning of buildings. This paper intends to inform about this standard, to describe the main content and to discuss some of the main achievements reached during the development of the document.

With the scope also including whole life costs (WLC or equally LCC 'life cycle costs') and environmental aspects, the concept of service life planning is more and more developing into an approach for integrated planning of buildings and constructed assets. In order to achieve this, it was necessary to illustrate the way the concepts of SLP, WLC and LCA can benefit from each other, and how they relate. Eventually it also showed reasonable to establish a framework in which these concepts are linked with each other. As a result, ISO 15686-6 on one hand describes a procedure for the parallel conduction of technical/functional, environmental and economic assessments. The procedure is intended to be applicable in project planning. At the same time, the standard illustrates how environmental declarations of building products together with the results of service life planning can be applied in order to perform a case-specific environmental assessment of design options. By this, the procedure takes an important departure from other established methods and tools for environmental assessment of buildings: Assessment references are not related to overarching targets and intentions of some tool developer, they originate from the performance requirements established in the clients brief. To benefit from the context of SLP, the assessment procedure follows a modular structure, where the various life cycle stages are not intended to be aggregated before the use stage scenarios have been adapted to relevant influential parameters from the planned in-use condition.

The result is an assessment result that shall enable the designer to find solutions that clearly address and satisfy the requirements expressed in the clients brief. On the other hand, due to the case-specific references, results obtained in this context do not contain substance for generic conclusions or assertions.

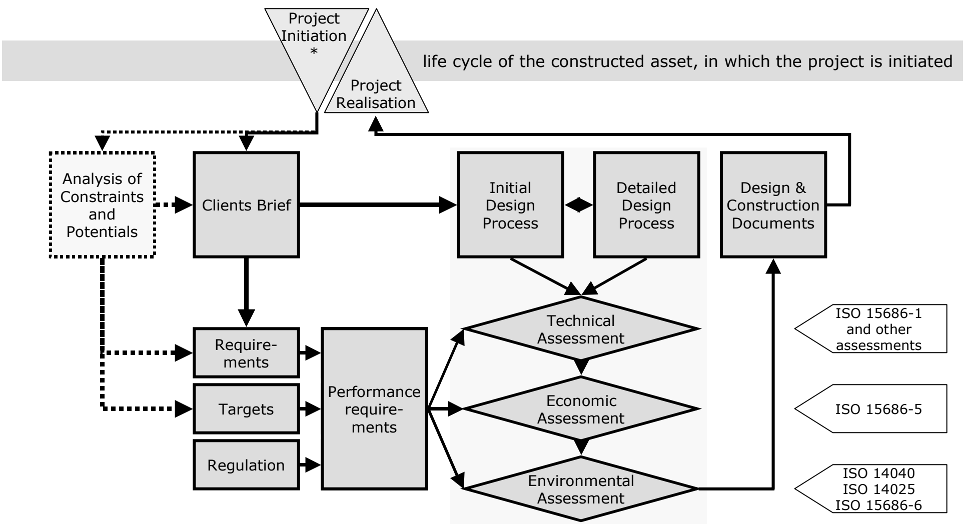
KEYWORDS
Service Life, Life Performance, Performance Requirements, Life Cycle Assessment

1 INTRODUCTION

Service Life Planning of buildings and constructed assets is subject of international standardization since 1993. The main purpose of standardization efforts in ISO/TC59/SC14 “Design Life” is to identify routines that support the design of buildings that meet identified performance requirements, throughout their design life. With this purpose, also the demand from the European Construction Products Directive [CEC], that buildings are to meet 6 essential requirements throughout their working life, is addressed. In parallel to the efforts of ISO/TC59/SC14, other standardization committees, like ISO/TC59/SC17 “Sustainability in building construction” and CEN BT/WG174 “horizontal standards for assessment of integrated environmental performance of buildings” as well as international research networks, like e.g. PeBBu, address topics that ISO 15686-6 focuses upon: the link between concepts for service life planning and environmental assessment, performed in parallel with cost assessments; all concerning the entire life cycle of a building and recognizing life performance of a building and performance requirements posed to the building and its parts.

The aim of ISO 15686-6 is a description of how to assess potential environmental impacts of alternative designs of a constructed asset. As it is part of the standard series of service life planning, it aims at application in the design stage, however recognizing that design not only takes place associated with new buildings, but that also e.g. refurbishments apply a design stage, see figure 1. The assessment of design options is a logical consequence of the aim, the assessment is not supposed to be a generic assessment of environmental impact of some product or service. Rather, it serves the purpose to enable a designer to identify a design solution that is beneficial in relation to identified project specific targets, and within the context of the current building and the requirements posed to it.

Figure 1: ISO 15686-6 procedure integrated into project planning



* Project may be initiated at any point in the life cycle of the building

In order to succeed developing this module of the service life planning context, it was first necessary to identify the link between concepts like environmental life cycle assessment, environmental declaration, reference service life and service life estimation. All together, these currently ongoing standardization efforts in the field of “sustainability in building construction” enable the development of integrated concepts for consideration of sustainability aspects of building construction [Trinius 2004].

Where the topic of sustainable construction on one hand provides the contextual frame and the philosophical reason for acting on the topic of service life, the methodologies developed to identify reference service lives and estimated service lives, also in terms of service life declarations, provide input to in especially environmental product declarations of building materials and components, and equally evident, to the assessment of environmental performance of buildings. Both items are addressed in ISO/TC59/SC17 and in CEN/BT WG 174. These working groups, as well as the European TG4 Report on Life Cycle Costing in Building Construction [TG4], already apply the concept outlined in ISO 15686-6, which in itself was based on discussions in the SETAC Europe Working Group on LCA in Building and Construction [Kotaji et al].

The current development of international standards follows to large extent a modular approach that shall allow the inclusion of use phase (and thereby service life) scenarios that can be adapted to better reflect the situation in which a material, component or system is to be applied. The ISO 15686 standards on Service Life Planning can be applied in order to generate information for such adaptable modules and scenarios, 15686-6 illustrates how to do this.

2 INTEGRATING ENVIRONMENTAL ASSESSMENT INTO SLP

In general, every product has some impact on the environment, occurring during any or all stages of the product's life cycle and being local, regional or global in character, or a combination of these. Environmental impacts associated with buildings and building products can be significant and should therefore be addressed as early in the design stage as possible. This also, as the economic consequences of design changes are larger the further the planning process has advanced, and as decisions already taken reduce the degree of freedom for decisions that have a contextual connection.

In order to avoid sub-optimization, environmental assessments of design options should most beneficially be performed in parallel with technical and economic assessments, all relating to project specific requirements and constraints. The purpose of all assessments is to provide decision makers and stakeholders with information relevant to their decision. Such comprehensive and reliable information about product performance is a rather complex item. The life cycle of buildings and their components are relatively long and the context of product application must be integrated in the information basis, where this context has an influence on life performance aspects of a design option. In this context, the environmental assessment draws significant benefits from the interlinkage with service life planning, as SLP aims to identify reasonable scenarios for predicted performance, and to make such scenarios more accurate.

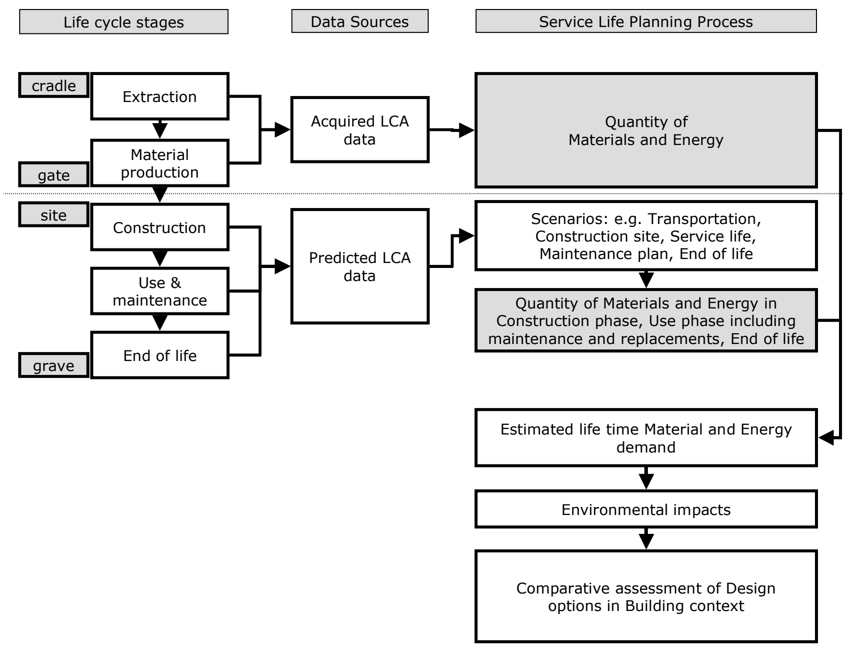
A general recommendation to address environmental and sustainability aspects in project planning is to early involve relevant stakeholders in the design process. The concept established in ISO 15686-6 provides for such involvement. An assessment is to be performed making reference to established performance requirements. It is then up to the actor committing the SLP or the environmental assessment study, to ensure that performance requirements address a relevant scope of concern. As this scope of concern may vary between projects, results obtained based on the approach of 15686-6 are project specific and not generic.

Determining and assessing environmental impacts of design options, and to decide whether differences between options are significant enough to be a reliable basis for decision making, requires detailed information about the design options and the context of their application. This also results in the fact that the later in the design process an assessment is performed, the more accurate the scenarios applied in the assessment can be. This conflicts with the statement above, that the earlier the design addresses environmental concerns, the easier it is to adapt. Therefore, the design process often follows an interactive route.

For early life cycle stages of products and components, LCA data can often be obtained by performing an inventory of the production processes involved in the supply chain. When striving to address the full life cycle perspective, assumptions on the life cycle stages from construction to the end of life often have to be made, see figure 2. The current trend of environmental declarations to include a full life cycle perspective means, that the declaration contains a scenario for the use phase of the product. Adoption of an EPD in that case also means to integrate such scenarios, whether they correctly reflect the situation in the planned building or not. To solve this potential problem, ISO 15686-6 as well as the SETAC WG Building and Construction, strongly recommend that information be kept separate. This modularity in information then allows to analyze the incorporated scenarios, and to adapt them where necessary [Trinius 2005].

Figure 2 – ISO 15686-6 life cycle stages, data sources and the link to service life planning

3 ENVIRONMENTAL ASSESSMENT OF DESIGN OPTIONS AS ELEMENT OF PROJECT



PLANNING

Environmental assessment of construction products, elements or entire buildings may be carried out prior to, within, or after project planning. The purposes of these assessments will be different, as in especially assessments carried out:

- Prior to the design and planning process may provide information about materials and components that the designer can make use of in his work, or that may inform the client about key items of environmental performance. A classical example would be an environmental declaration that includes an assessment of the declared environmental information.
- Within the design and planning process, where the designer performs assessments of various options of the design work, in order to identify the overall most beneficial option. In such assessments, the options are assessed considering the context of the planned building at the currently available level of detail. In comparative assessments, the design options must, as part of a functional component, relate to the same requirements.
- After completed planning, or even after realization of building construction, in order to verify the result of the planning process or in order to assess the produced building.

As ISO 15686-6 is developed as a module of service life planning, it addresses the second of these assessment situations. For the assessment procedure, this means that

- Assessment references can relate to project specific requirements and targets and can address performance requirements derived from various stakeholders' fields of interest.

- Potentials and constraints related to the current project can be considered in the assessment

- With the development from initial design to more detailed design, the information about the building and its life cycle can be refined, leading to the possibility to involve more detailed information in the assessment, both relating to product performance and in-use conditions.

- When carried out based on information available from other items of service life planning, that information can be incorporated in scenarios underlying the assessment.

- Technical, economic and environmental assessment can be performed in parallel; all based on the same project specific considerations, enabling the designer to apply a broader basis for his decision taking.

3.1 Comparative assessment results obtained on 15686-6 basis have no generic validity

When making comparative assessments based on LCA methodology, numerous requirements apply, all with the intentions to enable "just" comparisons, see the ISO 14040 series on LCA [ISO 14040].

Comparisons must be made on the basis of a common functional unit. For comparative assessments of design options, the context is a still evolving (and hence not entirely determined) design, which means that the exact context of the application of the design options is not yet known, and that often a proper functional unit may not be possible to be determined. Further, in case of LCA in the design stage, it appears most important that the design options meet the same performance requirements rather than having the exact same functional unit. This leads then directly to a limitation of the validity of assessment results. Results obtained according to the procedure laid out in 15686-6 are only valid for the design situation for which they have been obtained. They are only assumed to support decision-making in that very design process and have no general validity. They should not be applied "as true" for other design situations.

Figure 3 aims to clarify that the design options 1 and 2 are not necessarily to be equal in functionality, but that they, together with their context, have to relate to the same requirements.

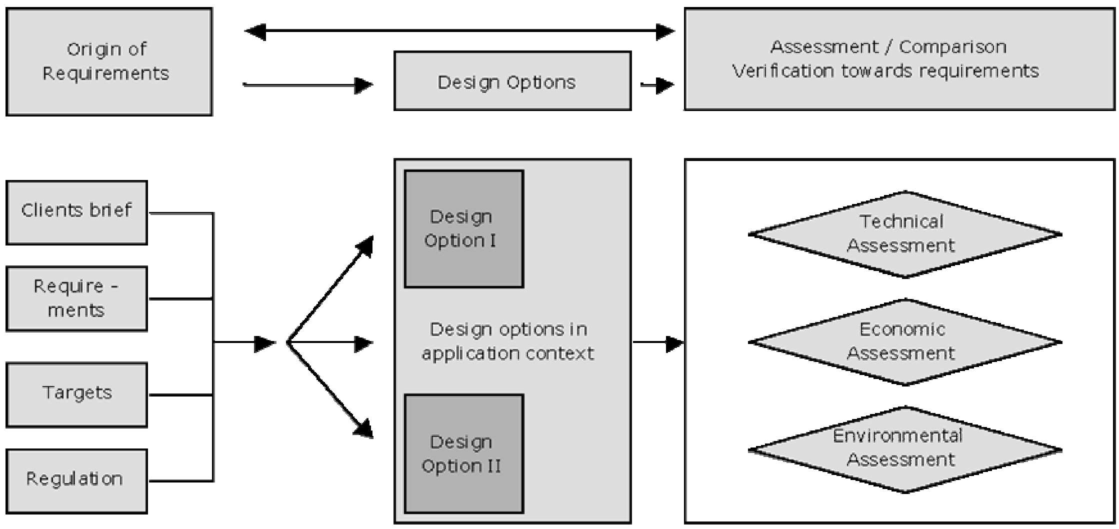


Figure 2 – ISO 15686-6 comparative assessment of design options

4 DISCUSSION AND CONCLUSION

The current trend appears clearly to be that the move from consideration of environmental aspects towards more holistic approaches. Often then, the term "sustainability" is involved. Broader approaches require an integration of numerous - formerly separated - approaches, which can be illustrated by the content and structure of ISO/TC59/SC17 and its proposed standards [ISO AWI 21930][ISO AWI 21931]. Benefits of integrating modules either relate to the importance in decision-making processes (like LCC and LCA), or appear as the modules provide each other with valuable information and strengthen each other's significance (like SLP and LCA/LCC). Where environmental declarations of construction products are equipped with scenarios covering the use phase and the service life, the link between SLP and LCA becomes evident. Meanwhile, the inclusion of life performance aspects in environmental declarations also means that aspects that are not related to the declared product alone, but refer to the application context of that product, are included in the declared information. A highly interesting field of assignment of performance and requirements is evolving, where product performance and building performance strongly rely on each other [Trinius, Sjöström]. Combined with the approach of performance-based building, SLP can address "life performance" in relation to project specific requirements. Both, these requirements, and the derived life performance can strengthen scenarios applied in LCA and LCC. Resulting is a concept for building assessment that is very much adaptable to project specific preconditions and performance aspects, as laid out in ISO 15686-6. Recently, focus has been directed onto standards for functionality requirements and serviceability [ISO AWI 21933-1]. "Sustainable Building" can very much benefit from this current development. And this development is not just academic or distant, international research networks, as well as standardization committees are actively and currently developing the modules needed for integrated and more holistic, yet more flexible assessments.

6 REFERENCES

CEC, 'Council directive of 21 December 1988 on the approximation of laws, regulations and administrative provisions of the Member States relating to construction products', Council of the European Communities, 1988, Brussels. 89/106/EEC

TG4, 'European Commission TG4 report on: Life Cycle Costs in Construction', final report of the tripartite meeting group on the competitiveness of the construction industry, Brussels 2003

ISO 14040, 'Environmental Management – Life cycle assessment – Principles and framework', ISO 14040, International Standardization Organization, Geneva

ISO/TR 14025, 'Environmental Labels and Declarations – Type III environmental declarations', ISO/TR 14025, International Standardization Organization, Geneva 2000

ISO 15686-1, 'Buildings and Constructed Assets – Service Life Planning – General Principles', ISO 15686-1, International Standardization Organization, Geneva 2000

ISO 15686-2, 'Buildings and Constructed Assets – Service Life Planning – Service Life Prediction Procedures', ISO 15686-2, International Standardization Organization, Geneva 2001

ISO 15686-6, 'Buildings and Constructed Assets – Service Life Planning – Procedure for Considering Environmental Impacts', ISO 15686-6, International Standardization Organization, Geneva 2004

ISO AWI 15686-9, 'Buildings and Constructed Assets – Service Life Planning – Guide on the Inclusion of Requirements of Service Life Assessment and Service Life Declarations in Product Standards', Standard developed by ISO/TC59/SC14, International Standardization Organization, Geneva 2004

ISO AWI 21930,'Building Construction – Sustainability in Building Construction – Environmental Declaration of Building Products', Draft International Standard developed by ISO/TC59/SC17, International Standardization Organization, Geneva 2004

ISO AWI 21931, 'Building Construction – Sustainability in Building Construction – Assessment of Environmental Building Performance', Draft International Standard developed by ISO/TC59/SC17, International Standardization Organization, Geneva 2004

ISO AWI 21933-1, 'Buildings and Constructed Assets – Service Life Planning – Functionality Requirements and Serviceability', ISO AWI 21933-1, International Standardization Organization, Geneva 2004

Kotaji, S., Schuurmans, A., Edwards, S., (Eds.) *Life-Cycle Assessment in Building and Construction*, Society of Environmental Toxicology and Chemistry 2003

Trinius, W., Sjöström, C., 'Service Life Planning and Performance Requirements', *Building Research & Information*, in print, London 2004

Trinius, W., 'Sustainable building – Integrated planning concept and realisation in project planning' *European Roundtable on Sustainable Consumption and Production,* Bilbao 2004

Trinius, W., 'Modules of Environmental Performance Assessment related to Durability and Service Life' *10DBMC International Conference On Durability of Building Materials and Components Lyon 2005*

7 PROJECT REFERENCES

CEN BT/WG174, European Standardization Working Group, developing standards for "integrated environmental performance of buildings"

PeBBu, Thematic research network, for information see http://www.cibworld.nl

ISO/TC59/SC14, ISO subcommittee on Design Life preparing ISO 15686 standard series on Service Life Planning

ISO/TC59/SC17 ISO subcommittee on Sustainable Construction: Four standard documents are under preparation:
(1) General Principles and Terminology;
(2) Sustainability Indicators;
(3) Environmental Declaration of Building Products;
(4) Assessment of Environmental Performance.

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LYON [France] 17-20 April 2005

Appropriate use of the ISO 15686-1 factor method for durability and service life prediction

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ABSTRACT

Developing the detailed design life of a building or other constructed work is a key stage in service life planning. To predict the design life for the overall works in detail the design lives of the individual elements and systems need to be configured in a hierarchy, building up from the smallest elements to the whole works.

The proposed components and materials can then be considered. Failure modes and the causes of loss of serviceability need to be established along with their effect on the service life, and where relevant the risk and likely cost impact of premature failure established.

ISO 15686-1 recommends that, as far as possible, this analysis is based on service life prediction data generated using the approach set out in ISO 15686-2. However, where such data is lacking, the Factor Method described in ISO 15686-1 may be used as a means of generating the data required to develop the service life design.

Whilst the Factor Method may be helpful in developing the design life, it is important to appreciate the difference in quality of data between the two approaches. This has important implications for the development of design lives, and this paper considers how the data source influences the quality of design life prediction.

The paper demonstrates the application of the Factor Method to a specific element. It includes a critique and qualification of the approach and suggests that more attention should be given to the degradation both in terms of likelihood and consequences of risks of unacceptable loss of serviceability, and proposes an appropriate course of action.

KEYWORDS

Design life, service life, reference service life, failure mode, factor method.

1 INTRODUCTION

Developing the detailed design life of a building or other constructed work is a key activity in service life planning. Probabilistic design is widespread for major, usually infrastructure, projects and service life prediction is an accepted part of the design. This is not often the case for many smaller scale works, for example low rise domestic construction, where the use of the factor method may be appropriate.

Since the draft ISO 15686-1 first appeared in 1997 a number of papers have been presented at Durability of Building Materials and Components conferences and at CIB Congresses. Interest has focused mainly on ISO 15686-1, (ISO, 2000) and particularly the so-called “factor method” described in clause 9. However, this focus may be at the expense of the service life prediction approach described in clause 8 of the same standard, which is the subject of the entire ISO 15686-2, (ISO, 2001).

Ideally, to predict the service life of a component or element within a building, the micro climate needs to be known, the performance of the component or element under the specified climatic conditions should be accurately characterised by data from real life exposure in identical conditions, and the construction and maintenance regime for the building should be clearly specified and likely to be delivered in practice. It is, however, idealistic to expect this.

In practice service life prediction is based on judgement. This might relate to the actual microclimate to be experienced, or to the expected performance of the materials and components incorporated within the building. ISO 15686-2 is devoted to the testing of materials and components for the purpose of deriving data to support such judgements. In the absence of service life data from real life or from testing to ISO 15686-2, then the factor method outlined in ISO 15686-1 may be a useful tool.

The ‘factor method’ originated in the Architectural Institute of Japan (1993), and was further developed by Bourke and Davies (1997). It involves modifying a “reference service life”, which is the expected service life of a product, component, or system under known conditions, using a series of “factors”. The factors take account of the environmental exposure (internal and external) and anticipated installation, in-use and maintenance regimes. The purpose is to provide an empirical estimate of the likely service life. ISO 15686-1 is clear that it does not provide an assurance of a service life. Because the factor method could be applied in an almost infinite variety of situations there are no “standard” factors, although Appendices to ISO 15686-1 provide some examples of how the approach might be used.

When ISO 15686-1 was drafted the Factor Method was seen as a second choice approach in the absence of more precise, or even ‘scientific’ data. The basic factor method philosophy is one of estimating based on performance of a similar system under different conditions. The factors are ‘rough and ready modifiers’, addressing all the key aspects of the design which may affect its performance.

The factor method may be described as a means of “producing an answer by Friday”. This is not to minimise or devalue the approach, rather to put the factor method into the context in which it is intended to be used. This paper looks at the development of a service life prediction with reference to both Parts 1 and 2, and considers how the source of the data used in the prediction influences the quality of the overall service life design. It uses as its example the detailed design and service life prediction of a roof.

2 THE DESIGN LIFE

It is important to have a macro context for the design life of the constructed facility and to ensure that the design life is appropriate. That is to say that the design life should reflect the existing needs of the owner and occupiers, and should also be able to meet future challenges that may arise from, for example, carbon based accounting and other sustainability criteria. This overall context will provide the basis for the design life of the works.
The objective of service life planning is to 'provide reasonable assurance that the estimated service life of a new building on a specific site with planned maintenance will be at least as long as the design life' (Clause 5.1 ISO 15686-1 2000).

In practice there are a range of stated design lives in any constructed work. These all influence the service life planning outcome during both the design and construction and during the subsequent operation of the asset. For example, a housing client may opt for a "Public Private Partnership" Contract for 35 years using an off site factory produced system, with a service life insurance scheme such as that operated by Construction Audit 1999, 2003. In this case the design life is 35 years.

ISO 15686-1 Clause 6.2 addresses "the brief", and requires a decision establishing the design life of the building and leads into the actions set out in Table 1. For the service life estimation stage the designer should 'identify the components requiring service life prediction'. Whilst the idea that the designer determines the design and service life may be implied in ISO 15686-1, it is important not to take a component out of context, but to set its design life in the context of the whole works.

3 CLUSTER MAPPING

The definition of context or configuration of the components can be assisted by the technique of cluster mapping. The components and materials are sorted by element or system within the works. Buildings are made up by combining components or materials into assemblies of elements and systems (Figure 1).

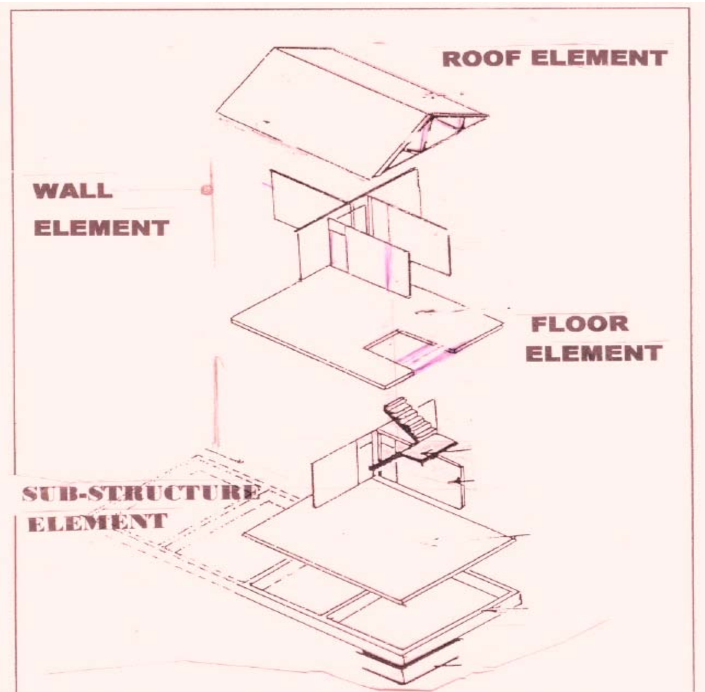

Fig 1 The Constructed Work's Configuration Breakdown

In cluster mapping the constructed work is broken down from its General Arrangement, Elevations and Sectional drawings into its respective elements and systems. The concept of the cluster map is illustrated in Figure 2. The cluster map is developed in the following stages:

Stage One identifies the overall design life of the works.
Stage Two identifies the individual Elements and Systems.
Stage Three establishes the design life for each element and system
Stage Four identifies the cluster of materials and components within each Element and System.

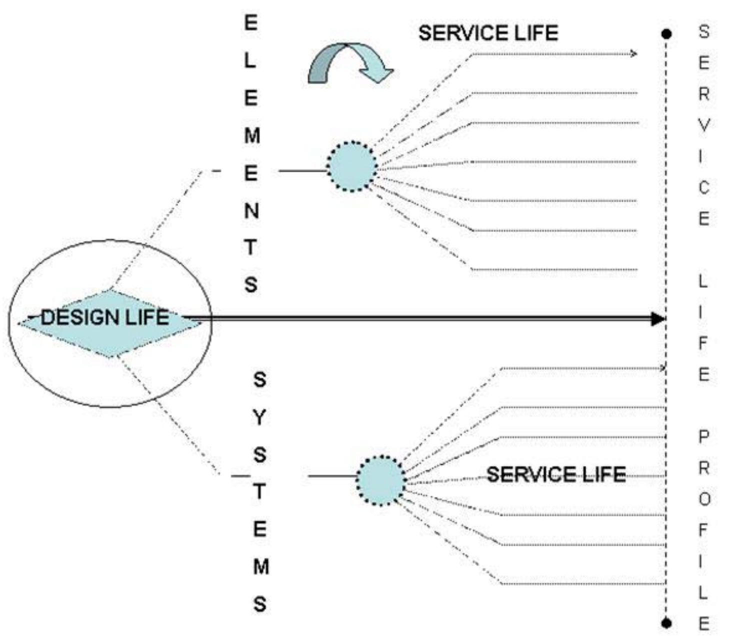

Figure 2 Cluster Context Mapping

The importance of Stages Two and Three is illustrated in Figure 2. The design life of the whole works drives the design life of the elements and systems, and requires an analysis of each element and system within the whole building. From the cluster map it becomes clear which elements and systems must have the same design life as the works, because replacement is not an option. It is also possible from the cluster map to identify elements and systems which may be replaced during the design life, such as rain screen cladding, waterproofing, sealants, boilers or air conditioning units.

This is important for sustainable construction. Reducing resource use in construction requires more flexible buildings, better able to be adapted and reconfigured or re-used. Such re-use may require changes to systems and elements in the building, such as facades and internal partitions. The design life of such elements and systems needs to be specified with a view to the possible future use of a building.

It is important to identify each component and material in turn within each element and system. The location and use affects the service life, and each material or component will have its own varying decay characteristics depending upon their configuration, environment and potential for maintenance. In the case of systems such as boilers there is also a need to consider their reliability and serviceability. As well as considering the failure modes, probability techniques such as the mean time to failure and survivorship analysis may also be used for such mechanical systems.

The overall design life has implications for all the different elements and systems. For example, whilst the foundation system may remain in service throughout the design life it may face aggressive ground

water or contaminated ground conditions that will result in protective membranes being of limited value, concrete durability being impaired or the compressive medium used to accommodate ground movement rendered ineffective. Ultimately failure of such critical elements may spell the end of the service life of the whole building – otherwise described as “service death”.
Such implications exist for all elements and systems, particularly when moving from empirical based construction to performance based designs with limited or non existent data. In all situations however, it is essential to map and formalise both the building context and its individual elements and systems configuration before a detailed analysis can be undertaken. It should also be understood that this process may need to be repeated as life cycle assessment and whole life costing are brought into the service life planning process, and may lead to the conclusion that a particular element or system cannot meet the design life requirements, and must be changed.

4 APPLICATION OF THE ISO 15686 APPROACH IN PRACTICE

A roof element has been selected to explore the relationship between ISO 15686-1 -2. The ideal scenario for predicting the service life of any component or element within a building is one in which the micro climate is known, the performance of the component or element under the specified climatic conditions is accurately characterized by laboratory, or better still, real life data and the construction and maintenance regime for the building clearly specified and likely to be delivered in practice.

This case study explores the need to analyse the composition of the element and the practicality of predicting its service life. It is preferable to predict performance based on laboratory or real life experience data. No more clearly is this shown, than in meeting performance requirements in the roofing element now being considered in Figure 2. For once the roof starts to fail there is a risk of internal damage both within an element like a warm roof and to the interior, which may result in premature failure of other elements, however good the detail or specification was.

4.1 The case study building

The building is exposed, facing westerly winds on its long face for up to 30 days a year. It is an earth stone wall barn structure of some 100 years or more in age but has a structurally decaying roof carcass and areas of roof covering are missing. The roof surface area during the summer months also receives strong sun rays for a significant part of the day with temperatures up to the low 30s Centigrade.

The project described is the conversion of an existing 5M X 20M two storey barn into a dwelling. The client requires a design life of 40 years. The site is in a marine coastal environment with wind speeds that may exceed 135km per hour for between 2 and 5 days a year. The building’s roof covering requirement has been defined by a Local Authority planning condition as an approved natural slate as the dwelling is in a conservation area.

To meet Building Regulations requirements a warm roof supported by plate connected truss rafters at 600 mm centres, suitably braced, has been specified. The specification relies on the established guides and standards (BSI 2003). The service life has then been assessed using ISO 15686, identifying what factors were critical and where the ISO 15686-2 or the factor method are relevant.

Figure 3 shows the resulting cluster analysis for the roof element. It identifies the sub-elements – ridge, battens, slates, rafters, wall plates, rainwater drainage and the soffit cladding system. This analysis can be used to identify weak links in the design. It may reveal the need to redesign in order to meet the design life requirement. It places increasing reliance on material and engineering science in the wider context of service life planning. This will become more important as we move to performance based regulation of construction. The clusters and their subsets are all duly numbered and identified in Table 1 in the eight sections. The responses to the issues in the factor method as described in ISO 15686-1 Part 1 are addressed. The factor classes are as set out in the ISO, namely:

A Quality of components
B Design level
C Work execution level
D Indoor environment
E Outdoor environment
F In use conditions
G Maintenance Level

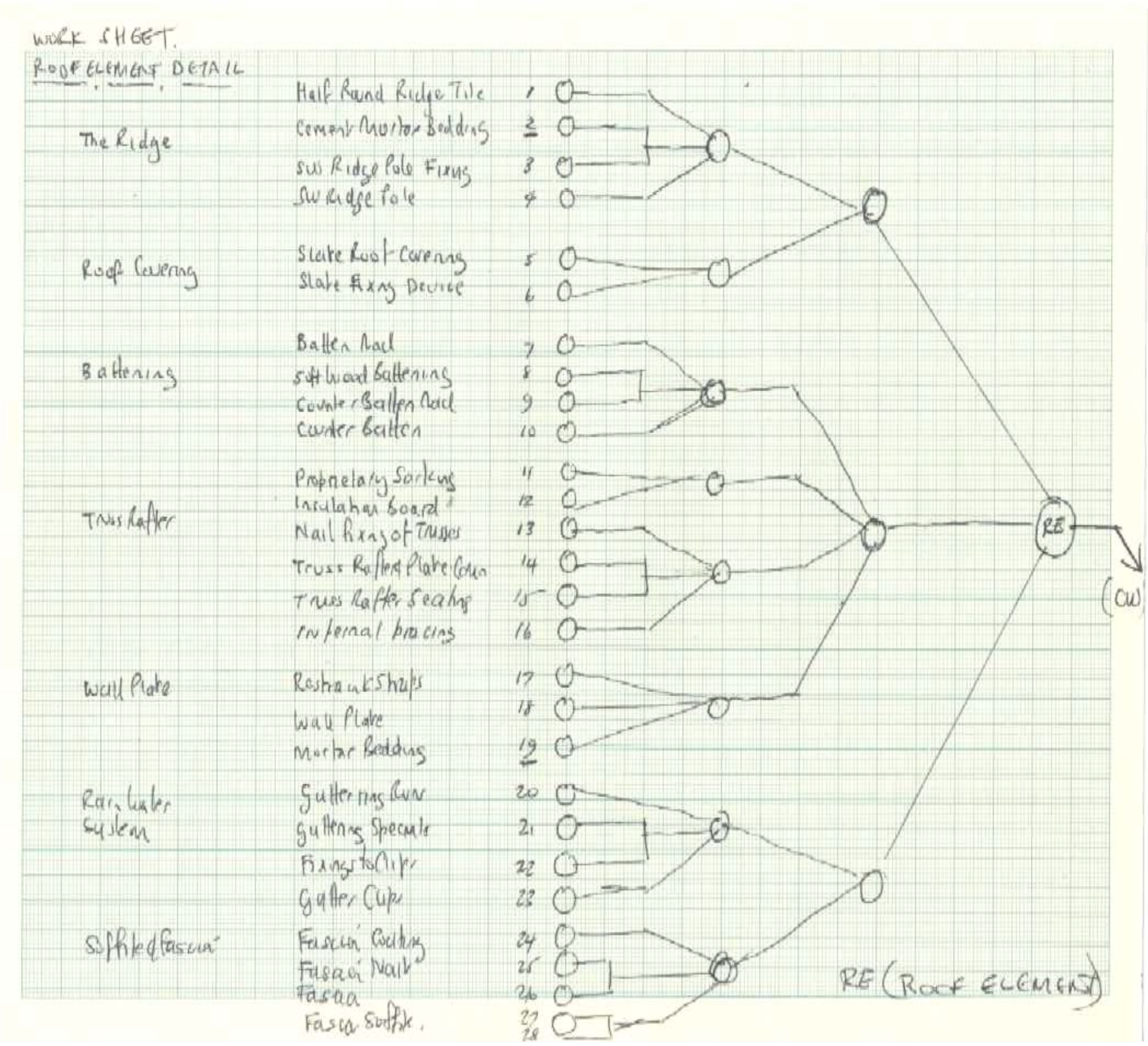

Figure 3 Cluster Mapping

Note: This Work Sheet originally had nine sub clusters. Eight are shown: the ninth is the chimney stack for a Combination Boiler. This has been omitted from the study as it was included in the analysis of the heating system. However, the interface with the roof was a consideration in practice.

Within each sub-cluster a specific action or response is noted, e.g. to respond to maintenance for the soffit, barge board and fascia area (Lines 23-28) whilst the treatment of the roof covering (Line 5) using the Factor Method needs considerable qualification.

Where relevant the risk and likely cost impact of premature failure is identified, along with those service life critical dependencies and possible failure modes or service losses that could arise in the element or constructed work. Based on this analysis, a reference service life (the expected service life under defined conditions) can be specified for each line in the table. This analysis can also feed into a life cycle assessment or life cycle costing analysis.

Table 1 Summary of the Roof Element's Constituent Parts

Roof Element		RSL	Factor Classes							Factor Class Comment
Ref	Constituent Part	Yrs	A	B	C	D	E	F	G	
1	Clay Half round ridge	40	*	*	*				*	Tile and mortar quality and workmanship play an important part here but some re-bedding might be expected on a maintenance repair basis
2	Bedding mortar	40	*	*	*		*		*	
3	Ridge Pole nail	-								No change of note expected
4	Ridge Pole SW Batten	-								No change of note expected
5	Roof covering	75	*	*	*		*		*	Environmentally exposed including during construction and in service access
6	**Slate fixing**	**Part 2**	*	*	*	*	*		*	Serious risk of corrosion compromises whole roof
7	**Batten nail**	**Part 2**	*		*	*			*	Performance is likely to be a core issue
8	Batten	75	*			*				Few known failures to date
9	**Counter batten nail fixing**	**Part 2**	*	*	*	*	*		*	Performance may be impaired if workmanship or specification is sub standard and risk of corrosion
10	Counter batten		*		*	*	*		*	Performance and reliability must be equal to the roof covering life
11	Sarking membrane			*		*				Performance over time not known but expected to remain serviceable
12	Insulating boards	N/K	*	*	*	*	*	*	*	Overstressing though hogging risk on installed EPS especially if wind loads increase due to climate change and risk of insect bird attack
13	Truss rafter	40	*	*	*	*		*	*	D some risk of deformation due to temperature and wind loading over time Poor installation/ fixing may impair whole
14	**Truss rafter plate connector**	**Part 2**	*	*	*	*		*		A, D, C Plate connector corrosion risk the longer the truss remains in service especially from a possible breakdown from the warm roof protection and/or RH
15	Truss rafter plate clip seating	-	*	*	*	*				Indoor environment might lead to some corrosion but not considered serious other than nailing
16	Internal bracing	-	*	*	*		*		*	The importance of workmanship and design method becomes critical over the climate change cycle and the area poses a risk
17	Restraint straps	-	*		*		*			
18	The wall plate	-								No change of note expected
19	Mortar bedding	-								No change of note expected
20	Guttering runs	20	*	*	*		*	*	*	Performance impaired if design/ installation is poor and annual maintenance is essential
21	Guttering + special components	15	*					*		Whilst there may be loss of serviceability it is likely as the parts age they will simply break whilst the gutter seating may drip. So in terms of the roof element one expects a cyclic replacement 15-20 years
22	Fixings to guttering brackets	15		*						
23	Gutter brackets	15			*					
24	Fascia protection	1-2		*			*			Extreme conditions where exposed timber may be the best solution
25	SW fascia and barge boards	30	*		*					Variable environment. Vent grill likely to suffer from marine and solar environment

26	Fascia soffit	16	*	*	*	*	*			some replacement may be necessary on the
27	and grill	9	*	*	*		*			Westerly Elevation and SW Barge Boards

* denotes factors critical to the performance and reliability of the cluster or of the element in service.

It can therefore be seen that the selection of the reference service life is critical to the success (or otherwise) of the whole approach. The cluster analysis leads to the conclusion that the performance of the whole roof structure is critically dependent on the truss plates and the nails used to fix the battens to the trusses and the slates to the battens. Because of the marine environment it is important to consider the potential corrosion risk arising from the salt laden atmosphere, and its likely effect on the performance and service life of the whole roof. Moser (2003) considered the impact of material degradation and its potential to impair the integrity of an element or component, leading to early unexpected replacement.

The cluster analysis enables identification of critical components and materials within the element, highlighting the significance of the service life data, and indicating which aspects of the data set require the greatest attention. In this case the performance of the metal fixings is vital, and the most accurate reference service life data is needed for these lines in Table 1. It is possible to take service life data from similar situations and apply factors, but there is a strong case for seeking the greater accuracy that data generated using ISO 15686-2 can offer. Service life data derived from quantified corrosion testing is likely be far more accurate in this case, and should be obtained if at all possible.

In the UK such data is available, although not obvious. The Code of Practice (BSI, 2003) for roofing and slating contains guidance on the appropriate fixings for use in a variety of environments, based on corrosion data and practical experience acquired over many years.

A further issue relates to the length of the design life and the consequent requirement for accurate data. The design life might only be 40 years, but it is important to know what it represents. It may be a period in which there is less than a given probability, say 5%, of failure, which is sometimes the case when it the basis for insurance against premature failure. It may be the length of time until a significant number of components or parts of a system need replacing. It may of course be the life of a structural element, failure of which ends the useful life of the building or works.

This context needs to be considered when using either the factor method or service life data derived using the approach in ISO 15686-2, as the context sets the requirements for accuracy of data. Table 2 shows the results of an analysis of the likelihood of the roof described above achieving three different design lives. It shows that certain aspects of the design may not be able to deliver a service life of 60 or 110 years. This shows once again that the reliability of the data used for service life prediction needs to be considered with care, and in the full knowledge of the overall context.

Table 2 Cluster Design Period Summaries

Cluster	Ref	40 years	60 years	110 years	Comment
Ridge	1-4	Ok	ok	Ok	Cyclic maintenance and replacement
Roof covering	5-6	Ok	ok?	No	Factor and Probabilistic Analysis required
Battening	7-10	Ok	no	No	Probabilistic Analysis required
Truss Rafter	11-16	Ok	no	No	Factor and Probabilistic Analysis required
Wall Plate	17-19	ok	ok	Ok	No response expected
Rain water system	20-22	Ok	ok	Ok	Cyclic maintenance and replacement
Barge Boards Fascia soffit	23-28	Ok	ok	Ok	Cyclic maintenance and replacement

Again, to assess the importance of this it is Aarseth & Hovde (1999) show that both the probability of an event and the frequency or occurrence must be considered in a risk based approach. Such methods are already being used in the maintenance and asset management field. When using the factor method for replaceable elements and systems it is important to consider the risk of failure as well as the element and system design life.

Questions often arise about “what the factors should be”. Unlike partial safety factors in engineering documents, however, there is no “standard” set of factors. Instead the user needs to decide for themselves what factors to apply in a given set of circumstances. The analysis outlined above will be a great help in doing this, as it will identify those aspects of the service life planning which require greatest attention, and are most critical to an accurate prediction of the service life. For example, a product known to be highly sensitive to the skill and care of the installer probably should not be specified for use where such skill and care is not readily available. In the case study the service life of the insulation media poses a considerable challenge, as it may be shorter than the specified design life. . Likewise the service life of the battening and truss fixings requires a material corrosion response.

For these reasons it is not possible to produce “standard” factors. Indeed, if it were possible to produce standard factors the factor method would be redundant, as it would be possible to produce standard service life data instead. However, the range of in use conditions and environments renders that task impossible. Instead, service life prediction requires the skill and care of an experienced practitioner able to assess the key components within an element or system, and the most significant possible failure modes and deterioration mechanisms, and predict the likely performance of the element or system under the proposed in use conditions.

Finally, it is important to note that the basis on which service life data has been chosen needs to be acknowledged for reference in the performance review or audit stage (ISO 15686-3, 2003) and the limitations and assumptions that have been made need to be recorded.

5 CONCLUSIONS

Where accurate service life prediction data is required design life mapping has the potential to identify the key aspects of the design, enabling greater focus on them. Judgement and qualification must be exercised when predicting service life, with particular attention to the accuracy and reliability of the data used. As with computers, so with service life: garbage in – garbage out!

Perhaps it is time to suggest that all researchers who produce papers on ISO 15686-1 should be required to read ISO 15686-2 and to demonstrate the connection between their work on factors with part 2? Perhaps it is also time for a mild celebration that service life planning has moved from academic ideal to practical reality in the last decade, and to encourage researchers to seek practical experience of service life planning, and to use that experience to identify an agenda for service life planning research for the next decade

REFERENCES

Aarseth L Hovde PJ (1999) A stochastic approach to the factor method for estimating service life. 8th International Conference on Durability of Building Materials and Components, Vancouver, Canada

Architectural Institute of Japan (1993) The English Edition of Principal Guide for Service Life Planning of Buildings, Architectural Institute of Japan, Tokyo 38PP

BSI (2003) BSI Code of Practice for Slating and tiling including shingles BSI UK ISBN 0580 417409

Bourke K and Davies H (1997) Factors affecting service life predictions of buildings: A discussion paper. BRE Laboratory Report 320 Garston

Construction Audit Ltd (1999) HAPM Component Life Manual EF SPON 376PP

Construction Audit Ltd (2003) HAPM Building Services Component Life Manual EF Spon pp?

ISO 15686 (2000) Buildings and constructed assets- service life planning Part 1: General Principles. Geneva 59pp

ISO 15686-2 (2003) Buildings and constructed assets – service life planning – Part 2 Service Life Prediction Procedures Geneva 32pp

ISO 15686-3 (2002) Buildings and constructed assets – service life planning – Part 3 Performance audits and reviews Geneva 38pp

Moser K (2003) Design Methods or Service life planning Cib W080 Durability in the building process Milan Polytechnic di Milano 12pp

The Performance Based Building Network Concepts & Operation

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ABSTRACT

Building (PeBBu) network. It gives an overview of the program, components, activities of the network so far and possible opportunities for future development. CIB has traditionally played a crucial role in the development of the performance concept and in its practical applications in the development of standards and regulatory requirements. With the establishment of the PeBBu project, the development of the performance concept has taken on a new dimension of practical approach and involvement of a large variety of stakeholders internationally.

Since its conception, the project has grown to include aligned activities such as two compendia projects and involvement with several CIB commissions. Extension into the Newly Associated States (NAS) countries and recent launch of the Australian network of PeBBu are examples of the fast spreading phenomenon of the performance approach. This paper also provides the status of PBB across the PeBBu member and some other pertinent countries. The PeBBu programme of work includes 6 scientific domains out of which domain 1 is active in addressing the issue of life performance of construction materials and components. The key accomplishments of this domain are part of the paper. Activities and outputs that engage and benefit various stakeholders are described and key accomplishments are cited. There are still many hurdles to face and many challenges to encounter. This paper concludes with the major priorities of the PeBBu network in its last phase of operation and then some beyond.

KEYWORDS

Performance Based Building, International Status of PBB, PBB Domains

PERFORMANCE BASED BUILDING: CONTEXT & RATIONALE

The process of designing, planning, coordinating and constructing a building involves an entire spectrum of building professionals who work cohesively together to address the needs of the client. However, this process is not so straightforward. Construction professionals, who usually have never met before, have to operate within somewhat unclear processes and procedures (that change from project to project). The way in which the building fulfils the expectations of all the stakeholders, including those who provide, manage and occupy the building, is increasingly becoming the measure of its success.

The design not only has to be buildable (in terms of cost and time), but stakeholders are increasingly enquiring about its maintainability, sustainability, accessibility, fire deterrent features, and its functional acoustic, energy and the performance. Each of these parameters has to satisfy a whole host of social, economic and legislative conditions. Traditionally, these conditions are governed by prescriptive codes and standards. Prescriptive standards are easy to understand and follow and the results are easy to monitor. However, their inherent inflexibility stifles innovation, leading to a poor match between true user requirements and the building, and poor value for money. Under the '*the customer is always right'* maxim that we take for granted for almost every product and service we purchase, such as cars, computers, luxury items etc. Why do we put up with the fact that it does not apply to buildings and constructed assets? By changing the focus from the input material's specifications (traditional, prescriptive approach) to the output user requirements (performance based building) we will increase both the quality and the long-term value for money of our buildings. (Prior & Szigeti, 2003)

The performance based building (PBB) concept provides a flexible and technically non-prescriptive framework for building design and construction. The PBB approach enables greater innovation, aids international trade and cost reduction. Its application consists of translating human needs (functionality, comfort, etc) first into functional and then into technical performance requirements, implementing them within a regulatory framework and enabling the construction of buildings that provide long-term satisfactory performances. The Performance Based Building (PBB) concept applies itself to the constructed asset planning, programming, design, procurement and construction, life cycle management and operation, and to building regulation control. Although its definition is acutely debated, it is considered in broad terms as "***the practice of thinking and working in terms of ends rather than means***" (CIB, 1982).

ESTABLISHMENT OF THE PEBBU THEMATIC NETWORK

Recognising the relevance of the performance based building concepts for the building and construction sector of Europe, the PeBBu Thematic Network was established in 2001 and runs until 2005 to further the knowledge, dissemination and application of the PBB concept worldwide. It is funded under the European Commission's (EU) 5^{th} framework – Competitive and Sustainable Growth

Organisations from both EU and non-EU countries involved in PeBBu are research institutes; research funding organisations; universities; architectural and engineering offices; contractors; manufacturers; regulatory bodies, building and construction consultants, industry associations and governmental agencies.

Objectives of the PeBBu network

The main objective of the PeBBu Network is ***"Stimulation and pro-active facilitation of international dissemination and implementation of Performance Based Building in building and construction practice",*** and in that context to maximisation of the contribution to this by the international R&D community.

The Network aims at combining fragmented knowledge in the area of Performance Based Building in order to build a systematic approach towards innovation of the building industry and applying user requirements throughout the building process. From this, white spots and a coherent future research agenda can be derived. End-users, policy makers, building industry and regulatory communities are closely involved in this development in order to facilitate dissemination and implementation of research results.

Components of the PeBBu Network

The PeBBu programme1 includes the following "core" components (also refer to Figure 1)

- International programming / coordination of research within 6 Scientific Domains
- Involvement of target groups / stakeholders through 3 User Platforms for respectively a) Buildings Owners, Users and Managers, b) Building and Construction Industry and c) International Standardisation and Conformity Community
- 4 Regional Platforms in Europe to act as the bridge to and the initiator of aligned national activities (Northern, West/Central, East and Mediterranean)
- Network Management - Establishment of a Network Steering Committee, a Technical Committee and a Network Secretariat.
- Mapping of national and international research related to various aspects of Performance Based Building

At the onset of PeBBu, the project began with 9 scientific domains2. These spanned across the various themes and aspects of performance based building. Recently, it has been decided to terminate 3 of

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[1] More detailed information on the PeBBu Network, its program of activities and its organisation can be found in the designated PeBBu website at http://www.pebbu.nl. This website contains an online database with information on current PeBBu member organisations, all ±250 persons active in the various tasks, task description, meetings and publication; More information on the established Scientific Domains with downloadable PDF versions of recent Domain Reports; 1st International State of the Art Report on PBB; Newsletters; International Research Mapping Database

[2] Domain 1: **Life Performance of Construction Materials and Components,** which is investigating the performance of a material or component over its design life and predicting the service life, given many variables.
Domain 2: **Indoor Environments,** which deals which the performance criteria and evaluation methods for healthy buildings.
Domain 3: **Design of Buildings** defining the user requirements in performance terms, implementation of knowledge and training needs for professionals.
Domain 4: **Built Environment** providing and interface between building and urban design.
Domain 5: **Organisation and Management** with the aim of managing the design, construction, operation and maintenance of a building using the performance concept.
Domain 6: **Legal and Procurement Practices,** which deals with legal issues of defining building quality in performance terms.
Domain 7: **Building Regulations** involves implementation of PBB regulatory systems & the role of national/international standards
Domain 8: **Innovation,** which establishes connections between innovation building life cycle phases using comparisons with other more innovative industries.

these domains (the domains on Built Environment, Organisation and Management, Information and Documentation) due to the vast scope of these domains, the lack of ongoing research and/or overlap with other domain work.

Meanwhile, in the first 2 years of PeBBu Programme, some other topics of interest relevant to PBB arose. These have now being developed as new tasks and are incorporated within the PeBBu workplan. The 3 new tasks are:

1. PBB & Construction Products Directive (CPD)
2. PBB Decision Making Tool-Kit
3. (CRISP) Sustainability Indicators for PBB.

In addition to these core components, there are various aligned activities in support of PeBBu (Figure 1). These however, are not funded from the EU PeBBu budget. These aligned activities have been contributing to major achievements of the PeBBu Network.

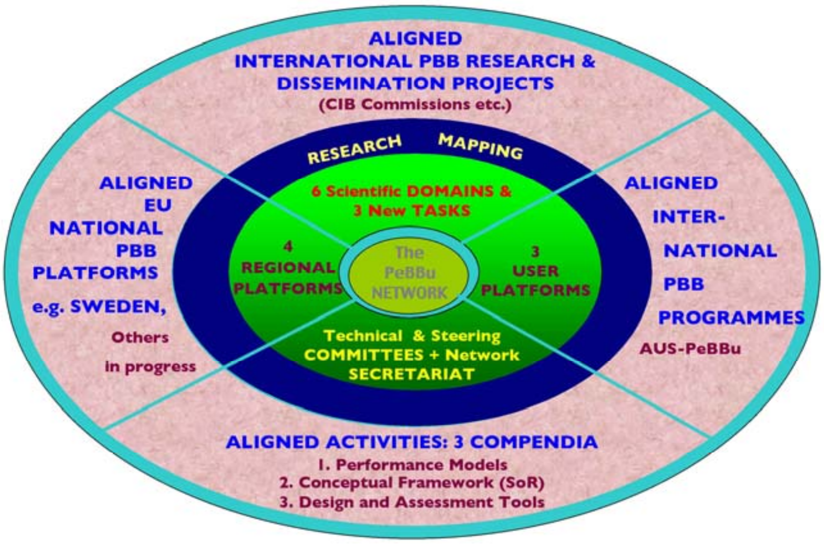

Figure 1: Structure of the PeBBu network

Key Achievements to Date

The PeBBu Network has made considerable progress in its first two years of operation. Some of the main achievements of the 1st two years of the existence of the Network are:

Operation as a Network:
The foremost aim of the PeBBu project is to be operational as a Thematic Network. In that aspect, the network is a big success with the network in place and regular meetings held to strengthen this network.

Expansion of the Network through Newly Associated States (NAS) and observer-members.
This entails the expansion of the Network to include several new countries from Eastern Europe. This ensures a complete European perspective for the stimulation and establishment of Performance Based Building practices. 13 new organisations from the NAS countries are now members of the PeBBu Network. Apart from this, several observer-members have become a part of this Network.

Launch of Aus-PeBBu.

Domain 9: **Information and Documentation** with an emphasis on the new information requirements needed to support the performance based approach.

An Australian version of the PeBBu Network has been launched in October 2003 in Australia. The Australian programme, referred to as Aus-PeBBu, is similar in structure to (EU)PeBBu. With the launch of Aus-PeBBu, Australia is now able to participate in the global move towards the performance approach, which has occurred in Australia and elsewhere through regulatory changes from a prescriptive to a performance based building code. It is expected that Aus-PeBBu will expand to include countries in the Pacific Rim and South East Asia with which Australia and cooperation agreements in place (including New Zealand, China, Malaysia, Vietnam, Hong Kong, Singapore, Indonesia and India). More information on Aus-PeBBu can be found on: www.auspebbu.org

Establishment of aligned activities such as the PeBBu Compendia.
The Compendium of PBB models developed a database that at present includes 30 such PBB models. PBB Compendium of Statements of Requirements aims for the development of a consensus based PBB conceptual framework and key-terminology.

Production of the 1st International State of the Art Report,
This gives an overview of the status off PBB in an international context. The International SotA analysis the spread of PBB principles through many National SotAs from the European context and a secondary research about proliferation of PBB principles in other parts of the world. The International SotA is being published as a CIB publication and this will be a good tool to disseminate vital information on PBB. This work has been detailed out in the next section.

Establishment of many strategic relationships.
Examples of these are:

- Between PeBBu Domain 1: Life Performance of Construction Materials, Components and ISO. This relationship has influenced writing of standards related to durability of construction materials and components;
- Co-operation with ISO TAG8 (the ISO Technical Advisory Group that is responsible for building related standards) on a multi-year programme within ISO that aims for the production of performance based standards that are to replace or to be added to the current prescriptive ones
- Other strategic relationships including PeBBu and aligned activities have influenced new work in the Indoor Environment area;
- PeBBu has also established relationship with the E-CORE projects where PBB will be one of the main building blocks in a future European RTD strategy.
- Co-operation with the Liaison Committee's of International Associations of Structural Engineering aiming for the establishment of a joint committee on PBB related pre-standardisation issues in the area of structural engineering

Consensus on PBB language, concepts and issues.
This is mainly a result of the PBB compendium 3 on Statement of Requirements.
Involvement with / and support of several CIB Commissions.
Several CIB (Task Groups and Working) Commissions have been established to facilitate international exchange and co-operation in areas that cover aspects of PBB on a voluntary basis. As an average each such commission incorporates 50 appointed representatives of organisations worldwide who meet on a regular basis and aim for joint, voluntary, international R&D projects in their area.

STATUS OF PERFORMANCE BASED BUILDING

In this section, the international status of Perfomance Based Building is given. A status of PBB as part of the PeBBu Domains is also given later in this section.

International Status of PBB

Over the last 50 years, there has been considerable development with the performance based approach. There have been numerous worldwide activities to develop and apply the performance concept in building. This section lists some of the highlights of PBB uptake worldwide.

- The **Public Buildings Service (PBS) of the General Services Administration (GSA)** funded the National Institute of Standards and Technology (NIST, previously the National Bureau of Standards) to develop a performance approach for the procurement of government offices, USA.
- The **UK building regulations** (introduced in the 1960s, and have since been regularly updated) are predicated on a performance basis, and are supported by a series of Approved Documents in which helpful guidance is given in the form of deemed-to-satisfy examples. More recent developments include the BAA project process, the Process Protocol of the University of Salford, the Ministry of Defense Prime Contracting method, and the Office of Government Commerce's (OGC) Gateway Process.
- In 1970, the International Council for Building Research Studies and Documentation (CIB) set up a **CIB Working Commission W060** to investigate the performance concept in building
- May 1972 saw the first international symposium on the 'Performance Concept in Buildings' by **RILEM-ASTM-CIB, Philadelphia, USA**. Several symposiums and conferences have since followed.
- The Nordic nations, Denmark, Finland, Iceland, Norway and Sweden, have set up a **joint Nordic Committee on Building Regulations (NKB).** In 1976, NKB developed the Nordic Model for the development of performance based building codes and standards, which are a hierarchy of necessary features for the development of performance based building codes.
- In 1988, the European Commission developed its **Construction Products Directive (CPD)** containing six essential elements for buildings and components, all expressed in performance terms. The Construction Products Directive is supported by structural standards known as 'Eurocodes' prepared by the European Committee for Standardisation (CEN). The Eurocodes are harmonised across the member states. They are applied with the support of a series of 'Interpretative Documents' and a system of European Technical Approvals issued by members of the European Organisation for Technical Approvals (EOTA)
- The 1997 **World Trade Organisation (WTO) Agreement on Technical Barriers to Trade** states that '…wherever appropriate, members shall specify technical regulations based on product requirements in terms of performance rather than design or prescriptive characteristics.'
- In 1982, the **Dutch National Coalition Agreement of the Lubbers cabinet** initiated a white paper, proposing a drastic deregulation of the building industry and a substantial reduction in the amount of building regulations. As a result in October 1992 a new regulation, called the Dutch Building Decree, which is performance based, was implemented and is yearly evaluated. It has evolved into the present Building Decree of 2000, which serves the purpose of deregulation.
- The concept of PBB was introduced in Netherlands by the **Government Building Agency** (GBA). However, since it was launched in the early 90s, the concept behind it had significant influence on the Dutch Public Building Regulations and Codes which in their actual form wouldn't have been possible without PBB.
- In the USA, **performance based contracting** is mandatory. The USA Federal Acquisition Regulations 2000 state that '…performance based contracting means structuring all aspects of an acquisition around the purpose of the work to be performed, with the contract requirements set forth in clear, specific, and objective terms with measurable outcomes as opposed to either the manner by which the work is to be performed or broad and imprecise statements of work.'
- The **Building Code of Australia** (BCA) and the **New Zealand Building Code** (NZBC) have developed performance based regulatory systems.

- In 2001, the thematic Network **Performance Based Building Programme (PeBBu),** funded by the European Commission, was set up to combine fragmented knowledge in the area of PBB into a systematic approach towards the innovation of the building industry

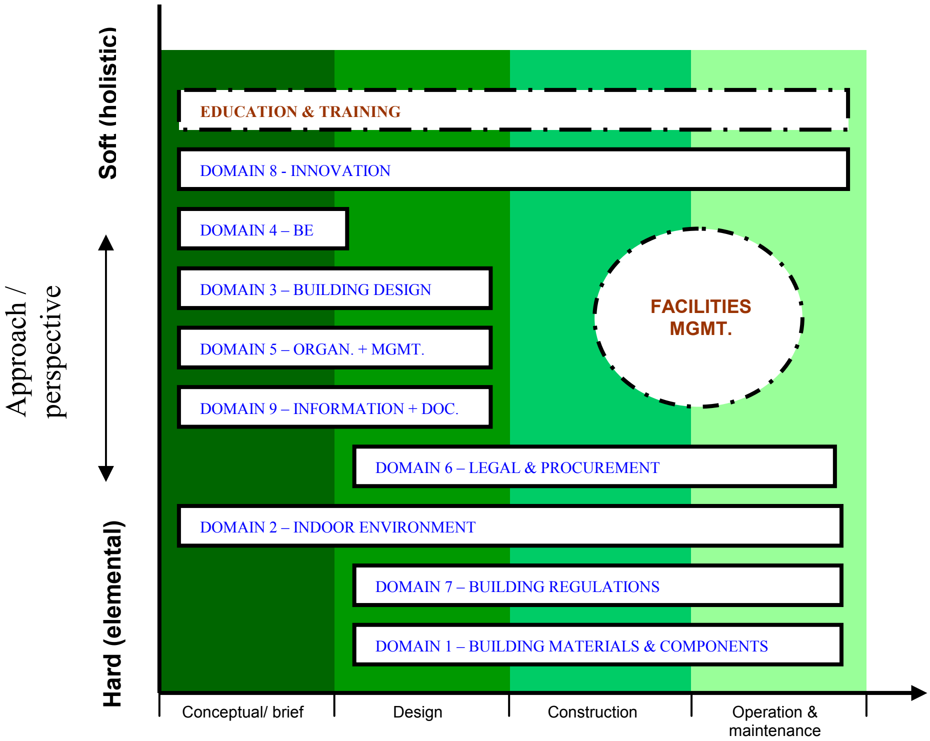

Figure 2: Lifecycle stages

Despite these leading programmes, implementation of the performance concept has been somewhat sporadic.

Status of PBB Domains

Although PBB is whole encompassing of the design, operation and maintenance of a building's lifecycle, under the PeBBu Network it has been divided into 9 (six continuing) scientific domains to aid the investigation process. These domains span the complete lifecycle of a constructed facility - from the conceptual/ brief stage through to operation and maintenance (see Figure 2). Moreover, they encroach on both soft and hard issues facing the design and construction process. The potential new activities of FM (facilities management) and education occupy definable "gaps" at this moment. This section summarises the state-of-the-art of the performance based approach in each of the 6 remaining domains of PeBBu and provides a brief future research agenda. (PeBBu domain reports, 2004)

Domain 1: Life Performance of Construction Materials and Components

The overall performance of a building relies on the performance of its materials and components. Therefore, how can the performance of materials and components be assessed in advanced to ensure the building performs as required? PeBBu domain 1 aims to address methods for predicting service life, particularly for novel building materials and components. Under PeBBu programme, this domain will further develop the Factor Method (system to estimate the service life when there is limited knowledge of long-term performance of components), develop an international suite of standards, and to design and prepare training sessions for both industrialists and academics.

Domain 2: Indoor Climate

This domain maintains that healthy buildings can be pursued by designers, constructors, building owners and building occupants, through the application of a number of qualitative and quantitative health-based criteria (methods, guidelines, protocols and tools to design, evaluate and measure the health status of buildings/ designs). The health of buildings in this context relates to air quality, ventilation, thermal comfort, noise and visual comfort. Although there is rich scientific literature and several experiences on the quality of the indoor environment, a uniform set of criteria across Europe or the world has not been defined. The majority of PBB implementation is isolated to different components and not in terms of the building as a whole. A general translation from subjective criteria to objective design parameters, and reverse when dealing with the evaluation, to a large part is still lacking. Research initiatives into the health and comfort of the building environment are ongoing. However, a lot of work is still required before PBB can completely replace current prescriptive buildings methods.

Domain 3: Design of Buildings

It has now become an economic necessity for the building industry to pay more attention to meeting user requirements. Therefore, there should be a focus on both the (technical) performance specifications for building parts, and on the management of (functional) user requirements and involvement during the building process. Thus, the problem of 'meeting with performance specifications' in the design stage of the building process should be addressed: -

- The translation of client and user requirements into objective, measurable performance specifications.
- Classifications and formats for performance specifications.
- The testing of (preliminary) design results against agreed performance specifications.

PBB in the design of buildings has mainly been undertaken in research and education rather than practice. In several countries there are programmes aimed at structural changes in the building industry. Examples are ‘Rethinking Construction’ in the UK and the ‘Process and Systems Innovation Programme for the Building Industry’ (PSIB) in the Netherlands. The primary barrier for performance based design of buildings rests with the culture and existing fragmentation of the construction industry. In retrospect, linking performance based design with IT (product modelling systems), demonstrating its value in educational programmes and developing a universal language of specifications, could speed its adoption.

Domain 6: Legal and Procurement Practices

This domain focuses on the problems encountered towards PBB specifications via procurement and the legal issues that subsequently arise. There is currently no state-of-the-art that could be applied across the EU, rather a collection of national practices. There are two significant factors driving towards PBB, international and government influences. International influences have arisen from experiences of multi-national companies around the world and a desire to replicate best practice in other countries. Government influences have been founded on their responsibility for construction output, and that they need to maintain or increase output whilst at the same time reducing public sector expenditure. In order to reconcile these two forces, governments have increasingly turned to methods that involve private finance in projects. These methods include: Design and Build (D&B); Design Build Fund Operate (DBFO); Build Operate Transfer (BOT); Build Operate Own Transfer (BOOT); Private Finance Initiatives (PFIs); Public Private Partnership (PPP). Currently it is reported that in excess of 100 countries are procuring construction and engineering works under the generic heading of PFI. The majority of uncertainty lies with risk and liability, not least the issue of duty of care v duty of result. Both private and government influences have resulted in moves towards PBB since performance specification lies at the heart of both D&B and PFI philosophy.

Domain 7: Building Regulations
Innovation in construction is heavily dependant on the building regulatory system. In many countries, the system is based on what is termed a 'prescriptive' approach, where a single, or very few solutions are provided as ways to comply with building regulations. This has the effect of creating a design and construction industry that is restricted to designs that fit those specific solutions. Performance regulations, which focus on intended outcomes, are intended to encourage innovation and trade by expressing what regulations are intended to achieve. Many countries are moving in this direction or have already implemented PBB building regulations. However, there is very little research in the area of performance based regulatory system issues on a policy level. Research, in most cases, focuses on the technical solutions to the regulations. On a national level the Australian Building Codes Board has been fairly active in conducting research that relates to the building regulations. More specifically, Australia formed what is called the 'Fire Code Reform Centre' in 1994 to look at various issues relating to fire safety in an effort to improve the building code. Countries such as France and Canada also conduct similar research in various areas. In the United States, there has been some research in the area of risk and public policy as it relates to building regulations.

Domain 8: Innovation
The principal focus of PeBBu Domain 8 is to examine innovation through the 'performance of buildings in use.' Innovation is taken to concern the multifaceted value creating, capture and delivery role of buildings. This includes the building appropriately meeting the needs of the client system and of whole life cycle performance. Currently, there is a mass of technical, product-related criteria (principally from a top-down regulatory direction) that helps maintain a base-line standard of construction for health and safety purposes. This does not cover all areas of construction, such as the process of construction, or many aspects of the use of buildings and their impact on society. Both the EU Innovation Scorecard and the UK's 'Key performance indicators' do not reflect any indicators in the area of innovation and learning. Nor do they focus on the client's/ user's needs. Performance metrics for innovation in PBB thus need to appropriately integrate building and business considerations. For the industry to succeed it needs to maintain a minimum level of innovation as a norm and this should be reflected in explicit innovation performance objectives for the various parts of the industry. The challenge for the domain will be to identify clear and realistic innovation objectives and measures, differentiated as appropriate by industry sector, which can make a real impact in practice. Part of this aspect will include consideration of the interactive roles of the various stakeholders running from materials suppliers, through builders merchants, designers, specialists and builders, to users and owners of the built facility.

FUTURE CHALLENGES & MAIN PRIORITIES

Performance based regulations and practices are currently both mandatory and voluntary. Culture and a lack of resources have somewhat prohibited the regulation bodies to fully 'prescribe' PBB practices. PBB must be contextualised to suit a particular country and lessons learnt from other PBB applications must be reviewed and adapted to differences in government, culture, and economy. Successful application of PBB requires the development of new knowledge, understanding and PBB tools.

Prescriptive-based building sometimes works well in simple deemed-to-satisfy buildings, thus, the application of PBB principles is not always necessary. Therefore, the two approaches should work in harmony: the performance approach should be available to those who require more complex innovative developments or design concepts driven by stakeholder requirements.

In order to obtain the maximum benefits of PBB, the following has to be delivered: -

- Actions to progressively align organisations business processes to PBB

- Education and training regarding the appropriate circumstances for using PBB
- The steady increase of the performance criteria captured within PBB

Other framework issues that need to be addressed to support the PBB concept include public policy, regulations, innovation (in various fields such as materials, building technology, design and construction process etc.). In particular, developing assessment methods that verify the approach to support the regulators, methods that allow the client to ensure that their performance requirement is actually delivered, and guidance of how to include PBB in an organisation's business strategy are crucial for the assurance of both the client and construction professionals.

The PeBBu project expects a further increase in stakeholder involvement in various aspects of PBB. However, specific methods need to be explored to engage effectively the critical stakeholders that can help hasten and broaden the adoption of the performance concept. Therefore the challenges lying ahead of the project are to determine the value and benefits of performance based building for different stakeholder groups, substantiated with real case studies, and to package them for a compelling presentation. This will ultimately facilitate client- or demand-driven innovation, made possible by the performance approach.

REFERENCES

CIB Publication. 2003, *Performance Based Building – 1st International State-of-the-Art Report*, Publication 291, CIB, Rotterdam, The Netherlands.

CIB Publication. 2002, *Measurement and Management of Architectural Value in Performance-Based Building. Report of working commission W60/W96*, Publication 283, CIB. Rotterdam, The Netherlands.

CIB Publication. 1982, *Working with the Performance Approach in Building. Report of Working Commission W6*0, Publication 64, CIB, Rotterdam, The Netherlands.

Foliente, G. 1998, *Facilitating Innovation and World Trade – the CIB Pro-active Programme in Performance Based Building Codes and Standards*, CIB Information Bulletin 2/ 98, Rotterdam, The Netherlands

Prior, J.J. & Szigeti, F. 2003, *Why all the fuss about performance based buildin*g?, CIB Bulletin. Available from http://www.pebbu.nl/resources/literature/

PeBBu Domain Reports (2004) Available from *http://www.pebbu.nl/maincomponents/scientificdomains/*

PeBBu Website: *www.pebbu.nl*

A Total Cost Accounting Approach in Design and Evaluation of Products

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ABSTRACT

To meet the requirements on sufficient functional capability, service reliability and minimum environmental impact and cost in product design and evaluation, a total cost accounting approach may favourably be adopted. Most appropriate product design alternative is the one with lowest total cost, the latter determined by the sum of development cost, production cost, cost associated with initial non-ideal function or performance, maintenance cost, possible cost of failures, end of life cost, and possible cost associated with ecological damage.

To apply the total cost accounting approach requires the conduction of a systematic suitability analysis of potential product design alternatives by making use of methodologies from lifetime technology and industrial ecology. Although it may not be possible to estimate a cost for all terms contributing to the total cost of a specific product design alternative, the result of a total cost exercise may, despite that, form a good base for selection of the best design alternative with respect to used materials, manufacturing, maintenance and recovery. The main purpose with the suitability analysis is to achieve a framework for compiling and integrating all data on available materials and product properties, environmental stress characteristics of application, product reliability and materials degradation, environmental problems and ecological risks associated with the production, use and end-of-life of product, cost characteristics for production, for use and for end-of-life of product.

The result of a case study in which two different selective solar absorber coating products are compared, one with an electrochemically produced absorber coating and the other with a sputtered solar absorber coating, has shown that the total cost accounting approach is possible to realise also in practice. Tools have been identified that enable all the essential factors for product design to be expressed in cost terms, each contributing to the total cost associated with maintaining the prescribed product function for a fixed service time. However, the important point is not to obtain an exact value of the total cost but to obtain a measure and understanding of the relative importance of the various factors contributing to the total cost and to be able to make comparisons in the cost characteristics between different product design alternatives.

KEYWORDS

Product design, total cost, functional capability, reliability, lon-term performance, ecological soundness, recoverability, selective solar absorbers

1 INTRODUCTION

In design and evaluation of products with respect to functional quality, cost effectiveness, reliability and long-term performance, ecological soundness and recoverability, a holistic viewpoint should be employed in the very first phases of design work.

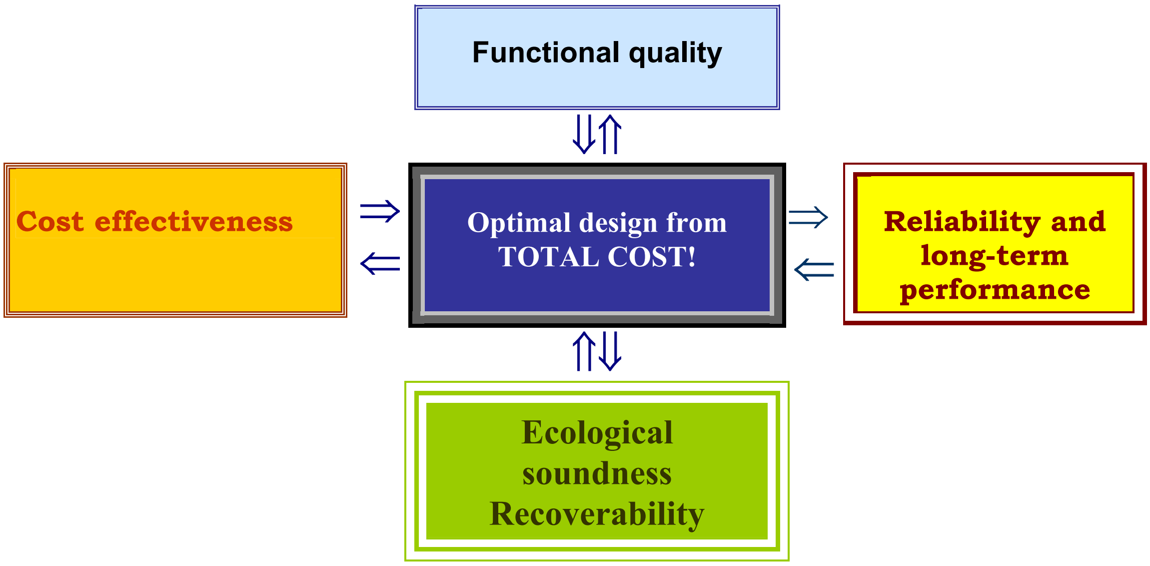

Figure 1 Factors dictating the optimal design of a product from a total cost point of view

A total cost accounting approach may therefore favourably be adopted in product design to rank a series of design alternatives for a functional unit of the product by making use of a suggested model originally described by Carlsson et. al [2001]; see Fig. 1. The total cost for each alternative is estimated by compiling the contributions from all the various factors that may be of importance for the suitability of the design alternative considered. However, the point of departure is not a particular functional unit and its life cycle but the intended function of the unit over time.

For a fixed service time, τ_s , the total cost (C_T) associated with maintaining the specific function defined for the functional unit is estimated from

$$C_T = C_P + C_{NIP} + C_M + C_F + C_{EoL} + C_E + C_D \qquad (1)$$

where
C_P = Production cost
C_{NIP} = Cost associated with initial non-ideal function or performance
C_M = Maintenance cost
C_F = Cost of probable failures and damage
C_{EoL} = End of life cost
C_E = Environmental cost associated with probable ecological damage
C_D = Development cost.

The production cost, C_P, shall be estimated for the installed functional unit in the product.

The cost associated with initial non-ideal function or performance, C_{NIP} , shall be defined relative to some reference functional unit. It shall be possible to relate to a decrease in the product or system performance and associate it with increased service costs, for e.g. energy, excess weight, material consumption, etc.

The maintenance cost, C_M , includes all the different kinds of active measures taken to maintain the specific function defined for the functional unit during the fixed service time. Those may thus sometimes include replacing the functional unit regularly with a new one during the fixed service time, if the service life of this unit is less than the fixed service time.

The cost of probable failures or decrease in functional performance, C_F , may be expressed as a sum of contribution from the most critical failure or damage modes, i.e.

$$C_F = \sum_{i=1}^{N} (C_{F,i} \cdot F_i(\tau_s)) \tag{2}$$

where
$C_{F,i}$ = Cost of failure by mode i
$F_i(\tau_s)$ = Probability of (or population fraction) failing by mode i during the fixed service time τ_s.

$F_i(\tau_s)$ is a function of both environmental stress and properties of the functional unit. The service life of the different individuals of the functional unit of the product may vary due to the fact that the different individuals are exposed to varying service stress conditions. But, it may also vary due to varying properties of the individuals. To estimate the probability $F_i(\tau_s)$ would require the distribution in stress level for the different individuals of the functional unit is known and also the distribution in the properties of the functional unit between its individuals.

The end-of-life cost, C_{EoL} , is the cost that arises when the fixed service time has passed. The whole product may then either be re-used or dissembled. The different functional units of it may thus either be re-used or dissembled and their materials recycled or disposed as waste. The End-of-life cost of a functional unit may therefore be positive or negative dependent on the value of the functional unit after the fixed service time has passed.

Costs dictated by environmental considerations may be of two categories: direct costs and indirect costs associated with probable ecological damage. Direct costs comprise contributions from e.g. use of cleaner materials in the functional unit and use of special equipment during production to reduce environmental impact, the cost of planned maintenance in the service phase and the cost of equipment required for the end of life phase of the functional unit. Those costs are therefore parts of the production cost (C_P), maintenance cost (C_M) and end-of-life cost (C_{EoL}), respectively.

The indirect environmental cost associated with probable ecological damage (C_E) may be estimated by applying a similar approach as used to express the cost of probable failures, i.e.

$$C_E = \sum_{j=1}^{M} C_{E,j} \cdot F_j(\tau_s) \tag{3}$$

where
$C_{E,j}$ = Cost of possible environmental damage j
$F_j(\tau_s)$ = Probability for environmental damage j to occur during the fixed service time τ_s.

To apply the total cost accounting approach requires the conduction of a systematic suitability analysis of potential design alternatives of a functional unit of a product by making use of methodologies from materials lifetime technology and industrial ecology.

The suitability analysis comprises three different kinds of analysis related to functional capability, reliability and long-term performance, end-of-life system and environmental impact; see Fig. 2

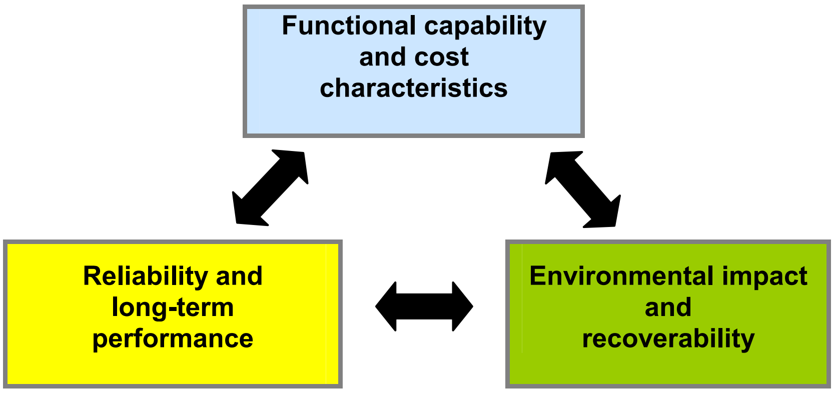

Figure 2 Three different kinds of analysis are needed in the evaluation of the suitability and total cost of a specific design alternative for a functional unit

The overall working scheme to be employed is quite complex and for more more details the reader is referred to the research report by Carlsson et.al. [2004]. The working scheme is composed of a number of steps of which some are closely interrelated to each other and therefore requires an iterative way of working with the suitability assessment.

The suitability analysis is not only aimed at identifying the design alternative with the lowest cost but also to obtain a checklist of potential risks and critical materials or functional unit properties and costs for the different design alternatives. With the suitability analysis is achieved a framework for compiling and integrating all data on available materials and functional unit properties, environmental stress characteristics of application, functional unit reliability and materials degradation related to different design alternatives of it , environmental problems and ecological risks associated with the production, use and end-of-life of the different design alternatives of the unit, and cost characteristics for production, for use and for end-of-life. The database resulting from a suitability analysis therefore will be of great value also in future product development.

2 SUITABILITY ASSESSMENT BY TOTAL COST FOR SELECTION OF SOLAR ABSORBER SURFACES

Renewable energy technologies, such as solar heating, often place special requirements on the long-term performance of the energy producing systems. Because of the relatively low energy price for today's conventional energy sources, most of today's solar domestic hot water systems, need solar collectors that can function for a long time period, at least 25 years, without significant decrease in their energy efficiency. Besides that, the low power density of the solar irradiation, maximum 1 kW/m^2, place additional requirements on the solar collector system with respect to design and cost effectiveness. As environmental concern is the most important incitement for installing a solar heating system today, the design concept chosen for the solar collector system must also be sustainable in nature.

The most important functional unit in the solar collector system is the solar absorber. The front surface of the absorber should absorb as much as possible of the incident solar radiation with intensity maximum in the spectral wavelength range from 0.3 µm to 3 µm. Moreover, it should not lose energy by thermal radiation meaning that its reflection in the infrared wavelength region from 3 µm to 20 µm should be high. An optimised solar absorber surface, exhibiting a high absorption, low reflection, in the solar range and a low emittance, high reflection, in the thermal range, is called a spectrally selective solar absorber surface.

Selective solar absorber surfaces are typically prepared by depositing a coating, which absorbs the solar irradiation and transmits the thermal radiation, to an IR–reflecting metal substrate or metal coating. Porous anodised aluminium has long been used as a substrate material for selective solar absorbers. The pores of the alumina are partly filled with small metallic nickel particles produced by electro deposition. The empty upper part of the porous alumina forms the anti-reflective layer. However, coatings produced with modern vapour deposition technologies are today considered of special interest as the production can be made cleaner without large quantities of chemical waste and the production conditions made easier to control when compared with the electrochemically produced coatings.

One such sputtered selective solar absorber surface, which may be seen as an alternative to the electrochemically produced nickel pigmented anodized aluminium absorber coating, consists of two layers on a substrate of aluminium, one absorbing base layer next to the substrate and on that one antireflective top layer. The base layer is composed of a mixture of nanosize grains of metallic nickel and of nickel oxide. In the upper part of the base layer the amount of metallic nickel is gradually decreasing and in the front surface only nickel oxide is present [Wäcklegård and Hultmark 1998].

You may ask how favourable the sputtered nickel based absorber coating is when compared with the old electrochemically produced. A suitability analysis with a total cost based comparison has therefore been made and the result nicely illustrates the pros and cons of the two kinds of solar absorber coatings.

The results of the analysis, which is described in full length in the research report by Carlsson et.al. [2004], shall here briefly be reviewed. It should, however, be pointed out that concerning the analysis of the long-term performance aspect, results are also used from a case study on accelerated life testing of solar absorbers performed in Task 10 of the IEA Solar Heating and Cooling Program [Carlsson et.al. 1994] and from a round robin test of solar absorbers conducted by the Working Group Materials in Solar Thermal Collectors within the same IEA program [Brunold et. al. 2000]

Cost estimates (€/m^2)

Design alternative	C_P	C_{NIP}	C_M	C_F		C_{EoL}	C_E (ELU)		C_D	C_{Tot}	
Electrochemically sputtered coating	26.7	68.3	0	81.8 (a)	13.3 (b)	0	294 (a)	157 (b)	-		
Sputtered coating	26.7	53.3	0	4.0 (a)	1.3 (b)	0	110 (a)	105 (b)	-		
Difference in cost between the two coatings	0	15.0	0	77.1 (a)	12.0 (b)	0	184 (a)	52 (b)		92.1* (276.1)** (a)	27.0* (79.0)** (b)

*Excluding C_E ** Including C_E under the assumption that 1 ELU = 1 €

Table 1 Summery of the results from the suitability analysis of the two kinds of solar absorber coatings. (a) refers to the case when the solar absorber is installed in a non-airtight solar collector with uncontrolled ventilation of air and (b) refers to the case when the solar absorber is installed in an airtight solar collector with controlled ventilation of air. The service time is 25 years. The price of energy has been set equal to 0.067 €/kWh. (C_P = Production cost, C_{NIP} = Cost associated with initial non-ideal function or performance, C_M = Maintenance cost, C_F = Cost of probable failures and damage, C_{EoL} = End of life cost, C_E = Environmental cost associated with probable ecological damage, C_D = Development cost)

The results of the suitability analysis of the two solar absorber coatings are summarized in Table 1 and as expected the sputtered coating seems more favourable to the electrochemically produced coating in all respects.

In case of production cost, C_P, the two coatings can be considered equal although the production of the sputtered coating requires a more advanced technique. However, due to the benefits of larger scale of production, better control and more rational handling of the production, the price of the sputtered coating can be kept at the same level as the electrochemically produced coating [Sunstrip AB 2001].

A cost for initial non-ideal performance arise in this particular case due to non-ideal optical performance of the absorber coatings, solar absorptance $\alpha = 1$ and thermal emittance $\varepsilon = 0$ ideally. To transfer the meaning of this non-ideal performance into cost terms, a simple model was used. The solar domestic hot water system is assumed to have a system efficiency of 50 %, which means that within 25 years the amount of solar energy collected and utilised should be 10 000 kWh/m². At an energy price of 0,067 €/kWh this corresponds to 667 €/m². Loss in solar system performance can in this case be related to loss in optical performance of the absorber coating through the function $PC = -\Delta\alpha + 0.25$, as the value of the PC function corresponds to the relative loss in solar system performance [Carlsson et al 1994]. Relative the ideal case of $\alpha = 1$ and $\varepsilon = 0$, a value of the PC function can thus be calculated and the initial non-ideal optical performance translated into cost based on the assumed amount of solar energy collected and utilized and the assumed energy price.

As can be seen from the data in Table 1, the second biggest contribution to the difference in total cost is obtained for the cost of initial non-ideal performance, which illustrates the importance of having a solar absorber surface with optical properties as close as possible to the ideal situation.

The long-term stability of the electrochemically produced nickel pigmented anodized aluminium coating was thoroughly studied by the IEA Task 10 group. Mainly three degradation mechanisms were identified that could lead to an unacceptable loss in optical performance of the absorber coating, namely (1) high temperature oxidation, which is mainly operative under stagnation conditions of solar collector at high levels of solar irradiation and thus when no heat is withdrawn from the collector, (2) electrochemical corrosion of metallic nickel which may result from exposure to high humidity and airborne pollutants under starting-up and under non-operating conditions of the solar collector when the outdoor humidity level is high, and (3) hydration of aluminium oxide from the action of condensed water or moister on the absorber surface during periods when the humidity in the collector is very high.

The conclusion from the results of the accelerated life test program conducted in IEA Task 10 was firstly that the thermal stability was quite satisfactory. In case of resistance to corrosion, the service life with $PC<0.05$ could be estimated to 12 years when the absorber is installed in a non-airtight solar collector. This then corresponds to an average loss of 5 % during the total service time of 25 years, which means that the corresponding cost of reduced performance amounts to 34 €/m^2 . The case taking into account the resistance to condensed water can be treated similarly. From the accelerated life tests results, the service life with $PC<0.05$ was estimated to 9 years and this corresponds to an average loss in system efficiency in cost terms of 47.8 €/m^2

The corresponding cost for probable failures for the absorber with the electrochemically produced coating but when installed in an airtight solar collector was made analogously by assuming corrosion is the only mechanism contributing to the loss in performance in this case. For the sputtered solar absorber, accelerated life test results from the round robin test performed by the IEA Working group on materials for solar thermal collectors were used [Brunold et al 2000] It should however be pointed out that the sample tested in this case was the sputtered coating but on a substrate of stainless steel instead of on aluminium as planned. The durability of the sputtered coating on stainless might be better compared with that of the sputtered coating on stainless steel, because of the higher corrosion resistance of stainless steel. However, the dominating degradation mechanism is assumed to be the corrosion of the metallic nickel particles and how important the corrosion of the substrate is for the durability of the absorber coating is therefore presently hard to tell.

As is clear from the data presented in Table 1, the biggest contribution to the difference in total cost between the two kinds of absorber coatings is obtained for the cost of probable failures C_F in that case when the solar absorber is assumed to be installed in a non-airtight solar collector with uncontrolled ventilation of air. This points to the importance of choosing a collector box of a proper design as regards ventilation so that the humidity and corrosivity loads on the solar absorber can be kept and maintained low during the entire period of service.

The contribution to the difference in total cost with respect long-term performance in the case of installing the solar absorber in an airtight solar collector is considerably lower when compared with the situation when a non-airtight solar collector is used. However, durability may despite that be an important aspect and efforts made to improve durability should be taken with the same priority as improving the initial optical performance.

When comparing the two design alternatives for solar absorber coating with respect to recoverability, the situation is very much the same for the two. The aluminium in the two absorbers can be recovered by recycling through the assistance of the manufacturer of the aluminium used [Sunstrip AB 2001]. The scrap price for the absorber is about the same as the cost for dismantling and transport. The direct end-of-life cost can therefore be set at zero for the absorber coatings considered; see Table 1.

In case of the indirect environmental costs or cost for probable environmental damage, LCA Impact assessment using the SIMA PRO 6 LCA software (see www.pre.nl/simapro), was employed to get a rough indication of the difference that exists between the two kinds of absorber coatings.

Considering first the manufacturing of the electrochemically produced nickel pigmented anodized aluminium coating, the environmental impact expressed by the EPS ecoindicator amounts to 6 ELU/ m^2 solar collector area. The corresponding figure for the sputtered coating was estimated to be around 4 ELU/m^2 solar collector area.

The importance of the environmental impact contribution from the manufacturing of the absorbers to the total environmental impact of the solar absorbers is however relatively small. This becomes evident when taking into account the supply of extra energy, which is needed during the service phase for two reasons. Firstly, extra energy is needed to compensate for the difference in initial non-ideal optical performance between the two absorber coatings. Secondly extra energy is needed to compensate for the in-service degradation in performance of the absorber coating which will occur.

Consequently, main contributions to the difference in the cost for probable environmental damage between the two kinds of solar absorbers are associated with differences in functional capability and thus reduced capability to use solar energy for the intended function of domestic hot water production. If the EPS indicator ELU is converted into cost, ECU or € as originally meant, the cost for probable environmental damage will be twice of the total cost excluding this cost !?.

Although the main purpose of the suitability analysis in this case was to compare two kinds of solar absorber coatings, the result of the analysis clearly illustrates the environmental benefit of replacing energy from conventional energy sources with solar energy. If this is taken into account, the reduction in the cost of probable environmental damage by producing domestic hot water is in the order of 1000 ELU/ m^2 solar collector area.

3 CONCLUSION

The result of the suitability analysis of the two solar absorber surfaces excellantly illustrates the benefits of the total cost accounting approach in the design of functional units for products.

That the analysis could be made at such a high quantitative level in this case is thanks to the comprehensive research conducted on selective solar absorbers within the framework of the IEA Solar Heating and Cooling Program. However, as has been mentioned the important point is not to obtain an exact value of the total cost, but, to obtain an understanding of the relative importance of the various factors contributing to the total cost. This means that is most often only the order of magnitude of the different terms contributing to the total cost that may be needed in the evaluation of the best design alternative of a functional unit of a product in the early stages of design work.

4 REFERENCES

Brunold S., Frei U., Carlsson B., Möller K., Köhl M.; Round robin on accelerated life testing of solar absorber surface durability", *Solar Energy Materials & Solar Cells*, **61** (2000)239

Carlsson B., Frei U., Köhl M. and Möller K., *Accelerated Life Testing of Solar Energy Materials – Case study of some selective solar absorber coating materials for DHW systems*, International Energy Agency Solar Heating and Cooling Programme Task X Solar Materials Research and Development, SP Report 1994:13, ISBN 91-7848-472-3

Carlsson B., Virtanen Y., Nøkleby J. O., and Sandström R. 2001, *A Holistic Approach to Materials Selection in Component Design;* Nordtest Report 476, www.nordtest.org

Carlsson B. et. al. *Design of Functional Units for Products by a Total Cost Accounting Approach* ; Report for Vinnova project 24291-1, Department of Technology, University of Kalmar , SE-39182 Kalmar, Sweden, (to be published 2004)

Sunstrip AB Private communication 2001, www.sunstrip.com, and Jan-Olof Dalenbäck, Chalmers Technical University, Gothenburg, Sweden

Wäckelgård E. and Hultmark G., Industrial sputtered solar absorber surface, Solar Energy Materials and Solar Cells 54 (1998) 165-170

An Integrated Lifetime Management System for Civil Infrastructures

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ABSTRACT

In our country, because there are a huge number of civil infrastructure systems, it will be becoming a major social concern to develop an integrated lifetime management system for such infrastructures in the near future. Then we need to develop an innovative system for long-term lifetime management engineering as Doctor for Infrastructure Systems. Namely, it needs to develop an integrated lifetime management system for civil infrastructure systems combined with the latest information processing technologies and intelligent health monitoring techniques, and also to establish the Doctor Degree for Civil Infrastructure Systems (INFRADOCTOR). This paper describes our Japanese strategy of life-cycle management in civil infrastructure systems for the 21st Century as a Center of Excellence (COE) Project in Japan.

KEYWORDS

Civil infrastructure, Integrated lifetime management, Health monitoring, Information technology, Prestressed concrete (PC) bridge.

1 INTRODUCTION

Our country, Japan is now a country with highly developed social infrastructures in both qualitative and quantitative terms. They include urban expressway networks and other social facilities and such structures as bridges, dams, tunnels, etc. It will therefore be necessary as in Europe and the United States to maintain the service life (performance & structural function) of social stock as long as possible and take appropriate measures as it ages, in harmony with the natural environment [Miyamoto & Frangopol [2002]]. The process is generally referred to as maintenance. Engineers equipped with state-of-the-art information technology need to be developed to work as doctors diagnosing (evaluating and assessing) and treating (repairing and strengthening) structures including prestressed concrete (PC) bridges, that is establishment of diagnostics. A world standard computer system must also be developed that supports engineers and helps them inherit knowledge and experience because maintenance requires large amounts of knowledge and experience. PC structures are larger and often need to work longer, more than 100 years, than non-engineering products. They are subjected to diverse types of deterioration mechanism such as corrosion, fatigue, carbonation, alkali-aggregate reaction, etc. Detecting deterioration and deformation as early as possible for maintenance may require health monitoring equivalent to in-home medical care services for humans.

This paper introduces a concept of strategic capital stock management (integrated life-cycle management system) (see Figure 1) and presents a management system that was developed for a PC bridge [Miyamoto [2002]; Miwa [2001]] as a specific example. The life-cycle management system incorporates life-cycle cost (LCC) and the concepts and analysis methods for management into the field of maintenance that used to be considered somewhat low-key as compared with the construction of structures. The system is an organic integration of studies in a wide range of academic fields including systems, electrical and mechanical engineering fields beyond the scope of civil engineering field. It is an environmentally friendly system aided by state-of-the-art information technologies such as network-based databases systems, multimedia virtual reality, intelligent monitoring, artificial life and artificial intelligence.

2 HEALTH MONITORING AND MANAGEMENT USING INFORMATION TECHNOLOGY (IT)

For example, it will be focusing on prestressed concrete (PC) structures including PC bridges, PC structures are more effective than reinforced concrete (RC) structures because prestressing the cross section of a member eliminates an unfavorable stress condition that is created by external forces and thus enables an effective use of the total cross section. If the tensile stress in any given cross section of a concrete member is controlled so as not to cause surface cracking under any combination of predictable external forces (as concrete is generally weak in tension stress), it is possible to construct a structure that requires minimum maintenance in the future. Then, it is generally say that cracking or other types of damage to concrete surface of a PC structure designed based on the above assumption therefore implies that the durability of the member has already been lost. To introduce of prestressing in all parts of a PC structure is, however, actually impossible. Especially, prestressing is often difficult at joints in cast-in-place backfill, in the longitudinal direction of a bridge slab, or at the anchorage of prestressing steel, then, reinforced concrete (RC) is used in some parts.

In view of the above, "preventive maintenance" rather than "corrective maintenance" is expected to be generally adopted for PC bridges. In "corrective maintenance", which was used for RC structures in most cases, maintenance is carried out only after deterioration and damage become apparent in daily and regular inspections. In "preventive maintenance", decisions are made for maintenance using a health monitoring system that monitors time-based change in structural behavior with sensors installed on major structural elements including prestressing tendons. The latter approach overlaps the concept of performance-based design, which has recently been applied worldwide.

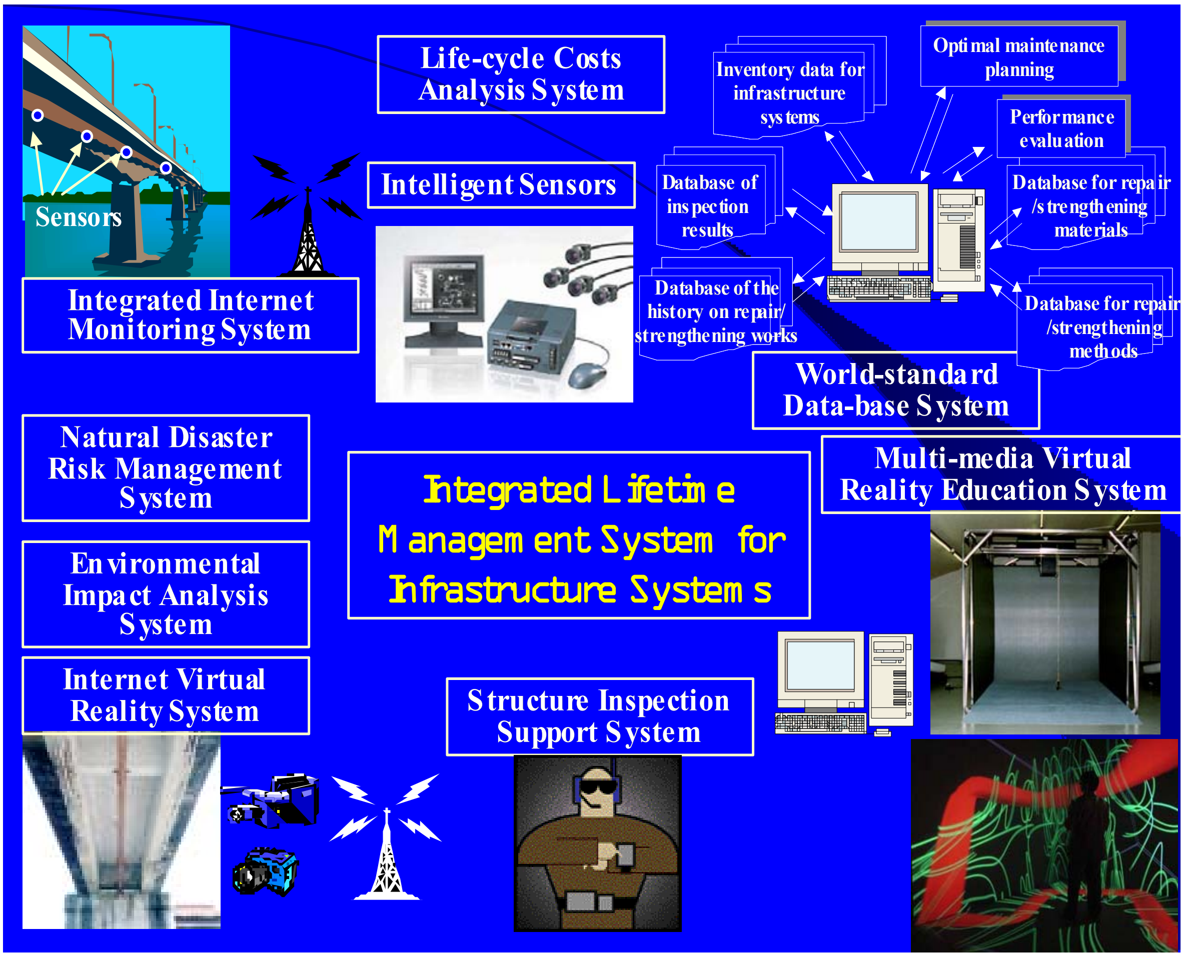


Figure 1. Technological components for building an integrated life-cycle management system.

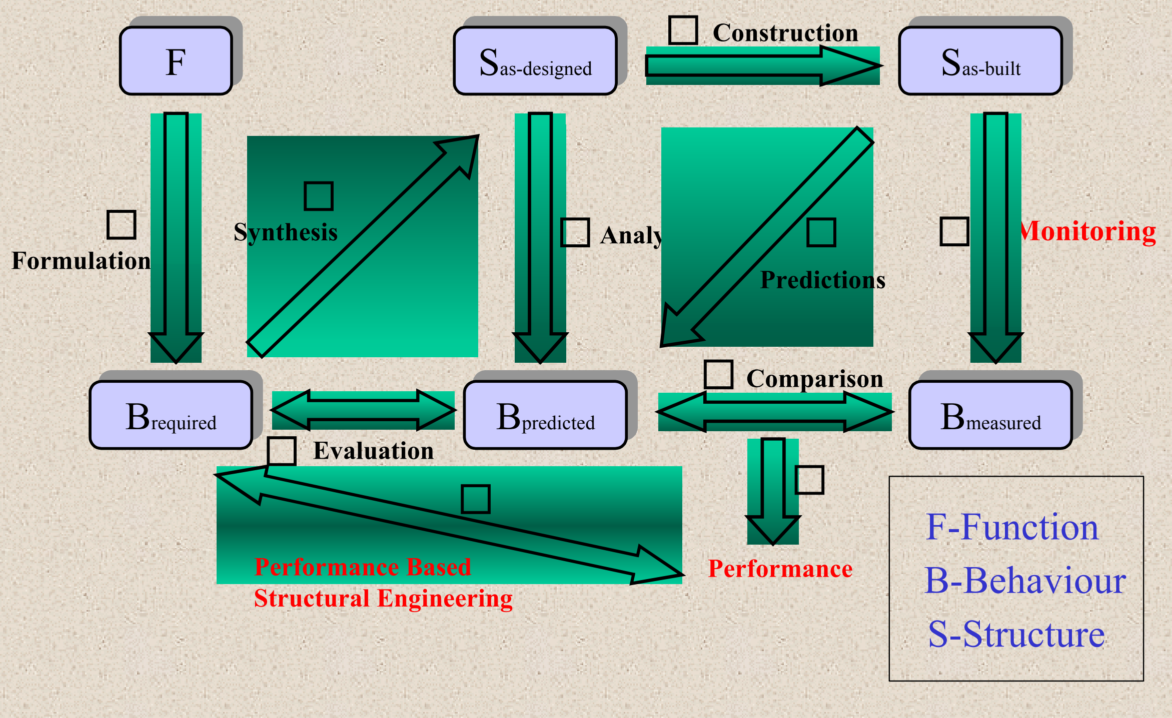


Figure 2. Tasks of health monitoring in maintenance of a structure.

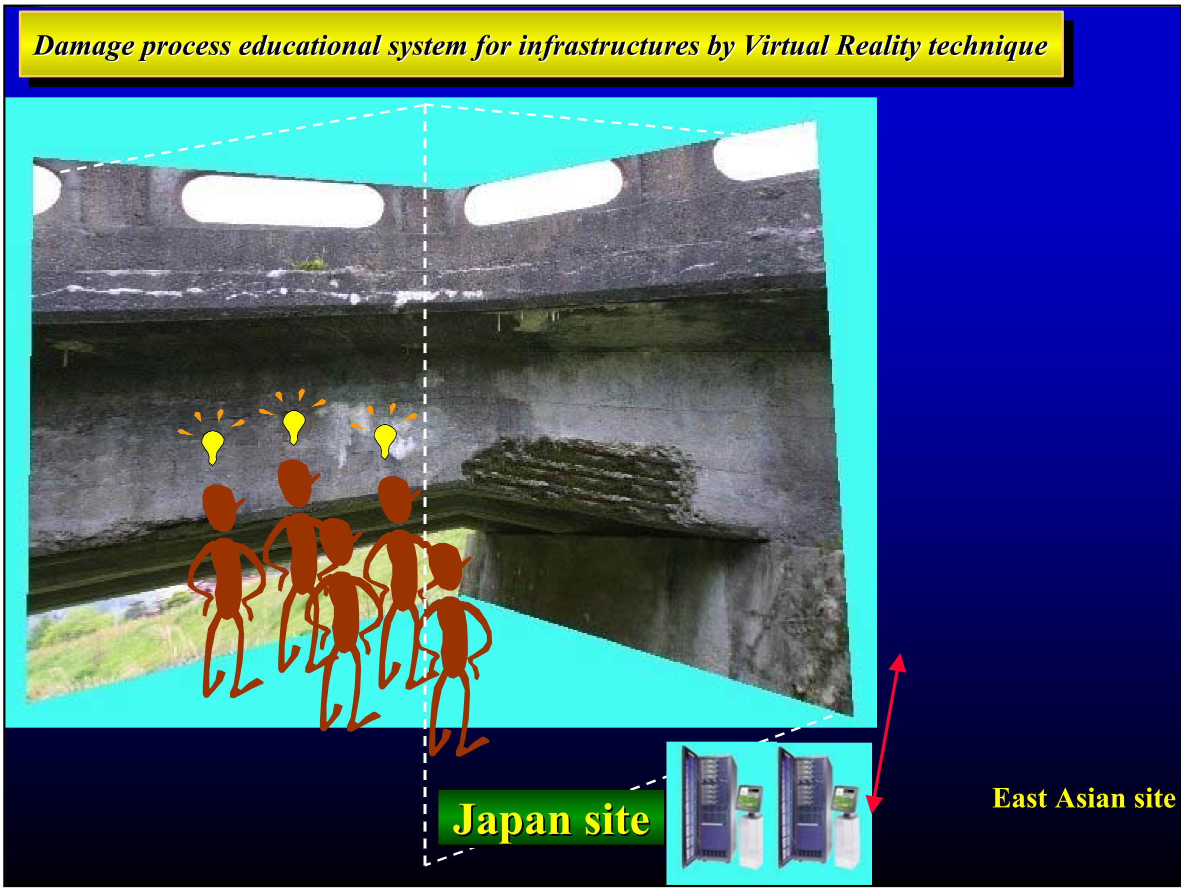
Figure 2 shows the position and role of the "integrated health monitoring system" in maintenance. The system enables real-time monitoring, a technological component for building an integrated life-cycle management system for PC and other structures (Figure 1). The "integrated health monitoring system" is composed of the Internet and other types of information technology, state-of-the-art information processing technologies and soft computing technologies. Figure 2 shows a flowchart of steps ((1)-(10) in Figure 2) that enable various interactive checking not only during design and construction but also during service. Monitoring (*steps 6* through *10*) is added to the conventional phases of design and construction of a structure (*steps 1* through *5*) [Smith [2001]]. The figure presents a correlation among three levels of behavior of the structure: required behavior (performance) $\boldsymbol{B_{required}}$, predicted behavior $\boldsymbol{B_{predicted}}$ and measured behavior $\boldsymbol{B_{measured}}$. ***S*** indicates the structure. In conventional design, functional (performance) requirements for the structure submitted by the owner are initially defined (*step 1*). Design is developed based on the comprehensive structural data obtained (*step 2*) and the behavior of the structure is predicted using an analytical model (*step 3*). After verifying (assessing) the performance requirements according to the prediction (*step 4*), construction is carried out (*step 5*). The completed structure is put into service. If the structure proves unsatisfactory as a result of performance verification, work (design) is carried out again from *step 2* or *3*. The steps are one-directional and lack a viewpoint of post-construction maintenance. Adding the monitoring phase represented as *steps 6* through *10* in Figure 2 enables the monitoring of a third behavior ($\boldsymbol{B_{measured}}$), which can be employed for improving the analytical model during design, the re-verification of performance requirements and the long-term checking.

For efficient maintenance, the integration of large quantities of element technologies that have been accumulated for newly constructed structures and the aggregation of knowledge and experience concerning maintenance are essential. To that end, a system should be established to continuously hand down the knowledge and experience related to maintenance, and to conduct comprehensive training of next generation engineers while aiming to develop a world standard (Figure 1). In life-cycle management of diverse social capital stock, the maintenance of a PC bridge for example, appropriate diagnosis and development of a maintenance plan compatible with environmental considerations are important. Sharing knowledge and experience requires the development of a knowledge and experience dissemination system that exceeds time and space limitations. Image processing technologies and sophisticated multimedia virtual reality technologies are applied to represent in a short time frame the damage to and deterioration of social infrastructures that actually occur over a long period of time, and to substitute virtual experience for real experience. Figure 3 gives an image of an education system. It is a multimedia virtual reality system almost encompassing the world. It enables trainees to have a virtual experience of various damage conditions on a PC bridge or to virtually experience maintenance experts' thought process with respect to the points they focus on and how to arrive at a diagnosis based on the environmental conditions and the damage.

3 LCC BASED PC BRIDGE MANAGEMENT SYSTEM [Miwa [2001]]

For the diagnosis of a PC bridge, preventive maintenance is generally adopted in combination with health monitoring because of the structural properties (see previous section) of such a bridge. Since reinforced concrete (RC) elements also constitute a PC bridge, corrective maintenance is also required for total maintenance of the bridge. This section describes a deterioration diagnosis system for PC bridges (**BREX** (Bridge Rating Expert System) for PC bridges). It is a feature of a bridge management system that is mainly intended for reinforced concrete bridges (**J-BMS**) that the author jointly developed.

Figure 4 shows a flowchart of **J-BMS** steps for developing an optimum maintenance plan taking life-cycle cost into consideration [Smith [2001]]. Under J-BMS, a fundamental maintenance procedure is followed from inspection to diagnosis and remedial action. First, visual inspection or a similar level of inspection and detailed inspection are conducted in combination for existing bridges. Inspection results are stored in a database. The inspection data on the bridge to be diagnosed are extracted from the database and input to the "Concrete Bridge Rating Expert System" with the technical specifications



for the bridge. Then, the system (deterioration diagnosis feature) is activated. The load carrying capability and durability of main girders and slabs of the bridge are rated on a 0-100 scale to indicate their soundness. Thus, the bridge is diagnosed and assessed. Next, future deterioration of the bridge is predicted using the "deterioration prediction feature" according to the soundness identified by diagnosis. Predicted progress of deterioration of members is visually verified. Finally, based on the progress of deterioration identified by the "deterioration prediction feature", the effects of remedial methods and the costs involved are assessed to deprive an optimum maintenance plan that specifies the selected method, timing of maintenance and life-cycle cost. This integrated management system focuses on existing concrete bridges. It enables not only the soundness diagnosis of existing concrete bridges and the selection of repair and strengthenig methods according to the diagnosis but also effective and efficient development of an optimum maintenance plan that produces the maximum effect within limited budgets. The system was constructed by applying such latest information processing technologies as neuro-fuzzy expert system, genetic algorithm (GA), and immune algorithm (IA) [Miyamoto [2002]].

Figure 3. Example of a system for transferring maintenance technology and knowledge based on multi-media virtual reality technology.

Discussed below are final diagnostic results for "8th -span of KS bridge", an eight-span post-tensioned prestressed concrete simply supported T-girder bridge in Yamaguchi prefecture. The results were obtained by inputting inspection data. Figure 5 shows part of the input screen for main girders and slabs. Dozens of parameters are input such as technical specifications for the bridge, various investigation and inspection results, and cracking conditions in slabs and main girders. Especially for PC bridges, results of inspection for deformations at the anchorage of longitudinal prestressing tendons, near the tendon sheath and at the anchorage of transverse prestressing tendons are also input to represent impacts on prestressing tendons (see Figure 5). Some parameters require multiple choice answers based on subjective judgements. Choices can be made at the click of a mouse. Figure 6 is a visual display of final diagnostic results reached by coupled calculation of diagnostic processes for PC bridges. The load carrying capability, durability and seaviceability of main girders and slabs of the bridge to be diagnosed are assessed on a 0-100 scale to identify their soundness. The results of comprehensive assessment of main girders and slabs can be displayed visually. Identifying the

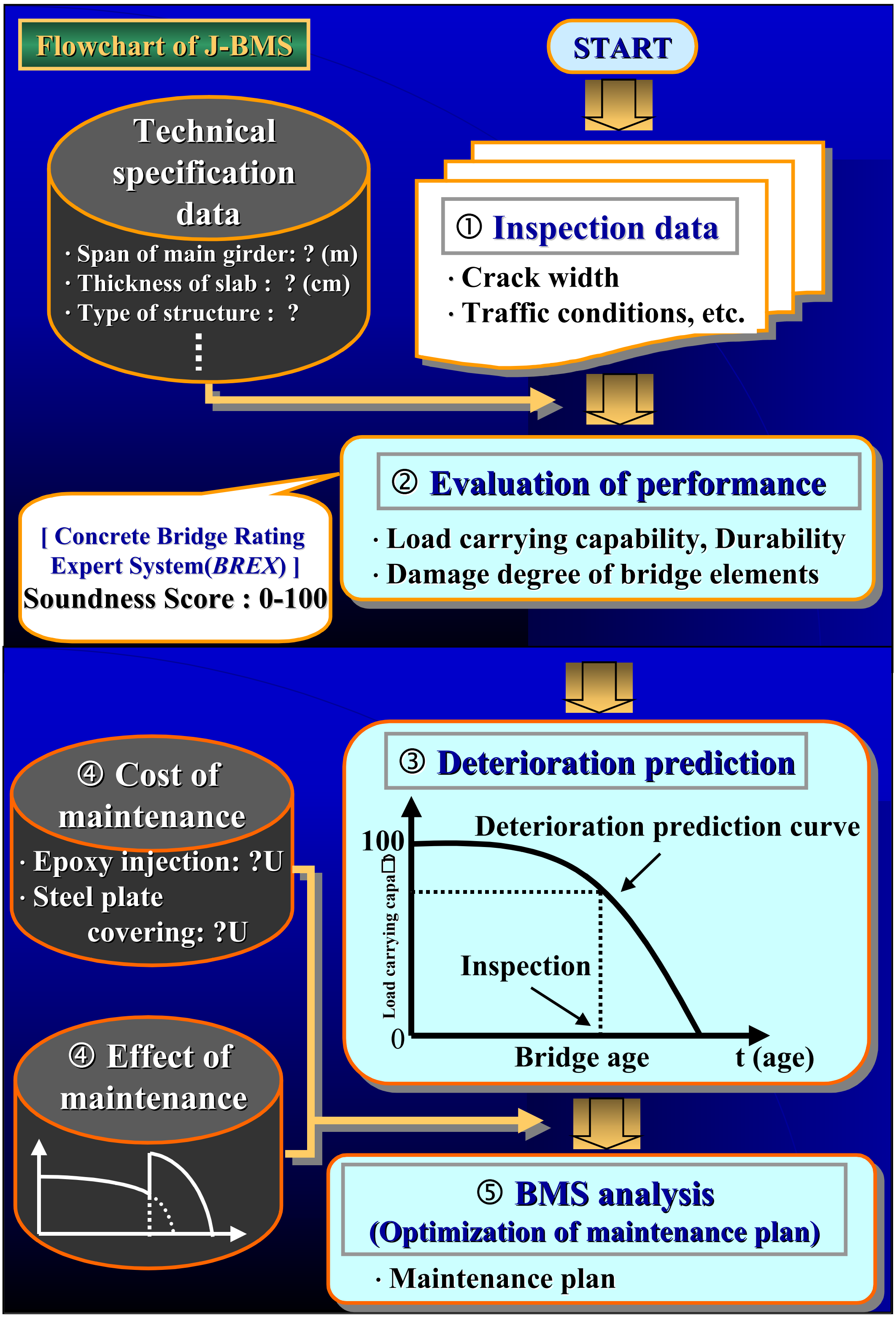


Figure 4. Flowchart of steps under a J-BMS-based concrete bridge management system.

deterioration of prestressing tendons is most important on PC bridges. The screen therefore displays the results of deduction for assessing damage at the anchorage of prestressing tendons and along the tendon sheath to represent deterioration.

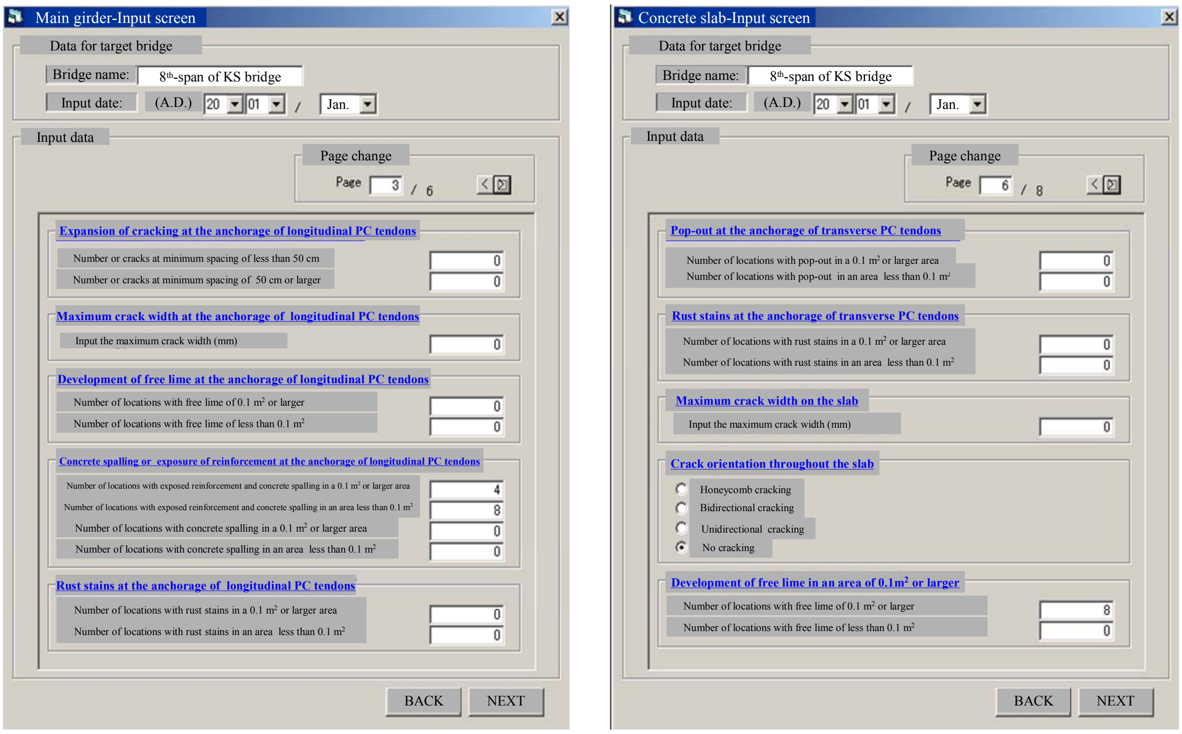


Figure 5. Inspection result input screens for the main girder (left) and concrete slab (right) of an existing PC bridge.

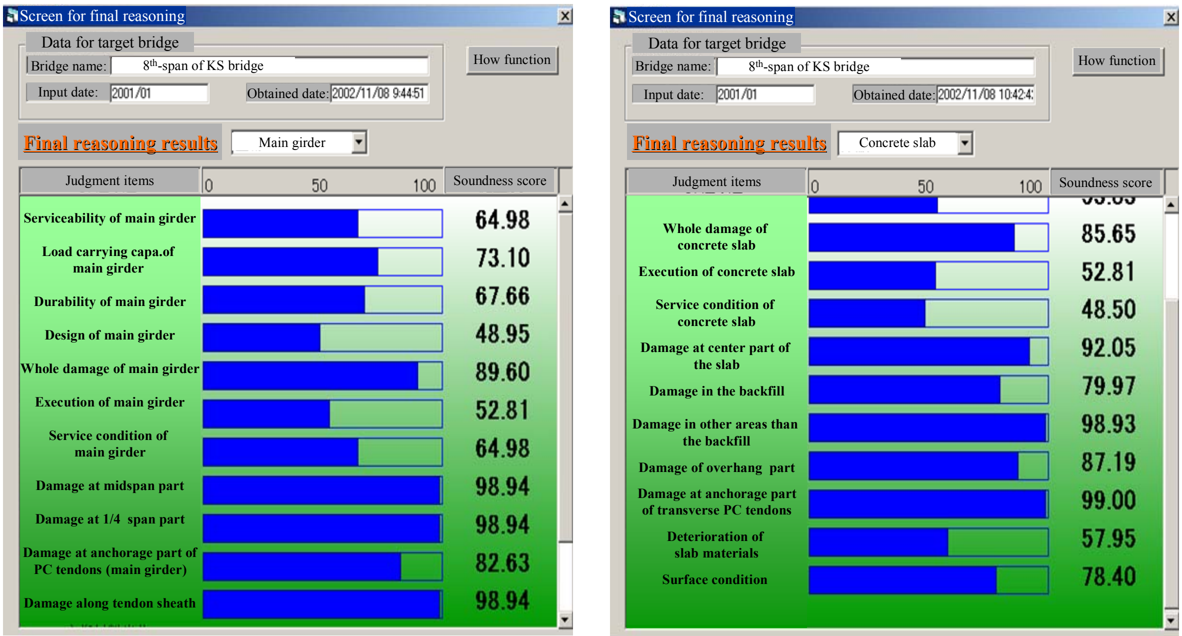


Figure 6. Output screens for final results from PC bridge-BREX for the main girder (left) and concrete slab (right).

The "deterioration diagnosis feature" is equipped with capabilities of verifying diagnostic results obtained by BREX and of learning by neural network. Clicking the button, "Execute How function" will direct the user to a frame shown in Figure 7. The frame shows all the diagnostic processes developed for PC bridges on the left. It also presents in the upper right details about the diagnostic processes for the item (damage to the anchorage of prestressing tendons on the girder in this example) for which the diagnostic results need to be verified (inspection items) and the relevant sets of production rules (if-then rules) ((i) damage to and rust stains on the anchorage of longitudinal prestressing tendons, (ii) appearence of free lime at the anchorage of prestressing tendons on the main girder and (iii) cracking in pattern (7)). Thus, it is easily verified how the rules are applied to deduce the final diagnostic results. Present shapes of membership functions are also provided in the lower right for the sets of production rules (i) through (iii). The objective is to enable the verification of knowledge base update (effects of learning) in the case of variance of the diagnostic result from "right answers" such as opinions of domain experts and experimental results. Thus, effective support is provided in communication between engineers and systems developers.

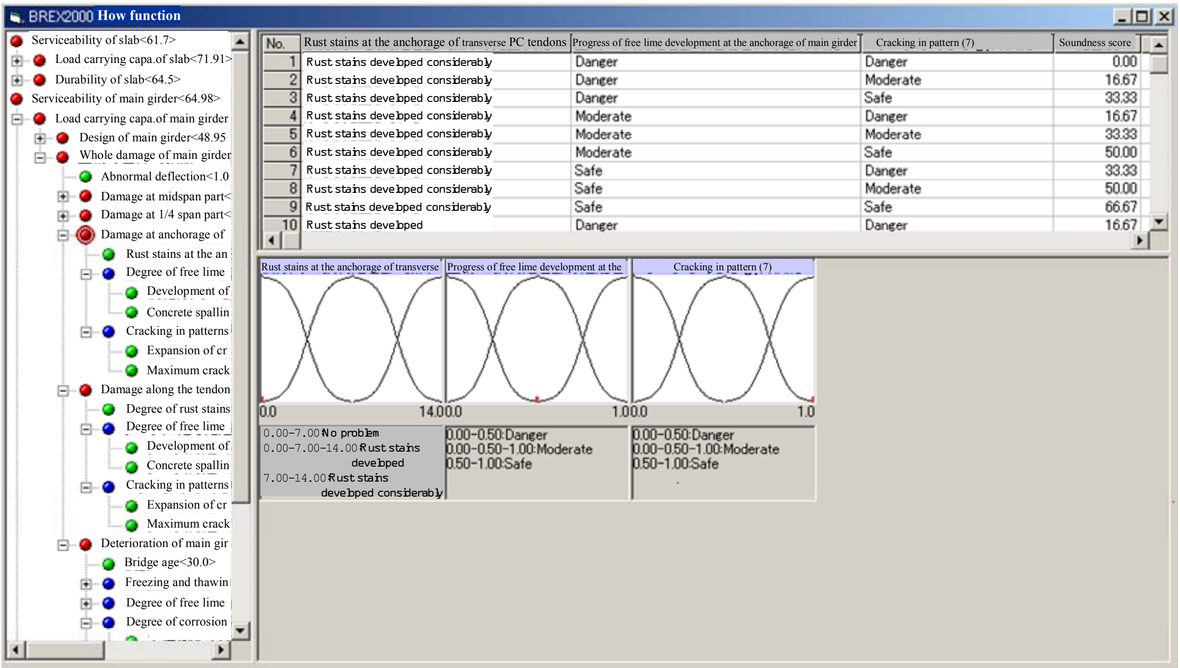


Figure 7. Screens for supporting final result verification and knowledge update.

4 CONCLUDING REMARKS

In advanced countries in Europe and the United States, life-cycle management of social capital stock including prestressed concrete (PC) bridges and other social infrastructures has recently been gathering attention. An example is LIFETIME Thematic Network, an organization based in Finland that is now being launched by the European Union as an area-wide project [Technical Research Center of Finland et al. [2001]]. Another group led by the United States has also been accumulating expertise and know-how of its own. The First International Conference on Bridge Maintenance, Safety and Management was held in Spain in July 2002 [Casas et al. [2002]]. Thus, there has been a move, mainly among the countries with well-developed social capital, to establish a common global goal. It is important for Japan to establish an advanced management system in the relevant field ahead of the rest of the world in close coordination with the recent move and create a base for research that will enable Japan to lead the world. Figure 8 illustrates how database systems are shared by combining health monitoring and next generation information technologies, and how world standards for an integrated life-cycle management system are developed. If Japan becomes capable of taking the initiative in the age of world standard development for maintenance owing to active approach, the efficiency of maintenance of structures including PC bridges will be increased and the field of maintenance will become technically more challenging. In the future, it is expected to be important to attract young excellent engineers to the field of protection of aged structures, which is analogous to the care of elderly people, and to make the field more dynamic. The author would hope that this paper would be of some help in the future.

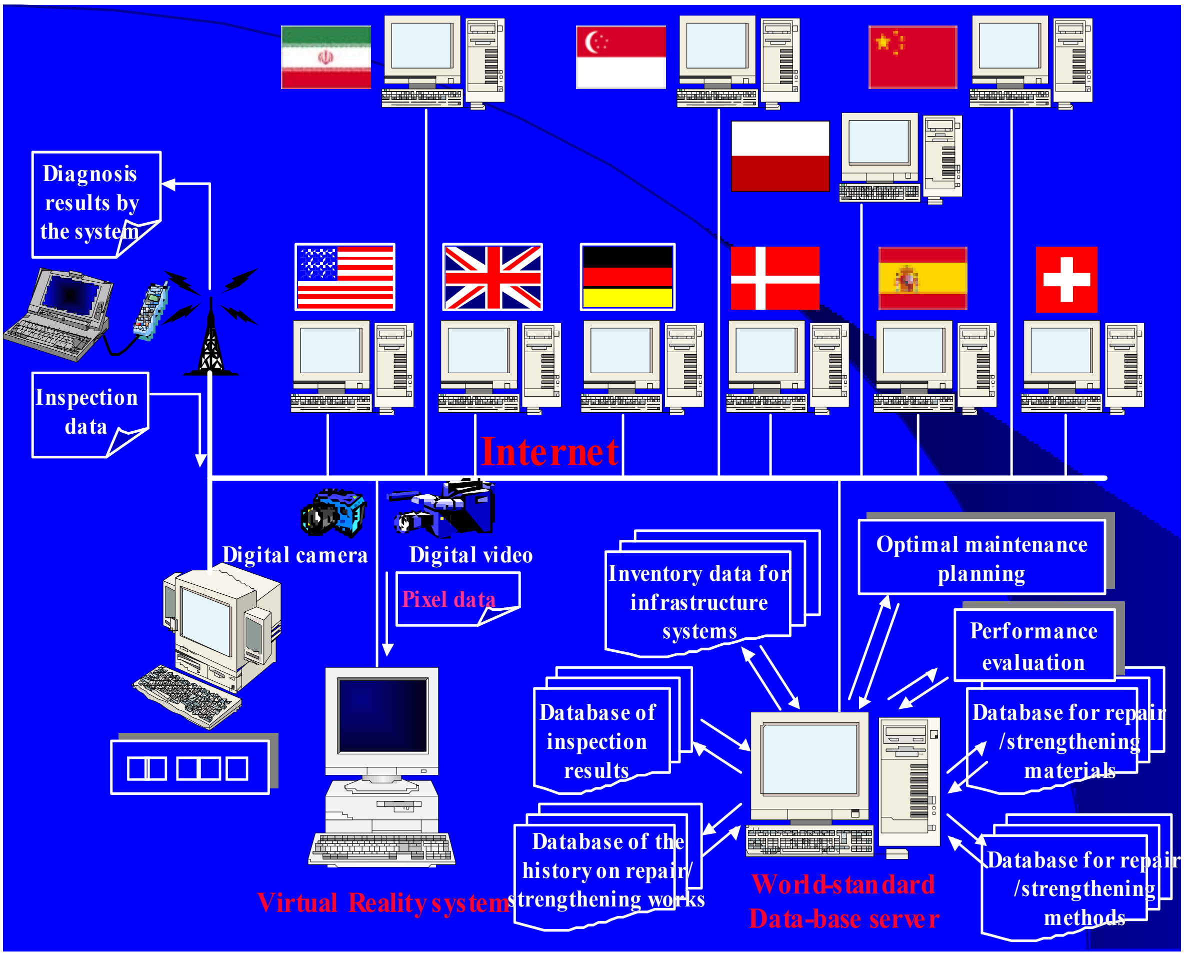


Figure 8. Image of a world-standard-oriented integrated life-cycle management system.

5 REFERENCES

Miyamoto, A., & Frangopol, D. A.(Ed) 2002, 'Maintaining the Safety of Deteriorating Civil Infrastructures', *Practical Maintenance Engineering Institute of Yamaguchi Univ*., Yamaguchi.

Miyamoto, A. 2002, 'A Practical Bridge Management System for Existing Concrete Bridges', *Concrete Structures in the 21st Century*(CD-ROM:C), fib2002 Osaka Congress, pp.111-120.

Miwa T. 2001, 'A Software Tool for Developing Bridge Rating Expert System and Application to PC Bridge Assessment', *Master Thesis of Yamaguchi University*, Yamaguchi.

Smith, I. F. C. 2001, 'Increasing Knowledge of Structural Performance', *Structural Engineering International (SEI)*, IABSE, Vol. 11, No. 3, pp. 191-195.

Technical Research Centre of Finland & VTT Building and Transport, 2001, 'Competitive and Sustainable Growth Programme', *LIFETIME Thematic Network*, (HP address: https://www.rte.vtt.fi/QuickPlace/lifetime_public/Main.nsf).

Casas, J.R., Frangopol, D. A. and Nowak, A.S.(Ed) 2002, 'Bridge Maintenance, Safety and Management', *International Center Numerical Methods in Engineering (CIMNE)*, Barcelona.

Recycling of PET and PC blends

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ABSTRACT

Polyvinylchloride (PVC) is the most commonly used polymeric material in building application. Nowadays, PVC brings a lot of problems because its degradation can take place during recycling as it is very sensitive to thermomechanical stresses.Moreover, it contains heavy metals as thermal stabilizers. Thus, efforts aimed at the substitution of PVC in building applications have to be intensified noticeably. The idea of our work is to use recycled polyethyleneterephtalate (PET) as an alternative to PVC and therefore this study focuses on detailed study of PET recycling.
However, PET recycling is a complicated process because of chemical and mechanical degradation of PET during recycling and therefore this process has to be carried out with utmost caution. The first step, before recycling, is to determine the level of PET degradation caused by ageing and the resulting properties. In addition, the properties can be improved by the addition of polycarbonate (PC). Thus, our work is also aimed to use wastes of these polymers for blend preparation.
Mentioned subject has many interests, which can be summarized as follows:
- an academic one – to study an elementary behavior of blends prepared from PET and PC, miscibility of which is limited. Furthermore, their compatibility can be improved by reactive transesterification which can occur during melt processing.
- a technological one – to find relations between the method of blend preparation and the resulting properties.
- an environmental one – to recycle polymeric wastes.
- an economic one.

First of all, characterization of both PET and PC wastes was carried out. Then, the transesterification of PET and PC wastes was studied. For this purpose, PET and PC blends in various compositions (80/20, 70/30 and 50/50 wt.%) were prepared using a twin-screw extruder. Consequently, properties of the blends were studied. The results show that extrusion is a necessary processing operation in order to obtain suitable properties. Transesterification seems to be detected in all blends, however, PET/PC 80/20 (wt.%) blend shows the best mechanical properties, which can be quite comparable to those of PVC.
In near future, ageing and particularly photoageing of the blends will be studied in order to assess the lifetime prediction.

KEYWORDS

Recycling, polymer blends, mechanical properties

1 INTRODUCTION

Polyethyleneterephthalate (PET) consumption has grown sharply during the last years due to the expansion of the PET bottle market [SPMP 2002]. Nowadays, PET from post-consumer bottles is recycled essentially by chemical way. Development of an industrial recycling process by mechanical way could be interesting. Moreover, PVC used in building applications brings a lot of recycling problems due to the stabilization by heavy metals and to the degradation during recycling process [Braun 2002]. Thus, our idea is to use recycled PET, alternatively to PVC, for building applications. However, degradation occurs also during the PET recycling process and involves a decrease of the mechanical properties [Silva Spinacé & De Paoli 2001] linked to the decrease of the glass transition temperature. A solution could be to blend PET with another polymer with a higher glass transition temperature such as Polycarbonate. Miscibility of PET and PC is strongly discussed in the litterature but a transesterification reaction enhances compatibility between the two polymers. After characterization of the wastes [Pawlak et al. 2000] and study of their photoageing kinetics [Fechine *et al.* 2002], several blend compositions (PET majoritary) were prepared by twin screw extrusion followed by injection molding. Rheological and thermal characterization were investigated at each recycling process step and mechanical properties were determined on the injection molded specimens. Results were compared with the PVC properties [Polymer Handbook 1999].

2 EXPERIMENTAL

2.1 Materials

PET wastes were obtained from post-consumer bottles disposal and they were crushed in order to prepare PET flakes. These flakes were washed and dried to eliminate sticks and labels. PC waste was used either from degraded water bottles or from synthetic glass. Flakes from PC bottles were crushed with a size of about 4 mm. Flakes from PC glass were obtained with a size of about 8 mm.

2.2 Techniques

To prepare polymer films, a compression molding press was used at 260°C. Pressure time was fixed at about one minute. In this case, degradation of the polymer was limited.
Pellets were prepared using a Clextral twin-screw extruder at 270°C. The material residence time in the extruder is about two minutes. Overall material transformations are resumed in Fig. 1.

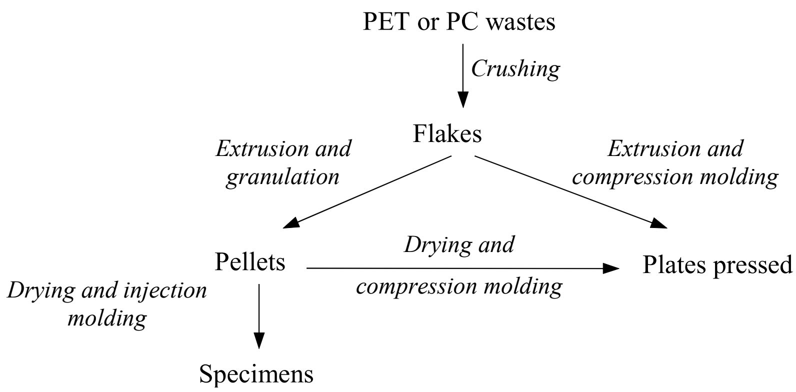

Figure 1. Recapitulative schema of transformations from wastes.

Accelerated photoageing was carried out in a SEPAP 12-24 [Lemaire *et al.* 1988].

Thermal properties were measured by a Mettler – Toledo 822e Differential Scanning Calorimeter. Used DSC method included three steps. The first was heating from 30°C to 300°C at 10°C/min in order to eliminate the thermal history of the sample. The second step was cooling at 10°C/min to 30°C. Finally the samples were heated similarly to the first step.
FTIR was performed by a Nicolet spectrometer. Melt rheology was performed on an ARES rheometer. To determine mechanical properties, a Zwick 5102 Charpy impact tester and a Zwick 1456 tensile tester were used.

3 RESULTS AND DISCUSSION

3.1 Waste properties

At first, melt rheology of PET bottle wastes was studied. These samples present significant differences in their rheological behavior because of their different manufacturing processes [Everall *et al.* 2002] (molding condition, drawing in the mold, etc…) or because of their application (using with gaseous liquid or non-gaseous liquid). FTIR measurement of both PET wastes and virgin PET (see Fig. 2) shows that their kinetics and level of degradation are virtually similar.

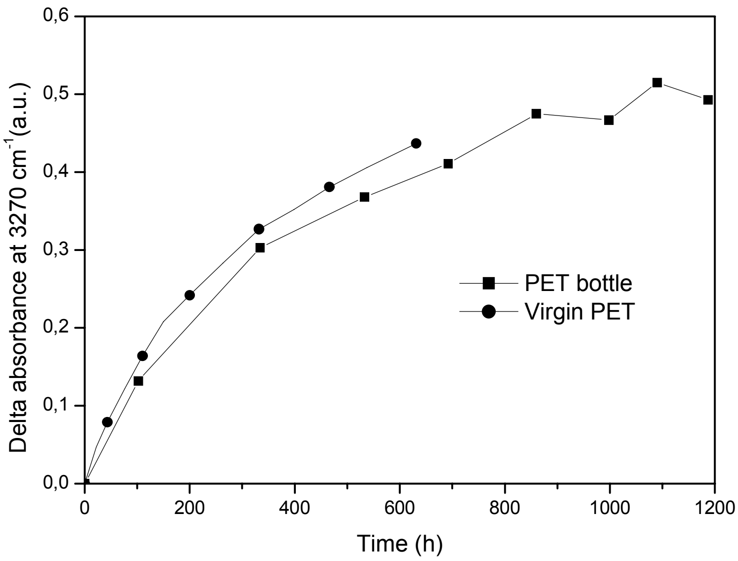

Figure 2. FTIR at 3270 cm^{-1} (hydroxyl region) during photo-ageing in SEPAP 12-24 for PET bottle and virgin PET.

PC bottle and PC glass present also differences. Melt rheology shows that PC bottle indicates higher viscosity than that of PC glass (Fig. 3).
This can be explained by blow-extrusion PC grade for bottle manufacturing and injection grade for PC glass.

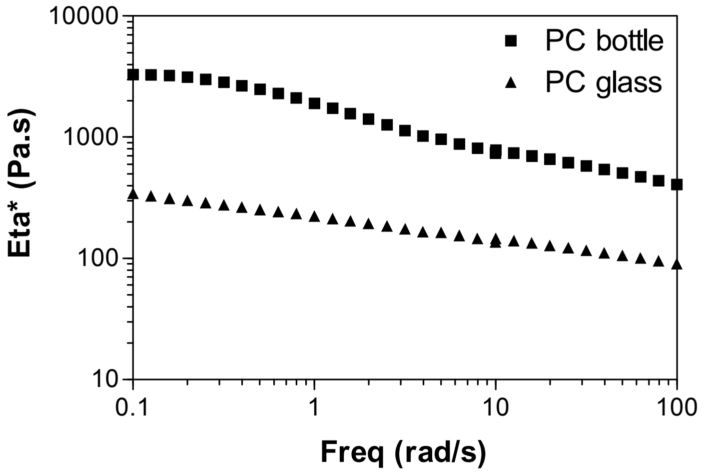

Figure 3. Complex viscosity at 260°C for specimens of PC bottle and PC glass.

3.2 Blends preparation

Pellets of blends were obtained using a twin-screw extruder from flakes dried for about 12 hours at 130°C. Addition of PC at low levels involves changes in PET processing conditions. Miscibility in the PET/PC blend seems to be favoured by a twin-screw extrusion [Robinson 1995].
As shown by Ignatov et al. [1997], transesterification occurs between virgin PET and PC in molten state. In our case, the process of transesterification was also detected between PET and PC wastes.
It was found that the transesterification is the most important in the 80/20 PET/PC wt. % blend. Extrusion of blends with small flakes prepared from PC bottle was easier than the extrusion of larger PC glass flakes. Size of the flakes seems to be also a very important factor for the quality of extrusion and the resulting pellets.In addition, to improve sample properties, compression molding was used. Blends with compositions of 50/50 and 70/30 (PET/PC glass wt. %) were compression molded in order to obtain sheets from melt material.
Furthermore, specimens were manufactured by injection molding directly from the flakes to compare mechanical properties with the others specimens prepared from the pellets.

3.3 Thermal properties

DSC and melt rheology show that in all the blends, glass transition temperature associated with PET phase is always higher than in the case of neat PET. In blends this glass transition temperature is near to 85°C while Tg of neat PET is around 80°C (Fig. 4). This proves that reaction of transesterification occurs in blends prepared from waste materials.

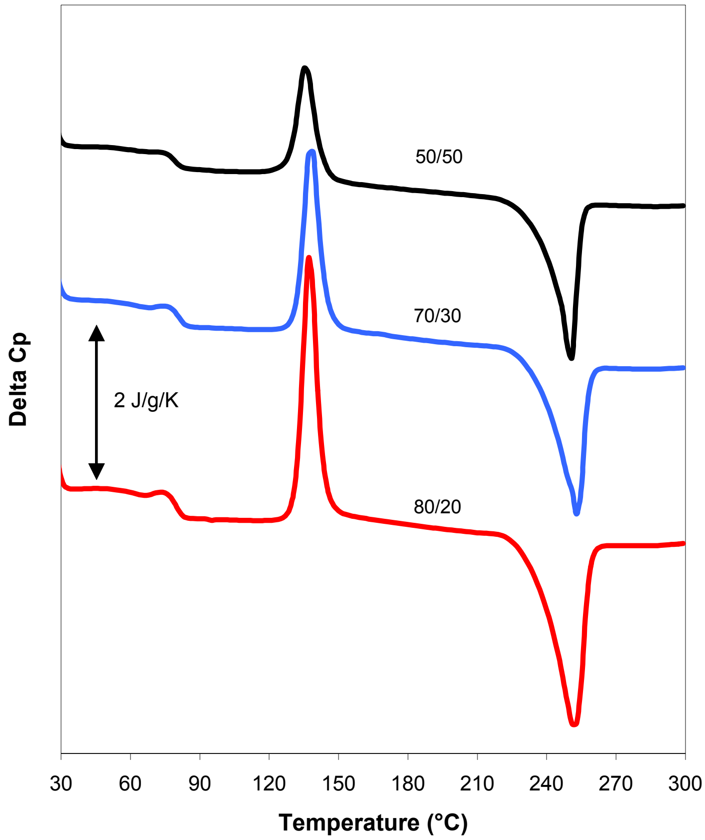

Figure 4. Thermograms of pellets for different compositions of blend.

3.3 Mechanical properties

Tensile, flexural and impact characteristics of the PET/PC blends are listed in Table 1. For specific applications PET and PC blends exhibit suitable tensile and flexural properties.

Test	Properties	Blend 80/20	Blend 70/30	Blend 50/50
Tensile	E-modulus (Young) [MPa]	3100	3110	3120
	Yield stress [MPa]	57-59	60	60
	Elongation at yield stress [%]	9-10	10	11
	Elongation at break [%]	<265	226	200
Flexural	Modulus [MPa]	2120-2150	2225	2240
	Strain for a conventional deflection of 6 mm [MPa]	70-71	71	71
	Yield stress [MPa]	77-81	82	85
	Deflection at yield stress [mm]	8,3-9,1	9,4	10,1
Impact	Impact strength [kJ/m²]	2,1-2,2	2,2	3,8

Table 1. Mechanical tests results for specimens prepared from pellets.

Comparison of these results (Table 1) with results obtained by specimens prepared from the flakes (Table 2) shows that extrusion is a necessary step to obtain improved properties.

Test	Properties	Blend 70/30	Blend 50/50
Flexion	Modulus [MPa]	2080	2116
	Strain for the conventional deflection of 6 mm [MPa]	69,6	70,7
	Yield stress [MPa]	82	86
	Deflection at yield stress [mm]	8,2	10,0
Impact	Impact strength [kJ/m²]	2,0	2,0

Table 2. Mechanical test results for specimens prepared from flakes.

4 CONCLUSION

Level of degradation caused by ageing is low when wastes of PET from post-consumer bottles are recovered. In case of PET recycling, PC addition to PET makes easier the overall recycling process and brings better properties than neat PET ones [Torres *et al.* 2000]. PET and PC blends present suitable mechanical properties for building application, close to the PVC properties.
Glass transition temperatures in blends show that the transesterification occurs between PET and PC wastes during blending in molten state. In the recycling process, extrusion seems to be necessary step to obtain applicable properties.
Our aim is to enhance the impact properties for using PET/PC blends in building applications and to compare photoageing kinetics of PET/PC blends with kinetics of neat polymers.

5 ACKNOWLEDGMENTS

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6 REFERENCES

Braun, D. 2002, 'Recycling of PVC', *Progress in Polymer Science*, **27**, 2171-2195.

Everall, N., MacKerron, D., Winter, D. 2002, 'Characterisation of biaxial orientation gradients in poly(ethylene terephthalate) films and bottles using polarised attenuated total reflection FTIR spectroscopy', *Polymer*, **43**, 4217-4223.

Fechine, G. J. M., Souto-Mayor, R. M., Rabello, M. S. 2002, 'Structural changes during photodegradation of poly(ethylene terephthalate)', *Journal of Materials Science*, **37**, 4979-4984.

Ignatov, V. N., Carraro, C., Tartari, V., Pippa, R., Scapin, M., Pilati, F., Berti, C., Toselli, M., Fiorini, M. 1997, 'PET/PC blends and copolymers by one-step extrusion: 1. Chemical structure and physical properties of 50/50 blends' *Polymer*, **38**[1], 195-200.

Lemaire, J., Arnaud, R., Lacoste, J. 1988, 'The prediction of the long-term photoageing of solid polymers', *Acta Polymerica*, **39**[1], 27-32.

Pawlak, A., Pluta, M., Moraviec, J., Galeski, A., Pracella, M. 2000, 'Characterization of scrap poly(ethylene terephthalate)', *European Polymer Journal*, **36**, 1875-1884.

Polymer Handbook Fourth Ed. 1999**,** V/162.

Robinson, A.M., Haworth, B., Birley, A.W. 1996, 'Elongational rheometry of polyethylene terephthalate/bisphenol-a polycarbonate blends', *European Polymer Journal,* **32**[9], 1061-1066.

Silva Spinacé, M.A., De Paoli, M.A. 2001, 'Characterization of Poly(ethylene terephthalate) after multiple processing cycles', *Journal of Appied Polymer Sci*ence, **80**, 20-25.

Syndicat des Producteurs de Matières Plastiques (SPMP). Matières Plastiques Chiffres 2002.

Torres, N., Robin, J.J., Boutevin, B. 2000, 'Study of thermal and mechanical properties of virgin and recycled poly(ethylene terephthalate) before and after injection molding', *European Polymer Journal,* **36**, 2075-2080.

Globalization and Competitive Strategies for Sustainable Construction in a Developing Country: North Cyprus Construction Industry

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ABSTRACT

Sustainability is of increasing importance to the efficient, effective and responsible operation of construction business. Achieving sustainable development is a global challenge, a challenge that is bigger for developing countries. Sustainability encompasses competitiveness and long-term strategies and combines economic objectives with understanding and operating within limits, increasing resource efficiencies and ensuring the social license to operate. There is a need to introduce more sustainable construction practices and performance in pursuing an overall development in the context of globalization and competitive strategies for the construction industry in developing countries. Our research aims to demonstrate that sustainability issues are of critical and strategic importance to construction business in a developing country: the case of North Cyprus. Our main objective is to identify the key issues and challenges facing sustainable construction in the developing world. This research includes review of background literature, interviews with managers on sustainability of building projects and analysis of this information to develop findings in the context of globalization and competitive strategies for sustainable construction in a developing country: the case of North Cyprus construction industry. This paper discusses possible competitive strategies that the small and medium-sized construction firms in the North Cyprus construction industry could adopt so as to benefit from the globalization of the construction industry for their sustainability. The paper concludes how globalization can offer benefits to construction firms in a developing country like North Cyprus if appropriate strategies are adopted.

KEYWORDS

Sustainable construction, globalization, competitive strategy, developing country.

1 INTRODUCTION

Sustainability is of increasing importance to the efficient, effective and responsible operation of construction business. It has been argued by Rodwin [1987] that the construction industry is unique in its ability to facilitate development by providing directly for human needs, stimulating investment and generating employment. Sustainable construction is an integrative and holistic process aiming to restore harmony between the natural and the built environment, and create settlements that affirm human dignity and encourage economic equity. The concept of sustainable construction now transcends environmental sustainability to embrace economic and social sustainability, which emphasises possible value addition to the quality of life of individuals and communities [Plesssis, 2001]. The World Summit on Sustainable Development Plan of Implementation provides a range sustainable development objectives that should be aimed for in order to achieve sustainability [United Nations, 2000].

A review of literature on sustainability suggests that sustainability can be described in terms of social, economic and environmental states that are required in order for overall sustainability to be achieved [Gibberd, 2003]. The physical environment and the construction sector are linked principally by the demands made by the latter on global natural resources, and this assumes huge environmental significance with the rapid growth in global population and the attendant implications for natural resources. This is especially the case with housing and infrastructure, which are very resource intensive. The call and desire for sustainable construction is in realisation of the construction industry's capacity to make a significant contribution to environmental sustainability because of the enormous demands it exerts on global resources [Plesssis, 2001]. The construction industry has the potential to enhance economic sustainability through its structure, conduct and performance. An economically efficient construction industry enhances environmental sustainability by ensuring least cost methods of construction and optimal allocation of resources and discouraging wastes.The combination of economic and regulatory measures by which sustainable construction can be encouraged, would be difficult to implement in the developing world. This is because of the absence of the necessary government and private institutions to facilitate the process. In the particular case of sustainability initiatives, the role of institutions as a necessary prerequisite to successful implementation of sustainable policies and processes is well documented [Ebohon, 1996]. Perhaps more significant is the constituents of the institutional problems cited in the developing countries as impediments to sustainable development. Commonly identified institutional problems in the developing world include inadequate skills and personnel, poor monitoring, competition and lack of coordination, lack of political will and public awareness of the concept of sustainability, and inadequate legislative frameworks.

The results and effects of globalisation itself have altered the shape of the construction industry together with the methods to which projects are procured. The outcomes have meant that the industry has been forced to respond to issues such as increased levels of client expectations, intense domestic competition, diminishing profit margins and the need to participate on an international platform. Although globalisation is acknowledged to have advantages and disadvantages, its impact on the sustainability of construction industries in the developing countries has caused enormous fear. The sustainability of construction firms as any other business organisation is influenced by both external and internal factors at industry and organisation levels respectively [Porter, 1990; Ngowi and Rwelamila, 1999]. The construction firms in most developing countries are satisfied with their domestic operations, and are generally uninterested in making foreign forays. Most of these firms are unaware that there exists a connection, in many cases a strong one, between the activities of foreign competitors and the future of the domestic market. When faced with increased competition, these firms prefer to retreat within the protective custody of domestic legal and political system rather than aggressively confront the foreign competition. There is almost no international element in the business strategies that most construction firms in developing countries formulate and implement [Ngowi and Lema, 2002].

Construction industry in North Cyprus is aware and recognizes the need to modernize in order to tackle the severe problems encountering it. These are *Profitability*, *Research and Development*, *Training*, *Price and Cost*, *Dissatisfaction of Clients* and *Fragmentation*. The necessary conditions for competitiveness for the North Cyprus construction industry include strong and sustained levels of productivity growth, openness to innovation and new technology and a commitment to delivering value for clients' money. There is growing interest in the role of innovation within the North Cyprus construction industry [Yitmen and Alibaba, 2004; Yitmen and Taneri, 2005; Yitmen and Öztürk, 2005]. In today's global world, construction firms need to learn how to compete and adopt strategies that give them market shares that are no longer geographically limited. Achieving sustainable development is a global challenge, a challenge that is bigger for developing countries. Sustainability encompasses competitiveness and long-term strategies and combines economic objectives with understanding and operating within limits, increasing resource efficiencies and ensuring the social license to operate. There is a need to introduce more sustainable construction practices and performance in pursuing an overall development in the context of globalization and competitive strategies for the construction industry in developing countries. This research aims to demonstrate that sustainability issues are of critical and strategic importance to construction business in North Cyprus. Our main objective is to identify the key issues and challenges facing sustainable construction in the developing world as well as the major barriers to practicing sustainable construction and also to encourage construction sector to become more thoughtful about planning, design, process and the finished product for the user, local community and environment. This can be achieved by the development of best practices in terms of economic sustainability: improving profitability and predictability, decreasing defects, quality improvements and leading to greater client satisfaction. These quality improvements can be equated to improvements in social and environmental sustainability.

2 RESEARCH METHOD

To meet the objectives of the study, a meeting was organized by the EUL Civil Engineering Department Research Group and representatives from all sub-sectors of the TRNC construction industry, related institutions, chambers, miscellaneous firms etc. were invited to discuss the key aspects of globalization and competitive strategies for sustainable construction in a developing country. Then the empirical data was collected through structured interview within the main large private sector construction organizations. Sixty key people from Senior managers, IT Managers and Quantity Surveyors of thirty contractors were conducted during the interviews. The outcomes from the meeting and interviews constitute the basis of the main structure of the questionnaire. The paper deals with the results of a survey conducted by research members on globalization and competitive strategies for sustainable construction in North Cyprus construction industry. Main topics in the questionnaire were as follows::

i. General information about organizations
ii. Sustainable Development in Cobstruction
iii. Innovation in building materials and methods
iv. Technical Infrastructure and Usage of information technology
v. Barriers, Challenges and Opportunities for Sustainable Construction

2.1 Literature Review

This stage involves a thorough review of literature about sustainability of building projects and analysis of this information to develop findings in the context of globalization and competitive strategies for sustainable construction in a developing country: the case of North Cyprus construction industry. The intensive literature review resulted in the identification of 15 factors affecting the phases of Building Life Cycle: *Briefing, Site Analysis, Target Setting, Design, Construction, Handover, Operation, Demolition/Refurbishment*. These factors are as follows:

(a) Access to facilities

(b) Occupant compfort
(c) Inclusive environments
(d) Participation and Control
(e) Education, Health and Safety
(f) Local Economy
(g) Efficiency of Use
(h) Adaptability and Flexibility
(i) Ongoing Costs
(j) Capital Costs
(k) Water
(l) Energy
(m) Waste
(n) Site
(o) Materials and components

2.2 Data Collection

The second stage involved the collection of data. A questionnaire survey, which was administered to almost all the firms, which are registered to the Association of Building Contractors, has been used in conducting the survey. The survey includes five main types of information involving *organizational structure*, *sustainable development in construction*, *innovation in building materials and methods*, *technical infrastructure and usage of information technology* and *barriers, challenges and opportunities for sustainable construction*. First, general company characteristics were sought which include the general functions of service areas of the organizations, size of the organizations involving the production, firms' turnover, number of permanent employees, human resources and development and target group of customers. Second, sustainable development in construction were focused. This portion of the questionnaire was used to analyze the development of best practices in construction in terms of social and economic sustainability considering the environmental impact. Third, innovation in new building materials and methods were sought. This portion of the questionnaire was used to determine the current use of new building materials and investment on high-tech methods and techniques. Fourth, technical infrastructure of organizations and their usage of Information Technology were sought. This portion of the questionnaire was aimed to find out the organizations' investment on Hardware, Software, Operating Systems, Networks (LAN, Intranet, Extranet) and communication systems between the functional departments within the organization. Fifth and finally, barriers, challenges and opportunities for sustainable construction were focused. This portion of the questionnaire was used to analyze the potential barriers existing in the construction sector such as lack of capacity of the construction sector, an uncertain economic environment, lack of accurate data, lack of interest of stakeholders for sustainability and lack of Integrated research. Also the challenges and opportunities available for the sector involving public awareness, making sustainability a priority, effective use of resources and improvement of the quality of the construction process and methods were analyzed.

The questionnaire was designed using a nominal scale for the real values of the independent variables. In evaluating the dependent variables, a scale of 4 intervals (with a '0' value given to no effect, '2' to a middle value, and '4' given to maximum effect). The respondents were asked to check a number on the scale, which reflects their assessment regarding the different factors. A list of all contractor organizations within the construction sector was obtained from the Association of Building Contractors. The list consisted of a total of 35 organizations. An attempt was made to contact every single organization. During this particular survey 30 organizations were contacted and 20 (%66) of these questionnaires were evaluated. Contact personnel in the companies for the questionnaire survey were either the top management or senior management in their respective departments, therefore their level of knowledge expected to provide responses was acceptable for the purpose of validity of the survey results.

3 FINDINGS

This section of the study discusses the barriers, opportunities and challenges for sustainable construction practices and performance in pursuing an overall development in the context of globalization and competitive strategies for the construction industry in North Cyprus.

3.1 Determination of Importance Indices

The participating contractors provides numerical scoring factors expressing their opinions on the significance of each factor in determining the factors effecting of development of best practices in terms of social and economic sustainability considering environmental impact in North Cyprus Construction Industry. The weighted average for each factor was calculated and then it was divided by the upper scale of the measurements in what is referred to as "important index" therefore the level of important of the fifteen factors of the eight phases of the Building Life Cycle were calculated using the formula [Kish, 1965]:

Level of Importance (Index) = $[\Sigma(aX) \times 100] / 4$

a= the score given to the factor by each organization (varying from 0-4)
X= n/N
n= Frequency of organizations
N= Total number of participant organizations

Table 1 shows a matrix of variations in level of important indices of the factors during phases of Building Life Cycle. The X-axis of the matrix indicates the process categorized into eight phases. The fifteen factors believed to have influences on the Building Life Cycle were listed in the Y-axis of the matrix with their index values. The matrix also includes the calculated sum, mean and the rank orders of all the phases of the process listed at the bottom of X-axis with their index values. Studying the matrix the first three factors carrying the highest level of importance are; "Efficiency of Use", "Education, Health and Safety" and "Participation and Control". In observing the three highest ranked phases of the process, it can be noted that all these phases carry almost similar level of importance. These are; "Construction", "Design" and "Target Setting". Overall, the factors have the highest importance indices in the phases "Construction" with a sum of 756 points. The factors carry the lowest level of importance indices in the phase "Demolition/Refurbishment" with a sum of 437.

3.2 Discussion of Survey

Both the structured interviews conducted to senior management in their respective departments and as well as the construction site observations conducted by research members during the survey study were relied on for the validity of the survey results.

The factor "Efficiency of Use", is ranked #1 and is perceived by respondents to have an influence on the phases "Briefing", "Site Analysis", "Target Setting", "Design" and "Construction" with a value of importance index 42.75. "Handover", "Occupation", "Demolition/Refurbishment" are the rest of the phases which have the lowest value of importance index 14.25. The interviews and observations highlighted that sustainable development (including technology specified) is designed and managed to be highly efficient and effective, achieving high productivity levels with few resources and limited waste and pollution. Briefing helps to establish the level of commitment by the client to ustainable development at the outset of the project and enables the client and the design team to monitor design development against an explicit performance brief. There is a shared understanding about sustainable development and strong commitment to addressing this amongst all stakeholders in the project. Site analysis aimed to ensure that the project, where possible, helped to address local social, nvironmental and economic problems, using, where available and appropriate, local resources. Target Setting is used to develop detailed sustainable development performance targets for the building. This draws on a range of information The site analysis is used to provide a description of the local context in terms of problems and resources. Finally benchmark performance figures for similar buildings in similar contexts are required. All of this information is then used to develop a detailed set of achievable, but challenging performance targets for the building. Design development enables the performance of different designs and strategies to be evaluated and the best options chosen. Extensive monitoring of site and construction processes is also required. Sustainable development issues should be addressed in tender documents and covered by the contract and a detailed briefing to the contractor. This enables the contractor to understand the requirements of the project and provides recourse if these requirements are not met.

		RANK	FACTORS \ PROCESS		Briefing	Site Analysis	Target Setting	Design	Construction	Handover	Operation	Demolition / Refurbishment
Sum	324		Occupant comfort	SUM	36	42	42	52	52	42	29	29
Mean of Imp.	11.4	4		IMP.INDE	28.5	28.5	28.5	42.75	42.75	28.5	14.25	14.25
				MEAN	2	2	2	3	3	2	1	1
Sum	311		Inclusive Environments	SUM	36	42	42	52	52	29	29	29
Mean of Imp.	10.7	5		IMP.INDE	28.5	28.5	28.5	42.75	42.75	14.25	14.25	14.25
				MEAN	2	2	2	3	3	1	1	1
Sum	287		Access to Facilities	SUM	36	36	30	46	52	29	29	29
Mean of Imp.	9.98	10		IMP.INDE	28.5	28.5	14.25	42.75	42.75	14.25	14.25	14.25
				MEAN	2	2	1	3	3	1	1	1
Sum	334		Participation and control	SUM	36	42	52	52	52	42	29	29
Mean of Imp.	12.1	3		IMP.INDE	28.5	28.5	42.75	42.75	42.75	28.5	14.25	14.25
				MEAN	2	2	3	3	3	2	1	1
Sum	335		Education, Health and Safety	SUM	36	49	49	57	54	30	30	30
Mean of Imp.	12.1	2		IMP.INDE	28.5	42.75	42.75	42.75	42.75	14.25	14.25	14.25
				MEAN	2	3	3	3	3	1	1	1
Sum	280		Local economy	SUM	36	29	30	46	52	29	29	29
Mean of Imp.	9.26	12		IMP.INDE	28.5	14.25	14.25	42.75	42.75	14.25	14.25	14.25
				MEAN	2	1	1	3	3	1	1	1
Sum	348		Efficiency of use	SUM	49	49	49	57	54	30	30	30
Mean of Imp.	12.8	1		IMP.INDE	42.75	42.75	42.75	42.75	42.75	14.25	14.25	14.25
				MEAN	3	3	3	3	3	1	1	1
Sum	299		Adaptability and flexibility	SUM	36	36	36	52	52	29	29	29
Mean of Imp.	10.7	7		IMP.INDE	28.5	28.5	28.5	42.75	42.75	14.25	14.25	14.25
				MEAN	2	2	2	3	3	1	1	1
Sum	293		Ongoing costs	SUM	36	36	36	46	52	29	29	29
Mean of Imp.	10.7	9		IMP.INDE	28.5	28.5	28.5	42.75	42.75	14.25	14.25	14.25
				MEAN	2	2	2	3	3	1	1	1
Sum	296		Capital costs	SUM	36	36	36	49	52	29	29	29
Mean of Imp.	10.7	8		IMP.INDE	28.5	28.5	28.5	42.75	42.75	14.25	14.25	14.25
				MEAN	2	2	2	3	3	1	1	1
Sum	247		Water	SUM	29	29	30	36	36	29	29	29
Mean of Imp.	7.13	15		IMP.INDE	14.25	14.25	14.25	28.5	28.5	14.25	14.25	14.25
				MEAN	1	1	1	2	2	1	1	1
Sum	264		Energy	SUM	36	29	30	36	46	29	29	29
Mean of Imp.	8.55	13		IMP.INDE	28.5	14.25	14.25	28.5	42.75	14.25	14.25	14.25
				MEAN	2	1	1	2	3	1	1	1
Sum	257		Waste	SUM	29	29	30	36	46	29	29	29
Mean of Imp.	7.84	14		IMP.INDE	14.25	14.25	14.25	28.5	42.75	14.25	14.25	14.25
				MEAN	1	1	1	2	3	1	1	1
Sum	305		Site	SUM	36	42	36	52	52	29	29	29
Mean of Imp.	10.7	6		IMP.INDE	28.5	28.5	28.5	42.75	42.75	14.25	14.25	14.25
				MEAN	2	2	2	3	3	1	1	1
Sum	281		Materials and Components	SUM	36	30	30	46	52	29	29	29
Mean of Imp.	9.26	11		IMP.INDE	28.5	14.25	14.25	42.75	42.75	14.25	14.25	14.25
				MEAN	2	1	1	3	3	1	1	1
				Rank	5	4	3	2	1	6	7	8
			Phases carrying the highest levels of Importance	Sum	539	556	558	715	756	463	437	437
				Mean Imp	64.13	59.38	59.38	92.63	97.38	38.00	33.25	33.25

Table 1 Matrix showing the variations in level of important indices of the factors

The factor "Education, health and safety", is ranked #2 and is perceived by respondents to have an influence on the phases "Site Analysis", "Target Setting", "Design" and "Construction" with a value of importance index 42.75. "Briefing" is the phase which have the second highest value of importance index 28.5. The interviews and observations highlighted that development improves levels of education and awareness, including awareness of sustainable development. Sustainable development considers human rights and supports improved health, safety and security. Site Analysis investigates the site in terms of Social, Economic and Environmental aspects in order to establish the context in terms of problems to be addressed and potential resources that can be used. Target setting uses outputs from the briefing workshop to establish the level of commitment by the client to sustainable development and the capacity and understanding within this area by the design team. The design is to be tested back against the target document. This will support the rapid evaluation of different decisions as it enables the 'performance' of an approach to captured readily. Construction monitoring includes tender documentation and site checklists.

The factor "Participation and Control", is ranked #3 and are perceived by respondents to have an influence on the phases "Target Setting" , "Design" and "Construction" with a value of importance index 42.75. "Briefing", "Site Analysis" and "Handover" are the phases which have the second highest value of importance index 28.5. The interviews and observations highlighted that sustainable development supports interaction, partnerships and involves, and is influenced by, the people that it affects. Participation of people is the key to achieving decisions needed to secure changes in the consumption patterns of the majority of the population. It is important to develop campaigns that on one hand inform the public regarding the benefits and opportunities of the use of environmentally friendly building materials and products, and on the other, encourage the change of consumer habits towards a more sustainable use of resources.

The key issues identified from the analysis of the the construction firms adopting to competitive strategies as to benefit from the globalization of the North Cyprus construction industry for their sustainability and are as follows:

- Improving, modernizing and re-introducing traditional construction technologies, building designs and settlement patterns.
- Using cultural tourism as an aid for the preservation of traditional architecture and cultural practices.
- Developing financing mechanisms to enable and encourage use of sustainable technologies.
- Encouragement of alternative financing mechanisms for low-cost housing.
- Lack of institutional capacity to deal with implementation, monitoring and evaluation for sustainable construction.
- Improving the skills levels and capacity of local construction industry role players.
- Integration of sustainability awareness into mainstream education.
- Mechanisms for the transference of knowledge from research institutions to the market, government and professionals.

4 CONCLUSIONS

This paper presents a survey study that identifies the key issues and challenges facing sustainable construction in North Cyprus. According to the findings of the research, the development of best practices in terms of economic sustainability: improving profitability and predictability, decreasing defects, quality improvements and leading to greater client satisfaction are required. These quality improvements can be equated to improvements in social and environmental sustainability. It is found that the factors "Efficiency of Use", "Education, Health and Safety" and "Participation and Control" hold the highest level of importance. From the processes point of view, "Target Setting", "Design" and "Construction" are the three phases which the interviewees believed would highly be influenced by the factors mentioned above.

The interviews and observations highlighted that the process of globalisation is inevitable. Considering the strategies that are commonly used in teaming up with foreign firms, *joint venture* and *partnering* are the most beneficial strategies in construction services. Joint venture would be a preferable strategy because it enables collaboration between the parties involved on a long-term basis, as compared to partnering that is often limited to one-off projects. Joint ventures are formed to provide global market change where each partner brings specialist expertise. This assists in searching out cheaper sources of material, equipment and personnel outside the home country.Therefore, if proper mechanisms are put in place, these global strategies could benefit construction firms in developing countries.

It can be concluded that the socio-economic components of sustainable construction are viewed to be the most challenging in North Cyprus. Construction industry hold knowledge and values that can contribute to a new vision for development, as well as the practical know-how needed to make it work. Government should develop and enforce legislation for the application of government standards on sustainable practices related to planning and implementation of actions concerning aspects like water, transport and traffic, energy, building material, waste and natural resources. Building companies should implement quality norms, performance based standards and ISO 14000 in order to promote changes that could result in the improvement of management and production. Construction industry should encourage sustainable housing construction and urbanisation by promoting the use of appropriate materials. North Cyprus should adopt a posture of selective participation in world trade and the development process, adapted to the specific socio-cultural and environmental contexts of each region. To ensure sustainable construction at all levels, long-term strategies must envision the processes of manufacture, the use of appropriate technologies and appropriate materials, and the creation of sustainable livelihoods. There is a need to develop specific local solutions to problems of natural and built environment quality. The construction sector must also begin to address the development, not just of appropriate construction materials but also appropriate technology that recognises the need to save resources and is cost effective.

5 REFERENCES

Ebohon, O.J. (1996) 'The scope and limits of sustainable development in Africa's built environment sector.' The International Journal of Sustainable Development and World Ecology, Vol.3, No.1.

Gibberd, J. (2003) Integrating Sustainable Development into Briefing and Design Processes of Buildings in Developing Countries: An Assessment Tool. Unpublished thesis.

Kish, L. (1965), Survey Sampling, New York, John Wiley & Sons Inc., pp. 162.

Ngowi A.B and Lema N. (2002), Impact of globilisation on the sustainability of construction firms in developing countries, The 1st International Conference of CIB W107- Creating a Sustainable Construction Industry in Developing Countries, 11-13 November 2002, South Africa.

Ngowi, A.B. and Rwelamila, P.D. (1999) What is a Competitive Advantage in the Construction Industry? Cost Engineering, Vol.41/ No.2, pp.30-36.

Plessis, C.D. (2001), Agenda 21 for Sustainable Construction in Developing Countries, *First Discussion Document*, CIB, CSIR.

Porter, M.E. (1990) The Competitive Advantage of Nations, The Macmillan Press Ltd., London.

Rodwin Lloyd ed. (1987) Shelter, Settlement and Development, UNCHS (Habitat), Nairobi.

Yitmen, I. and Alibaba, H.Z. (2004), The Impact of Information Technology Usage on North Cyprus Construction Industry, 3rd International Symposium – *Creating the Future,* 25-26 November 2004, Faculty of Architecture & Engineering, European University of Lefke.

Yitmen, I. and Taneri, C. (2005), Impact of Culture on Construction Performance: North Cyprus Construction Industry, CIB W92 Symposium 2005, International Symposium on Procurement Systems, Construction Procurement - *The impact of Cultural Differences and Systems on Construction Performance*, February 8-10 2005, Las Vegas, Nevada-USA, pp. 71-78.

Yitmen, I. and Öztürk, G.B. (2005), Challenges of Lean Construction for Competitive Strategy: North Cyprus Construction Industry, CIB W92 Symposium 2005, International Symposium on Procurement Systems, Construction Procurement - *The impact of Cultural Differences and Systems on Construction Performance*, February 8-10 2005, Las Vegas, Nevada-USA, pp. 521-528.

United Nations (2002), World Summit on Sustainable Development, Plan of Implementation, advance unedited text. http://www.johannesburgsummit.org/index.html 11/11/02 10.32.

The Swiss Vision: Closed Loop in Building Waste Management

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ABSTRACT

The Swiss construction consumes 60 million tons per year of new material resources, while deconstruction amounts to only 18% of this mass. This results in a permanent growth of the building substance at a rate of 2.3% per year.

The Swiss building process was analysed in view of a future sustainable development (agenda 21): "Business as usual" creates a constantly growing building mass. The vision "Closed loop" bases on equal building and deconstruction. To reach this state, the reused and recycled quantity of building waste has to be increased by more than seven times of its present mass. The vision causes concern in the Swiss building industry, as demanding and expensive new technologies in construction, deconstruction and recycling have to be developed. Furthermore, on top of the legal and pecuniary guides and restrictions already in force, some more stringent will follow.

Empa has compiled a state of the art report and has identified areas, which require substantial further research and development. This paper mainly covers the consequences for the recycling of mineral building waste: While the reuse of concrete is state of the art, granular material of inferior quality from brickwork is still deposited in landfills or used for subordinate functions. Empa proposes using granular material from clay and calcium silicate bricks as components for concrete and form bricks. Tests are positive and first building applications, taking into account the specific properties of this new material, are under way.

Reaching the closed-loop scenario, however, requires a joint effort: education of planners, owners, investors and builders, research for practicable methods of characterisation of (mineral) building waste and reliable production of concrete, form bricks and other building products from this waste. Design methods for the reliable use of the recycled products. With regard to further legal measures and their economical implications, the discussion is open.

KEYWORDS

Building waste management, recycling, concrete, sustainability

1 INTRODUCTION

Switzerland is a small country and its habitable areas are densely populated. In the last 50 years, heavy building activities have been going on, leading to a massive stock of buildings of 2'100 million tons [Mt] (300 tons per inhabitant).

The reasons for this development are mainly the following:

- Prosperity has led to double the living area per person from 1950 to 1990,
- Building out in the green is often easier than refurbishment and adaptation of run down old buildings for new uses (often protected for historic purposes),
- Building regulations still permit the allocation of new areas for development and building.

A study group had been set up and sponsored by the Swiss building industry and by institutional building owners, with the goal of an analysis of the situation and scenarios for a sustainable development [Schneider 2002]. This paper shows the consequences of the study, the steps taken in the past and the efforts still required, in order to reach a sustainable steady state in the Swiss building sector within the decades to come.

2 PRESENT SITUATION

In Switzerland annually 68 Mt of new material resources are consumed by the construction industry, i.e. some 10 t per inhabitant. Of this material, 90% originates from primary resources. Deconstruction today amounts to 11 Mt per year [Mt/a], i.e. 18% of the consumption. Of this quantity, 4.5 Mt/a are reused directly and 3 Mt/a are reused after recycling treatment. A total of 68% (7.5 out of 11 Mt/a) of the deconstruction material is reused. The remainder of 3.5 Mt/a (32% of deconstruction or 5% of construction) is disposed of in landfills and incinerating plants, see Fig. 1.

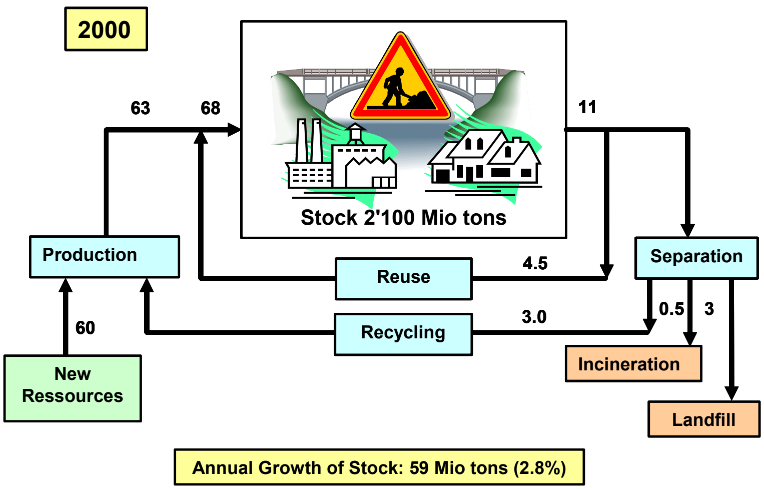

Figure 1: Streams of construction material at present in million tons per year

The study on the Swiss management of construction material [Schneider 2002], estimates that without further measures or restrictions, see scenario: "business as usual" in Fig. 2, in 2050 some 69 Mt/a are consumed by building activities (75% originating from primary resources). Deconstruction rises to 20 Mt/a and, in accordance to the expected relatively mild restrictions, 2.1 Mt/a are disposed of. This results in a permanent growth of the building substance from today's 2'100 Mt, at a rate of 2.3% per year, to more than double the value of 4'900 Mt in 50 year's time.

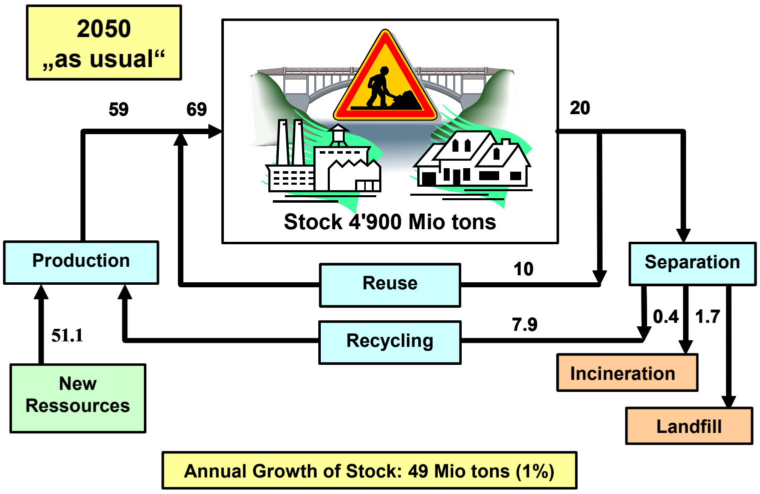

Figure 2: Streams of construction material for "business as usual" in 2050 in million tons per year

3 VISION: CLOSED LOOP

In a small and densely populated country like Switzerland, the building process cannot go on forever but has to be transformed into a sustainable steady state within the next decades, as on the one hand, resources are depleting and on the other hand, more and more land is consumed by man-made structures.

In the future land for new developments will be scarce, leading to an increasing rate of deconstruction and to a respective stream of deconstruction material. Finally, due to the influence of steering measures (duties and dumping/incinerating restrictions) a nearly steady state is reached where the same mass of 65 Mt/a is constructed and deconstructed. At the present knowledge, it is estimated that 5 Mt/a of material cannot be reused and have to be disposed of. The same mass of new material is fed into the stream, creating a nearly closed loop, see Fig. 3.

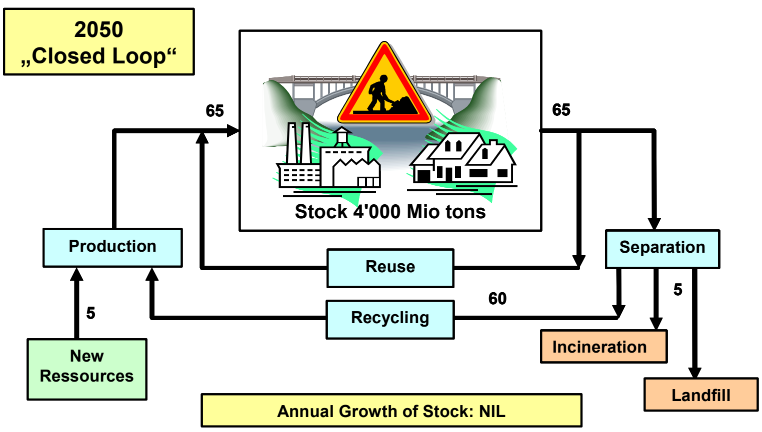

Figure 3: Streams of construction material for the vision "Closed Loop" in 2050 in million tons per year

Compared to the entire deconstruction material, reuse and recycling have to increase from 68% to 92% (60 out of 65 Mt/a). The mass of the material to be recycled and reused, however, increases from 7.5 to 60 Mt/a (factor 7.5)! This is quite a challenge, not only for the building industry, but also for society! The transition from the present state to this closed loop scenario within the next 50 years still results in an increase of the building mass in Switzerland to 190% of today's mass before stabilizing.

4 STATE OF THE ART IN REUSE

Empa has compiled a state of the art report together with a list of research needs and proposals for economic and legalistic measures in order to move towards the vision of the sustainable closed loop material management [Moser *et al* 2004].

4.1 Mineral construction materials

General
The main quantities of building materials used in Swiss construction are shown in Table 1. Deconstruction yields some 7 Mt of mixed waste (mainly brick rubble) and some 7 Mt of concrete rubble per annum.

Mineral Material	[Mt/a]	*Other material*	[Mt/a]
gravel & sand	51	bituminous mix	4,8
clay products concrete blocks calcium silica bricks	1,7 0,4 0,3	timber polymers	0,70 0,15
concrete (≈ 10 Mio. m^3/a) cement	25 3,5	sheet glass	0,08

Table 1: Quantities of material used in Swiss construction

Concrete
Concrete rubble is broken down to granulate, washed and sieved. Often, fines are extracted and natural sand is added mainly for ease of quality assurance. Reinforcement is extracted and introduced into the steel recycling process.

The reuse of concrete granulate can be considered as state of the art. Mixed rubble (e.g. from brickwork) is still deposited in landfills or used for very subordinate functions although the present Swiss code for concrete permits aggregates to contain recycling material (concrete and/or brickwork) up to 25% by mass for standard concretes (Swiss Appendix to [SN EN 206-1:2000]).

As deconstruction increases, more mixed building waste will be available. Empa has done research using granular material from clay and calcium silicate bricks as components for concrete and for form bricks. First experiments were done with laboratory mixes (A&B) and one mix (C) directly taken from a concrete manufacturer’s yard, see Table 2. Cement content and water/cement-ratio varied over a wide range [Olbrecht 1999].

Granulate mix	*Clay brick*	*Calcium silicate*	*Concrete*
A	40	30	30
B	70	15	15
C	7		93

Table 2: Mixes for recycling concrete [% by mass]

The degree of compactability [EN 12 350-4] ranged from 1.08 to 1.18. Mixes included plain concretes (without additives), mixes with plasticizer and mixes with plasticizer plus air entraining agent. In Figure 4a, the compressive strength is plotted versus the respective water/cement-ratio. Two straight lines show possible relations of the compressive strength in function of the water/cement-ratio. The dotted line below shows the same relation for alluvial material. The compressive strength is mainly depending on the w/c-ration of the hardened cement paste. The mixes containing brick rubble show a considerable higher w/c-ratio than the corresponding standard concrete. The extra water, included in the overall w/c-ratio, is absorbed by the porous aggregates and accordingly, the effective w/c-ratio in the cement paste is smaller. Concrete strengths in the range of 35 MPa are deemed practicable for applications. The elastic (Young's) modulus is plotted against cube strength in Fig. 4b.

Concretes from alluvial material show elastic modules $E \approx 11'000)\sqrt[3]{f_c}$, about double the value of the mixes A and B.

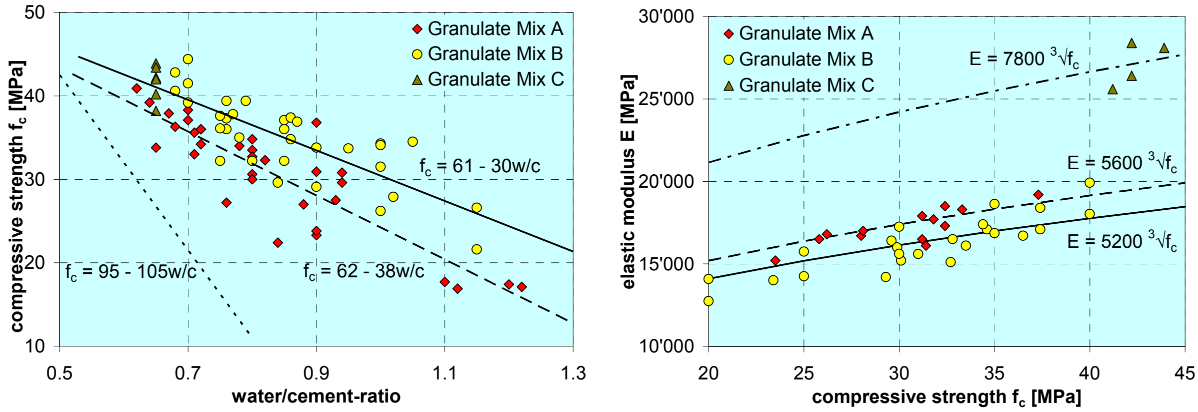

Figure 4: Concrete from mixed building waste: a) compressive cube strength versus w/c-ratio and b) elastic modulus versus compressive cube strength

Using the mixes A and B, reinforced concrete beams were produced and tested in bending. Deflections were larger than for standard concrete, corresponding to the smaller elastic modulus as shown above. Two further beams are still subjected to long term loading for creep measurements. Creep appears to reach about double the creep of standard concrete. Applications of concrete for load bearing members in buildings, using concrete containing relevant fractions of brick granulate are under way.

Form blocks were also produced and brick wall specimens were tested. Their load capacity was high, compared to standard brickwork, mainly due to the smaller void ratio of the prototypes.

Roads
Sand and gravel from deconstructed roads, is reused to 97%, the same rate applies to bitumen- and tar-bound layers (95%). Here some restrictions apply to the reuse of tar-bound mixes due to the release of poly-aromatic hydrocarbons during the process. Research has bee carried out on the permissible content for reuse [Richner & Hugener 2004].

Sheet Glass
Glues, sealants and coatings contaminate most sheet glass. For these reasons, sheet glass is generally disposed off in landfills.

4.2 Timber

The consumption of timber in Switzerland amounts to about 70% of the total timber growth. The life cycles of timber are quite well established although some fractions of disposal are based on estimates (Tab. 3). Today, the disposal of combustible waste is prohibited in Switzerland and some of the figures will change accordingly.

Direct reuse of timber is only of minor importance, mostly due to contaminations or due to the effort of defining and controlling acceptable levels of contamination. The exported timber serves the production of chipboard, which again is imported back. The remaining quantity of the timber is thermally disposed of in incinerating plants and the energy serves the generation of energy.

Source	[t/a]	*Disposal*	[t/a]
Building parts and products	300'000	Incinerating plants (estimate)	30'000
		Landfill (estimate)	220'000
Furniture and woodwork	235'000	Heating	80'000
		Export	160'000
Packaging material	165'000	Illegal disposal (estimate)	210'000
Total	700'000	Total	700'000

Table 3: Sources and disposal of timber

4.3 Polymers

Polymers are used in the Swiss construction industry since the 1950ies. Consumption increased almost linearly up to the present mass of 120’000 to 150’000 t/a (various sources, e.g. [Buwal 1995]), representing about 15% of the total consumption of 900’000 t/a in Switzerland. A large variety of polymers is used (Tab. 4).

Polymer		*Application*
PB	Polybuten	Piping
PE	Polyethylene	piping, sealing sheets for landfills)
PVC-U	Hard-Polyvinylchloride	Piping, light domes, web plates, shutters
PVC-P	Soft-Polyvinylchloride	Flooring, sealing sheets, seals
PP	Polypropylene	Piping, various applications
PS	Polystyrol	EPS, XPS: insulating foam
PMMA	Polymethylmethcrylat	Light domes, web plates
PC	Polycarbonate	web plates
TPE	Thermoplastic Elastomere	Seals, sealing sheets

Table 4: Polymers used in building

The variety of polymers and their use in small quantities dispersed all over the buildings do make recycling very difficult if not impossible. For these reasons, in Switzerland polymers are also disposed of thermally, also generating energy mainly saving oil, the primary energy resource.

4.4 Metals

Various metals are used in relatively large quantities:

- Steel is used in steel constructions and for reinforcement or prestressing.
- Aluminium is used in light metal constructions, on facades and in electrical installations
- Copper and brass are used as roofing, on facades and in electrical installations,
- Zinc as corrosion protection, etc.

Switzerland consumes 1.1 Mt/a of steel produced by melting. Production wastes amount to 21% and 79% are reclaimed from scrap metal. This source again divides into 0.23 Mt/a from private households and 0.82 Mt/a from industry. The total mass of steel scrap amounts to 1.3 Mt/a, exceeding the total consumption! In Switzerland today, full recycling or reuse is a fact with metals, due to their relatively high cost and ease of separation.

5 RESEARCH NEEDS

5.1 Mineral construction materials

5.1.1 Concrete
Regionally, more granulate is produced than can be sold as recycling concrete. The Swiss appendix to the new European code on concrete [SN EN 206-1 2000] permits granulates for standard concretes to be made of 25% of concrete or mixed granulate. This should enable producers to use up their stocks.

The implications of this rule on the quality of concrete have not been yet investigated in a general sense. Practical tools for producers and for designers are required containing the necessary technical information for the safe application of this new product. At present, it is up to the producer to show the conformity to the applicable standards and the designer might be left in the dark on properties deviating from his design assumptions.

Most research published covers recycling concrete using granulate from crushed concrete. For concrete made of mixed granulate containing considerable fractions of brick rubble, the following topics have to be addressed by research projects:

Production of concrete	*Design and construction*
• Microstructure • Durability • Contaminations: - Definition of detrimental components, - Permissible contents, - Methods for on site determination • Optimisation of materials processing for distinct quality levels • Deformation behaviour • Etc.	• Investigation of bond to reinforcement • Behaviour under shear • Punching behaviour • Behaviour under long term loading • Stability of columns • Modification of design methods • Applicable partial safety factors • Etc.

Table 5: Research needs for the use of recycling concrete containing mixed building waste

These projects should lead to clear guidance for the application of recycling concrete consisting of mixed building waste, starting from the processing of the material through to the design for safety and for service life, e.g. durability.

5.1.2 Asphalt mixes
Various projects are under way to solve some of the open questions listed in Tab. 6.

• Behaviour of multiply recycled surfaces (high content of bitumen, ageing of binder, etc.) • Optimum methods for cold and in-place recycling • Other applications for asphalt granulate • Methods for on-site determination of PAC-content and of contaminations • Etc.

Table 6: Research needs for the use of recycling asphalt

5.2 Timber and Polymers
The present solution of the mainly thermal exploitation of timber and polymer (plastics) waste does not leave important questions open. Ongoing Research concentrates on environmentally friendly construction methods.

6 INCENTIVES AND LEGAL MEASURES

In Switzerland permission for new areas for landfills are difficult to obtain and require procedures, which usually take more than 10 years. Resulting costs of deposits strongly influence material streams.

The fastest way of implementing new techniques is via incentive clients and authorities. The next possibility is via information and teaching of owners, clients and planners.

Generally, restrictions for incinerating or depositing building waste are the most efficient means for speeding up recycling. Dumping of treated timber is prohibited and incineration is permitted in incinerating plants only, at relatively high costs. Dumping of reactive material is charged with the highest duties. Deposits of non-reactive material are at present still permitted, except for concrete, which has to be recycled completely. It is envisaged to extend this restriction to all mineral waste.

It must be admitted though, that enforcement is best in the main centres and less adhered to in other regions of Switzerland.

7 CONCLUSIONS

The Swiss construction industry is forced by pecuniary and legalistic measures towards a closed loop system based on sustainability considerations.

On building sites, separation of the waste materials is required by regulations and enforced by the authorities. Metals (aluminium, steel, copper, etc.) are separated on site and recycled entirely. Switzerland collects at present more steel scrap than steel is consumed. Asphalt pavements and concrete are fully recycled as well. Timber and polymers are mainly disposed of thermally, thus saving primary energy sources. Part of the mixed mineral waste today is reused in downgraded functions. The bulk of this waste, however, can only be recycled after substantial research provides the engineering society with the necessary application tools and knowledge. The feasibility has been shown and some public owners are currently fostering pilot projects using mixed building waste for the production of concrete.

Economic problems are foreseeable because of the higher costs of Swiss building materials, resulting in imports from the surrounding countries with less stringent regulations and laws.

8 ACKNOWLEDGMENTS

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9 REFERENCES

BUWAL 1995: *Umweltgefährdende Stoffe: Richtlinie Bauprodukte und Zusatzstoffe in der Schweiz*, Buwal: Bundesamt für Umwelt, Wald und Landschaft, Bern.

EN 12350-4:1999, Testing of fresh concrete – Part 4: Degree of compactability, CEN, Brussels.

Hoffmann C. 2004: *Materialkenngrössen von Beton aus Mischabbruch* (in German; Material characteristics of concrete from mixed building waste), EMPA-Report 203600.

Moser K., Bertschinger H., Czaderski C., Hugener M., Kramer H., Richner P., Richter K. 2004, *Baustoffmanagement 21 an der EMPA, Stand des Wissens und Forschungsbedarf* (in German; Building material management at EMPA: State of the art and research needs), EMPA Report, published by novatlantis, Dübendorf, September.

Olbrecht H., Leemann A. 1999: Beton aus Mischabbruch, Recyclingtechnik, BR6/1999, pp. 50-52.

Richner P., Hugener M. 2004: *Closing the Loop in Construction Waste Management: the Role of Interdisciplinary Research and Transdisciplinary Research and Innovation*, Building for a European Future, Maastrich, pp. 497-509.

Schneider Th. 2002, *Baustoffmanagement 21: Strategie für die Realisierung eines nachhaltigen Bauwerkes Schweiz* (in German; Construction materials management 21: Strategy for the realisation of a sustainable Building Switzerland), Plattform für nachhaltiges Bauen, Gruppe strategische Grundlagen, Ernst Basler & Partner, Zürich.

Schneider Th. 2002a, *Umsetzung Strategie Baustoffmanagement 21: Vorschlag zum weiteren Vorgehen* (in German; Implementation of the construction materials management 21: Proposal for further actions), Plattform für nachhaltiges Bauen, Gruppe strategische Grundlagen, Ernst Basler & Partner, Zürich.

SN EN 206-1:2000, Beton – Teil 1: Festlegung, Eigenschaften, Herstellung und Konformität, (Concrete – Part 1: Specification, performance, production and conformity), Schweizer Ingenieur- und Architektenverein, Zürich.

A global methodology for sustainable road - Application to the environmental assessment of French highway

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ABSTRACT

Road construction and maintenance have to be examined not only considering economical and technical factors as it is usually done, but also from an environmental point of view. In this area, natural aggregates as well as bitumen and cement are widely consumed and therefore constitute natural resources to be preserved. Road construction and maintenance processes have to be taken into account in terms of energy and materials consumption as well as emissions towards environment.

In order to perform a global road assessment including environmental approach, the Life Cycle Assessment (LCA) standard methodology proposes a general framework aiming at evaluating the environmental effects of any product, from cradle to grave. This methodology consists in defining a system and a functional unit (FU) to set a base of analysis. Then, the Life Cycle Inventory (LCI) step, consists in assessing flows inputs and outputs (i.e. consumptions and emissions) within the defined systems boundaries. Finally, the LCI can be completed by a Life Cycle Impact Assessment (LCIA) which evaluates potential environmental impacts using indicators. LCA has been widely applied to manufactured products, and also more recently applied to roads.

Road is typically a complex layered structure of different manufactured materials, each having its own life cycle. An overview of the existing LCI studies of road environmental assessment, lead the authors to the conclusions that LCA methodology cannot be directly applicable to road cases. Therefore, in this paper, a specific model, which is based on the LCA principle, called ERM/GRM (Elementary Road Modulus/Global Road Modulus), is presented. Together with the detailed description of this methodology, its application in a French highway as ERM pavement inventory is presented for highlighting the respective contribution of each kind of industrial process to road construction and maintenance and the role of maintenance work in total consumption and pollutant flows.

KEYWORDS

road, life cycle, inventory, assessment

1 INTRODUCTION

Because roads play a major role in society, road construction and maintenance have to be examined not only considering economical and technical factors as it is usually done, but also from an environmental point of view. In this area, natural aggregates as well as bitumen and cement are widely consumed and therefore constitute natural resources to be preserved. Vehicles traffic pollution has been extensively studied in the last decades in Europe [Boiteux 2001]. Besides, road construction and maintenance processes have to be taken into account in terms of energy and materials consumption as well as emissions towards environment. Both environmental pressures and effects can therefore be studied. For example, natural aggregate consumption for the French civil engineering sector reaches 200 million tons per year for roads alone, out of a total produced of 400 million tons [Michel 1997]. Besides, Life Cycle Assessment (LCA) is a well known standardised methodology [ISO 14040 1997] [ISO 14041 1998] [ISO 14042 2000] [ISO 14043 2000] proposing a general framework aiming at evaluating the environmental impacts of any product, from cradle to grave. LCA has been widely applied to manufactured products, and also applied to roads. Previous studies have been performed in order to propose LCI of typical roads (table 1). However, such studies on French roads [Ventura *et al*. 2003] [Ventura *et al*. 2004] showed that LCA could hardly be adapted as a prospective road life environmental analysis tool, including both structure and maintenance of the road with initial construction.

Hence this study is focused on a research concerning a specific global methodology, devoted, as a first step, to the analysis of road pavement environmental assessment. In this context, a literature survey was first done to analyse the pertinence of the existing LCA methodologies applied to roads. Then, a new developed methodology, called ERM/GRM (Elementary Road Modulus/Global Road Modulus) is presented. Finally, an application of the ERM for one French highway pavement environmental inventory during an analysed period of 30 years, including initial construction and maintenance is detailed and discussed.

2 ROADS AND LIFE CYCLE ASSESSMENT (LCA)

2.1 LCA principles

LCA is a standardised methodology proposing a general framework aiming at evaluating the environmental effects of any product, from cradle to grave. This method is mainly used for (i) comparing products giving similar service during their service life, (ii) highlighting parts of the products manufacturing processes that have major environmental impacts, in order to optimise their influence.

The LCA methodology first [ISO 14040] requires to describe the goal, scope of the study and to define a system and a functional unit (FU) to set a base of analysis. Then, the Life Cycle Inventory (LCI) step [ISO 14041], consists in assessing flows inputs and outputs (i.e. consumptions and emissions) within the defined systems boundaries. Finally, the LCI can be completed by a Life Cycle Impact Assessment (LCIA) step [ISO 14042 and 14043], which evaluates potential environmental impacts using indicators. Several literature studies have been focused on roads LCI and present environmental flows. An overview of hypotheses discussed in literature is given in next section.

2.2 Overview of LCA applied to case studies

Table 1 shows informations on existing road LCI studies. The authors and references deal with three main types of pavement materials (asphalt concrete, cement concrete and mixed) with differences between studied structures. Only references N°3, 6, 8 account for scenarios using recycled material: reclaimed asphalt pavement, blast furnace slag, and crushed concrete waste. Most of the studies consider a 1 km length road section. Cases N° 6, 8 base their analysis on a pavement surface unit. Except for case N°6, road maintenance works are taken into account during the analysed period. That latter varies from 30 years to 50 years. Studies N° 3, 5 consider road end of life, consisting of road demolition at the end of analysed period. It can be seen from this literature review, that the FU of a road LCI study requires many details (i) a length of road, (ii) a complete description of road structure (materials and geometrical configuration of layers), (iii) a period of analysis, and (iv) a description of maintenance policy. This FU implies to precise a lot of numerical values (length, width of road section; number and thickness of layers, materials properties…). Therefore, results of one case study cannot be generalized. Indeed, the change in one parameter value may involve important variations of environmental inputs and outputs ranges, especially because road products are responsible for huge amounts of flows. Furthermore, LCA methodology does not analyse interaction between the products and the territories where they are manufactured and used [Blanc 2000]. Avoiding territory seems difficult to conceive concerning roads, because they are built and remain inside a territory and technical scenarios often depend on territory specificity. Furthermore, LCI takes the product end of life into account, the principle of which is not in agreement with maintenance policies, planned to avoid road demolition, never occurring in France. Finally, from the analysis of these existing studies, it seems that LCI, which is rather a retrospective way of products environmental assessment, could hardly be adapted as a prospective road life environmental analysis tool, including maintenance. If roads environmental assessment is wished to be included in decision making processes, evaluation methodologies must be applicable to most cases, and a modelling tool becomes necessary. In this context, a new tool dedicated to roads, has been introduced in a previous study [Ventura *et al*. 2003].

N°	Analysed Road element	Analyse period	Maintenance works consideration	Analysed pavements	Recycling consideration	End of life consid eration	Applied Method Phase	References
1	1 km of length	30 years	Yes	CRC, CS, AC, Semi-rigid,	No	No	LCI	[Chappat & Bilal 2003]
2	1 km of length	50 years	Yes	AC, CC	No	No	LCI	[Lundström *et al*. 1998]
3	1 km of length	50 years	Yes	AC	Yes	Yes	LCI	[Mroueh *et al*. 2000]
4	1 km of length	50 years	Yes	AC, CC	Yes	No	LCI	[Pontarollo *et al*. 2001]
5	1 km of length	30 years	Yes	CRC, CS, Mix, AC	No	Yes	LCI	[Peupurtier 2003]
6	1 m^2 of pavement	40 years	No	CRC	No	Yes	LCI	[Rouwette & Schuurmans 2001]
7	1 km of length	40 years	Yes	CC and AC	No	No	LCI	[Stripple 2001]
8	150x3.8 m^2	(*)	Yes	AC	Yes	No	LCA	[Ventura *et al*. 2004]

(*) This study is for a maintenance work of French road.
***CRC**: continuous reinforced concrete, **AC**: asphalt concrete, CC: cement concrete, CS: concrete slab.*

Table 1. Summary of existing road environmental assessment studies

2.3 Towards a new environmental road assessment method

According to the above discussion the developed tool (Figure 2) is based on: LCA methodology (figure 2 left side), whereas a specific geo-spatial model is proposed for the road (figure 2 right side). Geo-spatial modelling successively includes an Elementary Road Modulus (ERM) analysis and a Global Road Modulus (GRM) analysis. This tool separately considers road layers (each with a given life cycle either closed with demolition for upper layers or opened for lower layers) and road structure. Based on LCI, a chosen elementary road modulus (ERM) is developed to perform an inventory of the whole road structure including construction, exploitation and maintenance. The ERM is a modular road element resulting from traffic considerations and French level of service scenarios. It can be composed of different kinds of road pavement layers or earthwork, with various materials. Hence, road construction and maintenance techniques are modelled. The ERM is fully modular in order to be adapted to many

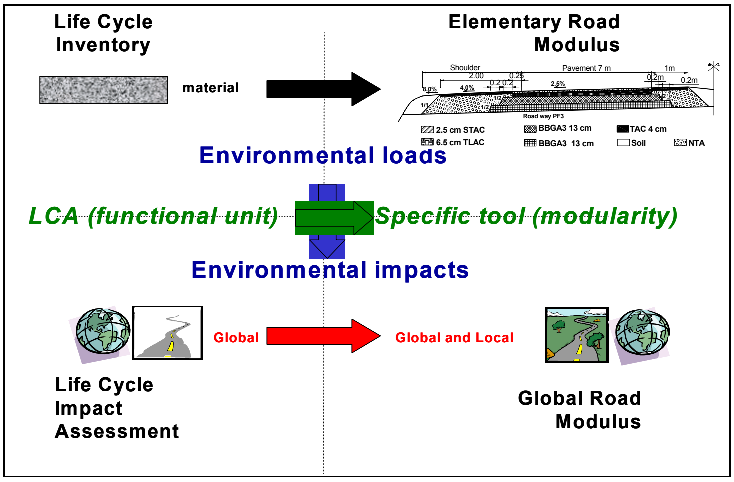


Figure 2: Principle of Road Modulus tool

different cases and, the investigated system includes : road initial construction, exploitation and maintenance. Each period is described through subsystems according to LCI methodology. Hence ERM leads to an environmental inventory of each subsystem input/output parameters. Once environmental loads are determined at the scale of the ERM (either pavement or earthwork at present), a second phase is applied: the Global Road Modulus model. LCIA methodology is considered and adapted to assess environmental impacts. Various kinds of GRM are in progress including indicators that both take into account global impacts and territory aspects. This future development, is not presented in this paper.

3 ERM MODELLING PRINCIPLES

3.1 ERM pavement model

3.1.1 System description

ERM inventory aims at calculating raw materials consumption, energy consumption and pollutant emissions from the following processes (i) raw materials manufacturing and transportation from the processing site to the road works site, (ii) road works equipment used during pavement construction and maintenance. Figure 3 presents the system boundaries of an ERM pavement. All subsystems inside the frame are included into environmental flows calculations. Flows induced by plasticizers, wastes storage and treatment, air entraining agents, and maintenance of equipments, are not included.

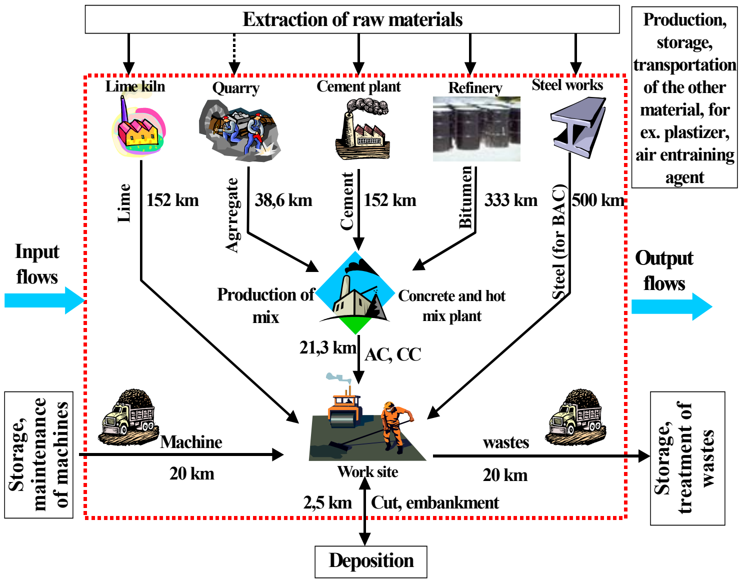


Figure 3. Boundaries and subsystem of the ERM pavement.

3.1.2 Inventory calculations

According to road service life, ERM allows to calculate environmental flows, for long periods of time. As literature available on LCI refers to specific dates, environmental data have to be updated and projected into the considered periods of time of the analysis (present and future). Thus, several assumptions have been set to perform such analysis (i) when several literature data sources were available, their trend with time was examined, (ii) when only one or few literature data sources were available, their time projection has been assumed to follow the Kyoto protocol that is, for the EU, a decrease of the emission of greenhouse gases by 8%, from year 1990 to 2010. Indeed, it is well known that during past periods, consumptions and emissions of many technologies and processes have been reduced. Such trend is then assumed to continue in the future. For years beyond 2010, trend was assumed to be constant and equal to the 2010 values. Figure 4 presents procedure of data harmonisation with time, with 1990 as the reference year. It has been necessary to identify and develop the

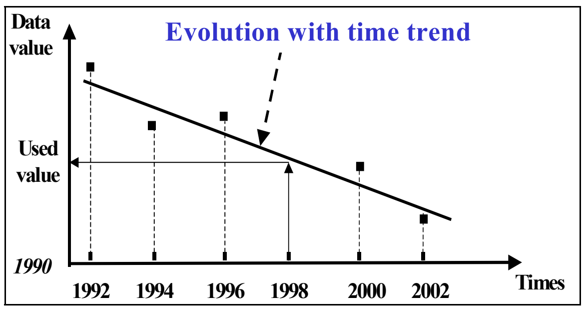


Figure 4. Time harmonisation of environmental data

inventory calculation method for each subsystem of figure 3. Figure 5 presents an example of this method in the case of road equipment engines. These engines are used both at the time of road construction and maintenance. Each equipment running in, has been analysed. Functioning periods of time of road works machines have been calculated from their technical capacities and from pavements characteristics. Total emissions were obtained from unit emissions factors of engines.

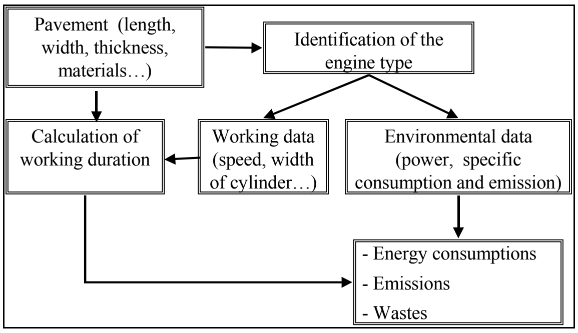
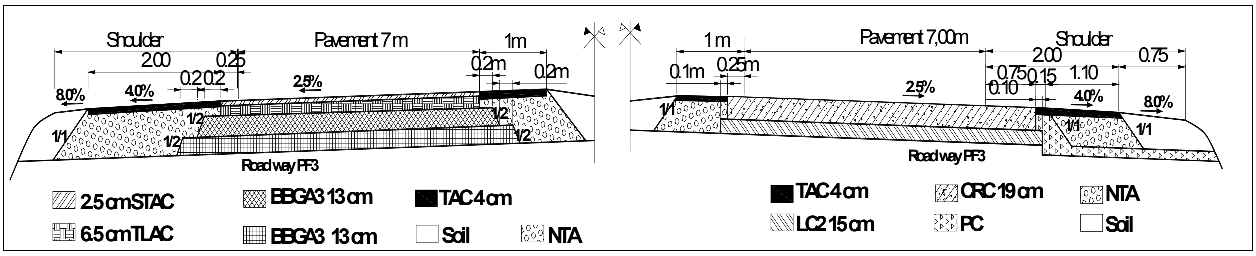


Figure 5. Example for the inventory calculation method- case of the construction engines

4. PRESENTATION OF TWO ERM PAVEMENT CASE STUDIES

4.1 ERM parameters description

An example of ERM application (figure 6) is presented for two different French highways sections of 1 km length, using either Asphalt Concrete (**AC**) pavement or Continuous Reinforced Concrete (**CRC**). Road structures are identified according to the French standard and the technical guidelines [SETRA-LCPC 1998], [LCPC-SETRA 1997] for traffic TC^6_{30} with 2000 heavy lorries by day, by slow lane and by direction. The light vehicles take 80 % of traffic composition, against 20% heavy vehicles; 90% of heavy vehicle are found on the slow lane.



(a) **AC** pavement (b) **CRC** pavement

BBGA3: bituminous-bound graded aggregate class 3, ***CRC****: continuous reinforced concrete, LC: Lean concrete, NTA: non treated aggregate, PC: porous concrete, STAC: super thin asphalt concrete, TAC: thin asphalt concrete, TLAC: thick layer asphalt concrete*

Figure 6. ERM geometrical parameters [SETRA-LCPC 1998]

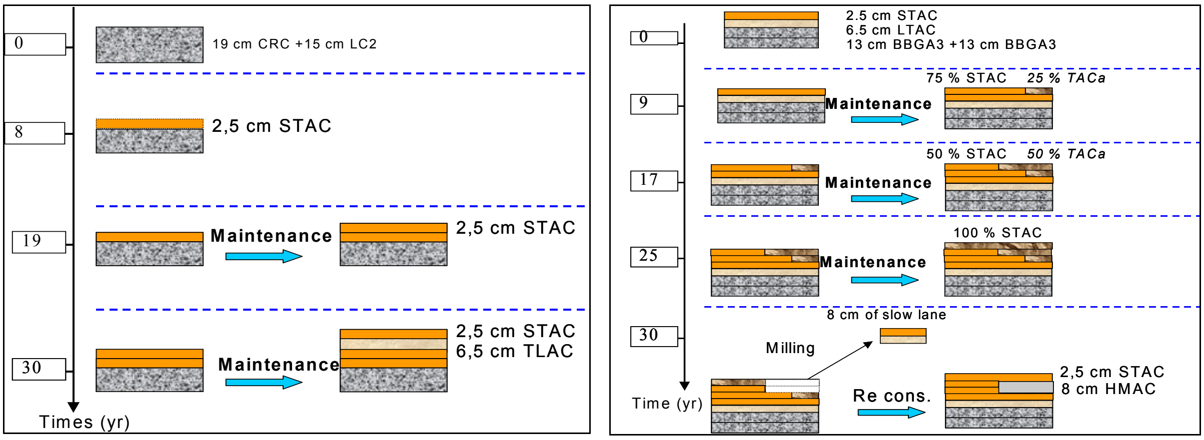
Pavement materials used to calculate resources consumptions and emissions processes are presented in table 2. Environmental data concerning cements were taken from [St-Laurent cement 2003], [Lafarge Cement 2004], [Taiheiyo Cement Corporation 2003], [WBCSD 2002]. These data account for cements in general, and do not detail any particular product. They were assumed to correspond to a pure Portland cement, although in France, blended cements, that contain mineral additives and thus less clinker, are always used in concrete pavements.

Materials	Composition (two lanes / 1 direction)
CRC (for CRC layer)	800 kg sand 0/5 + 440 kg gritting 5/10 (Ryolithe) + 585 kg gritting 10/20 (hard limestone) + 1.65 kg plasticizer + 0.06 kg air entraining agent + 325 kg cement CEM II/A32.5 + 145 l water. % of steel surface in relation to the concrete surface: 0.67 %
LC	860 kg sand 0/5 + 935 kg gritting 5/25 silicate -limestone + 1.25 kg plastizer + 0.025 kg entraining agent + 250 kg cement CEM II/A 32.5 + 170 l water.
STAC	Bitument content : 5.62%
TLAC	Bitument content : 5.2%
BBGA3	Bitument content : 4.35%

Table 2. CRC and AC pavement [SETRA-LCPC 1998] [LCPC-SETRA 1997] materials compositions

Scenarios clearly differentiating types of cements, should be considered in the future as it may influence results of environmental flows.

Roads service life, including maintenance operations, has been implemented in the ERM during a 30 years period of time. Figure 7 (a and b) shows expected maintenance operations for that period. The chosen road construction reference year is 1990, because all available literature data are almost between 1990 and 2002.



(a) **CRC** pavement (b) **AC** pavement

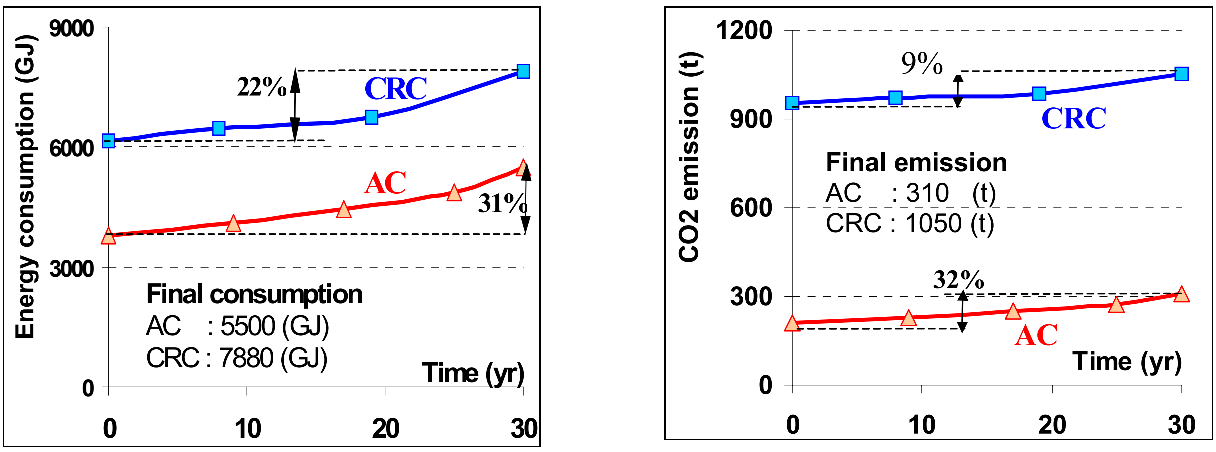
TACa: Thin asphalt concrete class a, HMAC: high modulus asphalt concrete

Figure 7. CRC and AC pavement maintenance description according to [Laurent 2004]

4.2 ERM pavement inventory results

The inventory calculations as well as all the emissions results cannot be detailed in this paper, they will be part of the PhD report of T. Hoang, that will be published in 2005. Hence, ERM inventory only includes: (i) raw materials (aggregates, bitumen, cement, steel) and energy consumptions, (ii) airborne emissions (CO, CO_2, CH_4, Volatil Organic Compounds, NO_x, N_2O, Particulate Matter (PM), SO_2), (iii) water emissions (Dissolved Organic Compounds, Oil(aq) Total nitrogen and Phenol), (iv) wastes (milled aggregates). This paper only presents energy consumptions, CO_2 emissions, and aggregates and bitumen consumptions as natural resources.

Figure 8a presents energy consumptions for both types of pavements. It shows that **CRC** pavement consumes more energy than **AC** pavement, while maintenance road works contribution to energy consumption respectively varies from 22% to 31% of total consumption. Figure 8b presents CO_2 emissions and exhibits a strong difference between both types of pavements. **AC** pavement requires one more maintenance operation than **CRC** pavement, during 30 years.

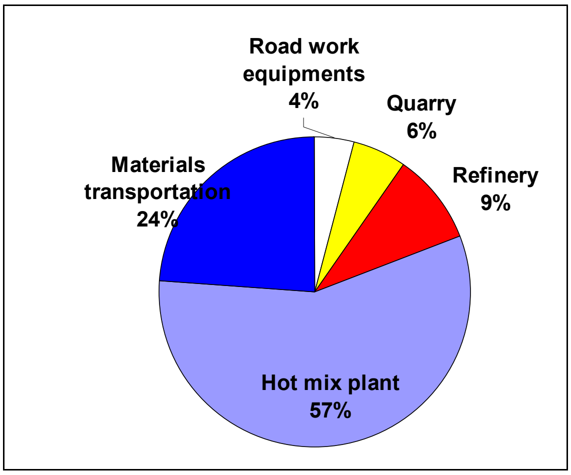


(a) Energy consumption **(b) CO_2 emission**

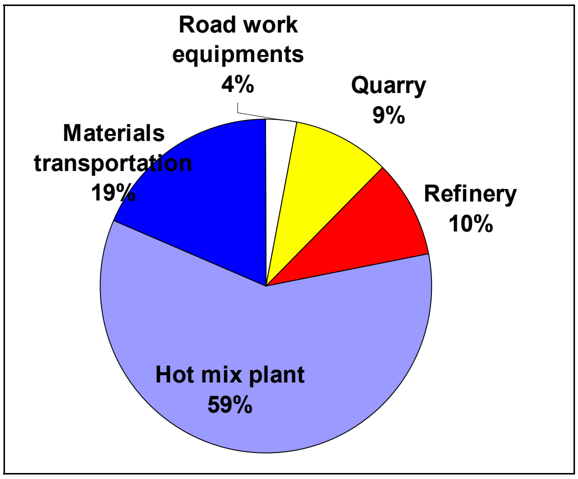
Figure 8. Energy and aggregates consumption for construction and maintenance works during 30 years

According to the studied scenarios, **CRC** pavement emits almost three times more CO_2 than **AC** pavement. For **CRC** pavement, contribution of maintenance work to total CO_2 emissions is only 9%, which is weak compared to the 32% contribution of **AC** pavement maintenance work. Besides,

bitumen was not considered as feedstock energy as in other studies. For instance, according to [Pontarollo & Smith 2001], the primary energy of one cubic metre of asphalt incorporating 20 % Reclaimed Asphalt Pavement (RAP) is 1.62 GJ, while the corresponding feedstock energy is 4.48 GJ. Therefore, to include this amount of energy in the global balance leads to a change by a factor 2.8. This point is highly discussed between experts. According to [Consoli et al. 1993], "feedstock energy is important to be included if the feedstock is a commodity used as a source of fuel", and bitumen is not known to be used as a source of fuel. This possibility may become a reality in the future, and may thus induce specific processes that should then be also included in the system. The "bitumen feedstock energy" question cannot be considered as a simple question and deserves further investigations.
Furthermore, comparing figure 8a and 8b, it can be noticed that the time variations of energy consumption and CO_2 emissions are similar for **AC** pavement, and different for **CRC** pavement. In the case of **AC** pavements, this result is classical because most of CO_2 emissions usually come from combustion processes, which are the main used energy sources in the system. The contribution of electricity power is only 4 % of the total energy consumed by the system. In the **CRC** case, there is no simple relationship between energy consumption and CO_2. In that case, electricity power is also weak: 12 % of the total energy consumption.
AC pavement energy consumption and CO_2 emissions have been detailed by subsystem on figure 9a and 9b, the subsystem repartition is the same. Most of energy consumption and CO_2 emissions are due to the hot mix plant and transportation. Contribution of the quarry to CO_2 emissions is weaker compared to its contribution to energy consumption, because the main source of energy used is electricity power (53%) for that subsystem.



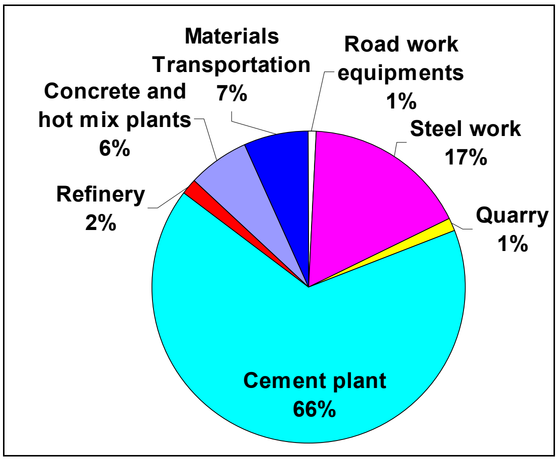
(a) CO_2 emissions



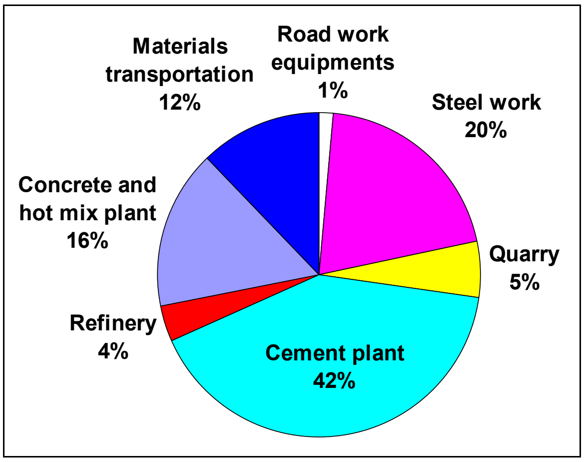
(b) energy consumption

Figure 9. Subsystems contributions for AC pavement after 30 years

As for CO_2 emissions (figure 10a), the cement plant contribution to the global system is much higher: 66% of total CO_2. Indeed, in addition to combustion processes, CO_2 is also produced during chemical process of clinker production [Holcim 2004].
The same results are shown on figure 10a and 10b for **CRC** pavement. Cement plant, concrete and hot mix plants, steel manufacturing and materials transportation consume 90% of energy (figure 10b). The cement plant consumes the most with 42%.



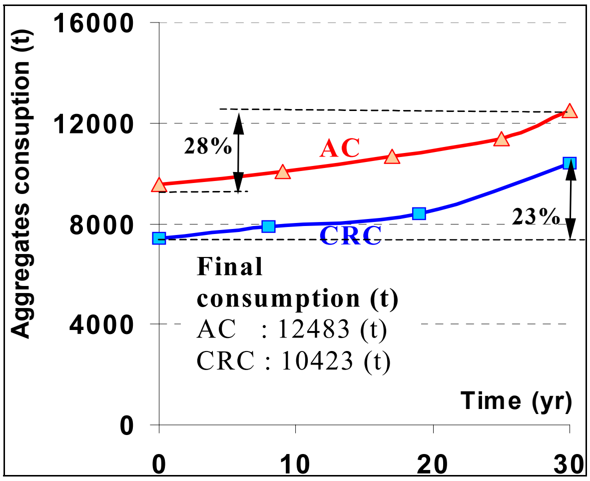
(a) CO_2 emissions



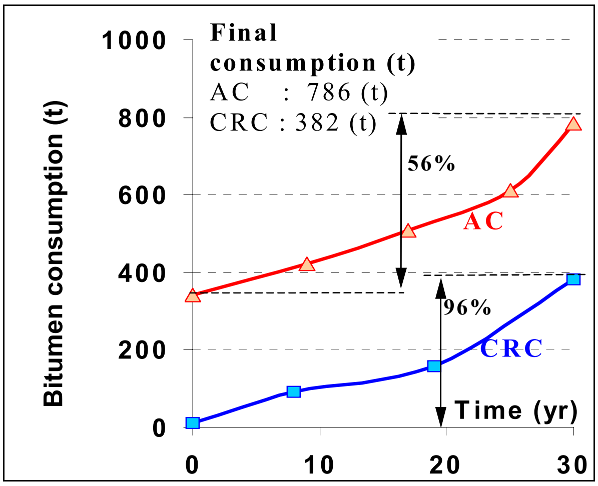
(b) energy consumption

Figure 10. Subsystems contributions for CRC pavement after 30 years

Figure 11a presents aggregates consumptions, which is greater for **AC** pavement than **CRC** pavement. This result is opposite to previous results [Ventura *et al*. 2003]. Some changes of the studied scenarios explain such a difference. In this previous paper, pavements structures and maintenance policies followed official French guidelines [SETRA-LCPC 1998]. Here, structures and maintenance have been changed to be more representative of real French conditions [Laurent 2004]. The wearing course of the **AC** pavement has been changed from 8 cm TLAC, to 6.5 cm TLAC + 2.5 cm STAC. Furthermore, the **CRC** pavement structure has also been modified: the cement concrete course is not covered by an asphalt concrete wearing course immediately after construction, but during the first maintenance operation after 8 years. These changes led to an increase of the total aggregates consumption for **AC** pavement and a decrease for **CRC** pavement. Contribution of maintenance works to the total aggregates consumptions is quite important: 23% for **CRC** pavement and 28 % for **AC** pavement. The **AC** pavement bitumen consumption (figure 11b), is logically greater than **CRC** pavement one. First, **AC** pavement requires bitumen during construction whereas **CRC** pavement does not. Secondly, **AC** pavement scenario needs one more maintenance operation than **CRC** pavement. The part of bitumen consumption for maintenance is very important even for **CRC** pavement (96%), because maintenance works always use asphalt to rebuilt surface layers.



(a) aggregates



(b) bitumen

Figure 11. Aggregates and Bitumen consumption for road construction and maintenance after 30 years.

By the way, this difference between both studies shows that structures, constructions, and maintenance policies changes can reverse results and provides arguments to the necessity of elaborating a modular evaluation tool, including uncertainty analysis.

5. CONCLUSIONS AND PERSPECTIVES

A new tool for environmental evaluation dedicated to roads, has been developed. Based on LCA methodology, it is entirely modular, and applicable to many different scenarios of road structures construction and maintenance policies. This tool calculates environmental inputs and outputs flows, in a first phase called ERM, called ERM inventory, which has been detailed in this paper. ERM has been applied to two main types of heavy traffic French using **CRC** and **AC** pavements highways and a 30 years analysed period. They are analysed and give the inventory of the following flows: consumed materials (aggregates, bitumen, cement, steel), consumed energy, released airborne pollutants (CO, CO_2, CH_4, Volatil Organic Compounds, NO_x, N_2O, Particulate Matter, SO_2), and released water pollutants (Dissolved Organic Compounds, Oil(aq) Total nitrogen and Phenol). Only energy and raw materials consumptions, as well as CO_2 emission results are presented in this paper.
Energy consumption and CO_2 emissions results highlight that in the case of **CRC** pavement, cement and steel plants are the dominating subsystems. Besides, for the **AC** pavement, the contribution of the hot mix plant is found to be the most significant. Comparing both types of pavements, **CRC** energy consumption and CO_2 emissions are greater than **AC** (bitumen feedstock energy is not accounted for). This is primarily due to the concrete plant contribution that nearly consumes half of the total system's energy and releases extra CO_2 due to the chemical reactions of the elaborating process. Natural aggregates consumption of **AC** pavement is greater than the one of **CRC** pavement. AC pavement also uses more bitumen because it is the main binder used for this kind of pavement.
The presented results only concern chosen examples, and cannot be generalised. They are greatly linked to materials, equipments, and maintenance scenarios, and, as it has already been noticed, scenario changes may invert results. This points out the necessity of conducting uncertainty calculations.
The next step of the methodology is still to be done. It will consist in including other subsystems (electricity production, coal processing…). Relationships between road and the crossed territory are a part of the model still to develop in the form of GRM tool. Coupled to environmental evaluations, external costs are also in progress for a full analysis.

6. ACKNOWLEDGMENTS

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7. REFERENCES

Blanc, A. 1999*, Analyse de cycle de vie des filières de traitement des sites industriels pollués.* PhD report of INSA de Lyon.

Boiteux, M. 2001, *Transports: choix des investissements et coût des nuisances,* Report of "Commissariat General du Plan - France", 149 p.

Chappat, M. & Bilal J. 2003, *La route écologique du futur. Consommation d'énergie & émission de gaz à effet de serre*. Report of COLAS, 40p.

Consoli F., Allen D., Boustead I., Fava J., Franklin W., Jensen A.A., de Oude N., Parrish R., Perriman R., Postlethwaite D., Quay B., Séguin J., Vigon B., 1993, *Guidelines for Life-Cycle Assessment : a code of practice 1st ed.*, SETAC ed., p.24.

Holcim, 2004, *Resource utilization and CO_2*, Report of Holcim cement production group, website http://www.holcim.com/CORP/EN/b/IEW/oid/47529/module/gnm50/jsp/templates/editorial/editorial.html, consulted on may 13th 2004.

ISO 14040 standard, 1997, *Environmental management. Life cycle assessment. Principles and framework.*

ISO 14041 standard, 1998, *Environmental management - Life cycle assessment - Goal and scope definition and inventory analysis.*

ISO 14042 standard, 2000, *Environmental management - Life cycle assessment - Life cycle impact assessment.*

ISO 14043 standard, 2000, *Environmental management - Life cycle assessment - Life cycle interpretation*

Lafarge Cement, 2004, *Our environmental commitment Performance & objectives* . Report, consulted 03/2004 at : http://www.cement.bluecircle.co.uk/env_per.htm#

Laurent, G., 2004, *Evaluation économique des chaussées en béton et classiques sur le réseau routier national français.* Research of LCPC N°CR33 , p. 51-56.

LCPC-SETRA, 1997, *Chaussée en béton,* French guideline, 134 p.

Lundström, K. 1998, *Influence des chaussées en béton et asphalte sur le milieu,* 8th International Symposium on Concrete Road, 13-16 Septembre 1998, Lisbon-Portugal. Theme V: Safety and Environment, pp. 195-202.

Michel, F. 1997, *Les granulats. In: UNICEM & Ministère Education Nationale (Editors)*, Paris, France, 39 p.

Mroueh, U. M., Eskola, P., Laine-Ylijoki, J., Wellman, K., Esa Mäkelä Markku Juvankoski, E. M. M. & Ruotoistenmäki A. 2000, *Life cycle assessment of road construction. Rapport de Tielaitos*, Helsinki, Finland. 59p. and appendix.

Peuportier, B., 2003, *Analyse de vie d'un kilomètre de route et comparaison de six variants*. Rapport de Centre d'Energétique de l'Ecole de Mines de Paris pour CIM béton. 48p.

Pontarollo, G. & Smith, T. 2001, *A life-Cycle Analysis Of the Environmental Impacts of Asphalt and Concrete Road,* Paris 2001 IRF World Road Congress, 17p.

Rouwette, R.R.J.H. & Schuurmans A., 2001, *LCA of concrete motorway pavement. An example of the use of JPG LCI data*. Final Report for IN01/18.

Saint-Laurent Cement, 2003, *Une question de respect. Rapport sur le développement durable,* report. 30p.

SETRA-LCPC, 1998, *Catalogue des structures types de chausses neuves',* French guideline, SETRA-LCPC.

Stripple, H. 2001, *Life Cycle Assessment of Road. A pilot Study for inventory Analysis*, Report of IVL - Swedish Environmental Research Institute, 96 p. and appendix.

Taiheiyo Cement Corporation, 2003, *Environmental Report*. Annual Report, 18p.

Ventura, A., Hoang, T., Jullien, A. & Moneron P. 2003, *Integrating lifetime aspects into initial evaluation of road pavement : how to proceed?* ILCDES, Kuopio, Finland, 1rst-3rd Decembe, 2003.

Ventura, A., Chabane, M., Monéron, P., Jullien, A., Guidoux, Y. & SHEMID., M. 2004, *Comparaison environnementale de couches de liaison de chaussée recyclées à différents taux par la méthode d'analyse de cycle de vie.* Bulletin des laboraoires des ponts et chaussées, 250-251, ref. 4527, p. 93-113.

WBCSD, 2002, *What LCA an tell us about the cement industry*. Research report, p.5.

Estimating the mass flow in building stocks

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TT6-64

ABSTRACT

Building stocks, as the greatest physical, economic and cultural capital of european nations, induce large amounts of energy and mass flows through their construction, operation, maintenance and demolition. While the energy flows for use and operation of single buildings are well known since the early 70s, the estimation of the mass flows in buildings and building stocks is a new field of research in life cycle analysis (LCA). For doing this, a great amount of statistical data is needed. Former studies have composed the stock through typical houses, without proving the representativity of a typical house. To improve the method, additional data concerning the behaviour of the whole stock, in particular material and demolition data are needed. The presented approach uses empirical data of effective building life spans and material composition from a random sample of a middle size town as well as data from a complete inventory count of demolitions to estimate energy and mass flows in the past. With the information of the year of construction and the year of demolition, class specific average life spans are calculated. Using tables of building class specific masses from other sources, the energy and mass flows for a particular moment can be estimated. In the future trend extrapolation of historical mass flows will be calculated.

KEYWORDS

building stock, building classes, dynamic, mass flow, life span.

1 INTRODUCTION

To reach the goals of sustainable development one option is to reduce the ressource demand and the emission of the different economic sectors. An analysis of the different production sectors and their material intensities identifies the most important factors.

A quick view on the data of the German production sectors shows the composition of the goods input (cf. [Figure 1]) and of the waste (output) flows production (cf. [Figure 2]). The considerable importance of the building sector appears immediately.

In 1990 the building sector has absorbed about 1109 million tons of material input, which is approximately 28% of the German national material input (without water).

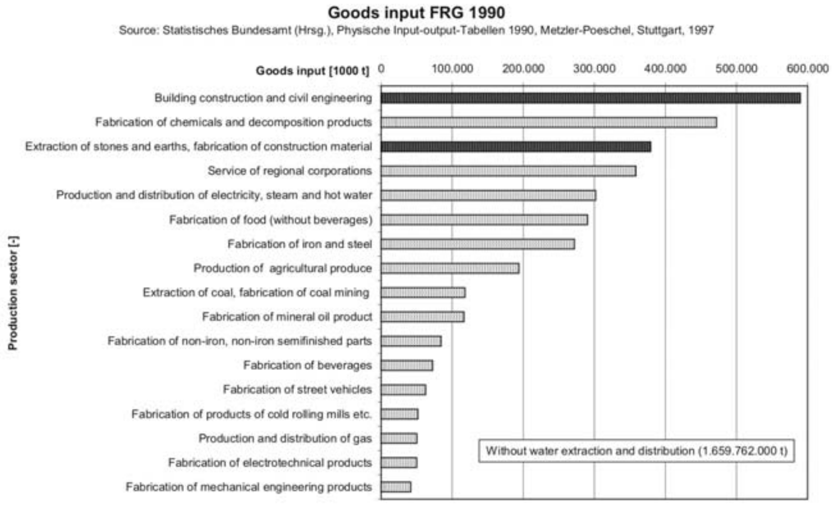

Figure 1. Material input of different production sectors in Germany in 1000 t (1990, former West German states) [Statistisches Bundesamt 1997].

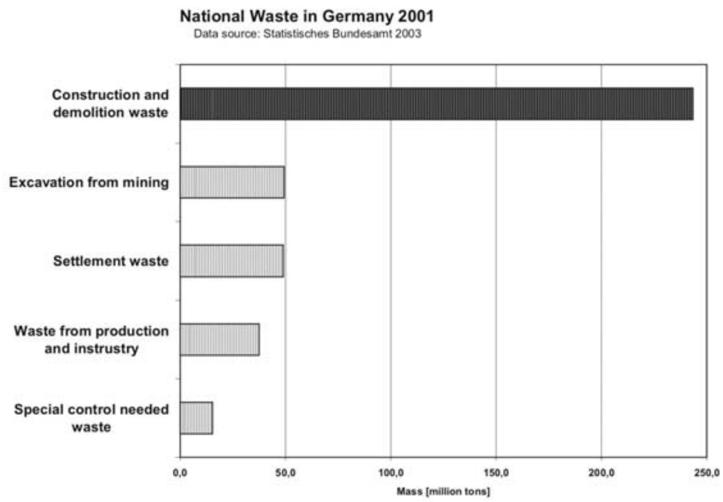

Figure 2 Distribution of the national waste in Germany 2001 [Statistisches Bundesamt 2003].

The mass flow into the existing building stock (980 million tons) is an important part in the mass flow of a nation (20 %). [Figure 2] shows that the material output of the building sector in Germany (243,4 million tons) is nearly two-thirds (61,7%) [Statistisches Bundesamt 2003] of the total national waste in the year 2001. The input/output relation is not all symmetrical: for nearly 1 billion tons of input only a fourth leaves the buildung sector. The building stock, as an intermediate deposit, is still growing strongly. The materials of the building stock of today will be the waste of the future. We are far from a recycling society.

To identify the potential of reducing the overall enviromental impact and ressource consumption of the building activities, a detailed analysis of the building stock and its dynamics is necessary. The question of the building stock has been occasionally of interest for different reasons [Kohler and Hassler, 2002]. First studies after the second world war have looked more closely on the material composition of houses, especially on residential buildings [Wedler & Hummel 1947], in order to plan the reconstruction and the reuse of the materials. The aim of a study by Fleckenstein [Fleckenstein *et al.* 1997] was the prognosis of the future demand on mineral construction material. Görg [Görg 1997] made a prognosis on the probable evolution of waste by analysing the composition of typical houses. Kloft [Kloft 1998] studied the energy and mass flow in the residential building stock. The study of the Enquete Commission [Kohler *et al.* 1999] made a first attempt to describe the whole German building stock and its present and future mass flow, using both top down and bottom approaches. Bringezu [Bringezu, 2000] analysed the input and output flows on a national level, situating the builing sector. The building stock of New Zealand is the subject of several studies performed by Johnstone [Johnstone 2001].
To describe any building stock as a basis for the estimation on the present and future massflow three types of basic data are needed:

- material composition of the stock
- size of the stock (floor space, volume)
- dynamic of the stock (growth rate, demolition rate etc.)

2 NEW APPROACH

Taking into account the shortcommings of the top down approaches (input/output statistics) and the bottom up approach through typical buildings, a new approach consisted in studying in detail the historical evolution of a representative urban building stock. This should allow to produce new knowledge both on the dynamics of a particular stock and of possible simplifications in the collection of data on building stocks in general. In the project "Validating an integrated, dynamic model of the German building stock" [Ferrara 2004]) funded by the Deutsche Forschungsgemeinschaft (DFG) 1999-2003 the building stock of the town of Ettlingen near Karlsruhe in the province Baden-Württemberg in the South of Germany was analysed in depth. It was chosen because of the size of the town (approx. 39,000 inhabitants) and the easy access to the town administration and archives. Approximately 48% of Germany´s residents live in towns of this size (20,000 – 49,999 inhabitants) and nearly ¼ of all German towns belong to this size class [Behnisch 2004].

From all sources found, the most important ones for collecting detailed building stock data for the study are:

- The archive of the former public building insurance company ”Sparkassenversicherung Badische Gebäudeversicherung” where all buildings were assured up to 1994.
- Digitalised cadastral plan of Ettlingen (2002, registered buildings in the year 2000)

As mentioned above, the most important data in this context were the dynamic data. In general these known data are growth rate, demolition rate and maybe refurbishment rate. As we were looking at particular buildings the data did not concern rates but states (in a particular year) of “in construction”, “in operation” and “in demolition”.
The operation phase contains subphases like modernisation, renovation, modification, conversion, etc. This phase is not documented in great detail because there is no interest of the town administration or the insurance companies in these processes.

In literature the life span of buildings is generally assumed between 80 and 100 years. This assumption is based on economic criteria and has no direct relation to real building life spans. One objective of the research project was the determination of the real building life span of the different use and age classes (cf. [Bradley 2004]).

The data acquisition was realised with a specific data base (cf. [Figure 3]). The data set to collect the building data is divided into eight subsets (general information, dimension, construction, floors, layout plan, building photo, remaks, GIS visualization) with specific input fields. To maximize the input speed and to minimize the error rate, most fields contain classified answers with pull down menus. The most effort was dedicated to the history type of entry (which change in which year). The data base was organised along the same life cycle oriented data scheme as the integrated LCA software used for LCA and LCC calculation [Kohler and Lützkendorf, 2002]

3 DATA

In the town of Ettlingen a complete inventory of all individual demolitions was established for the period of 1936 - 1994 through the available sources. The construction years for the demolished building were known. A rough estimation of new constructed buildings in each year was generated in [Bradley 2004] from the sample data taken in the first part of the project [Bader *et al.* 2001]. The collection of data for the second phase (building operation) in the building life for all buildings was intended, but has not yet been accomplished for all demolished buildings. The material data was provided by the sources, but could not be collected entirely during the project time. Therefor a hybrid approach is used to calculate the mass flow in building stock.

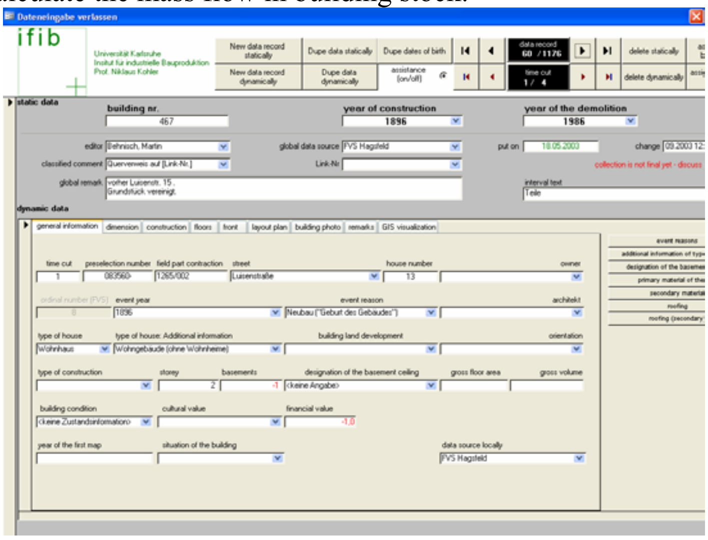

Figure 3. Data Base to acquire building stock data [ifib 2003]

	AC-1	AC-2	AC-3	AC-4	AC-5	AC-6	AC-7	AC-8	AC-9
Period	Before 1835	1836 – 1870	1871 – 1918	1919 – 1933	1934 – 1949	1950 – 1964	1965 – 1976	1977 – 1994	1995 – today

Table 1. Age classes (defined by constructive, social, political and economical properties and consistant with general German statistical classes).

For a simulation of the mass flow of a building stock, a detailled look on its structure and properties is necessary. The age classes and the use classes generate a matrix of 9 by 9 use classes. For selected age and use classes some data from the literature is used to demonstrate the methodology.

UC-1	**Residential buildings**	one family houses, terrace houses, semi-detached buildings, multiple family dwellings, tower blocks

	Non-residential buildings	
UC-2	Institution buildings	(hospitals, monasteries, barracks, prisons, etc.)
UC-3	Office and administrative buildings	
UC-4	Agricultural plants	
UC-5	Non agricultural plants	
UC-6	Factory and workshop buildings	
UC-7	Trade and storage buildings	
UC-8	Hotels and public houses	
UC-9	Other non-residential buildings	(universities, museums, theaters, gymnasia, etc.)

Table 2. Second level of use classes defined by Statistisches Bundesamt [Statistisches Bundesamt 1978].

The existence of age and use classes reduces the needed data. For each class a specific material composition can be elaborated and used for further calculation. The use of an open data base allows to refine the data with each building entered.

4 ANALYSIS OF THE STOCK DATA

The first step after collecting data in Ettlingen was the analysis of the data to find out some basic properties of the structure and the dynamic of the building stock in the sample town. [Figure 4] shows the development of new construction and demolition in Ettlingen during 1936 and 2000. From the first view one can see a much higher number of new constructions than demolitions. As a result of this one can observe a net stock increase (cf. [Figure 5]). The second result of this analysis is a higher dynamic in the non-residential building class on both processes. In the observed time span there are more non-residential buildings newly constructed than residential buildings and even more non-residential buildings demolished then residential buildings. This means that the non-residential building class has a higher dynamic than the residential building class. The average demolition rate for the time from 1936 to 2000 of residential buildings is approximately 0,1 % and for non-residential buildings eight times higher (approximately 0,8%). There were very few war destructions in Etlingen.

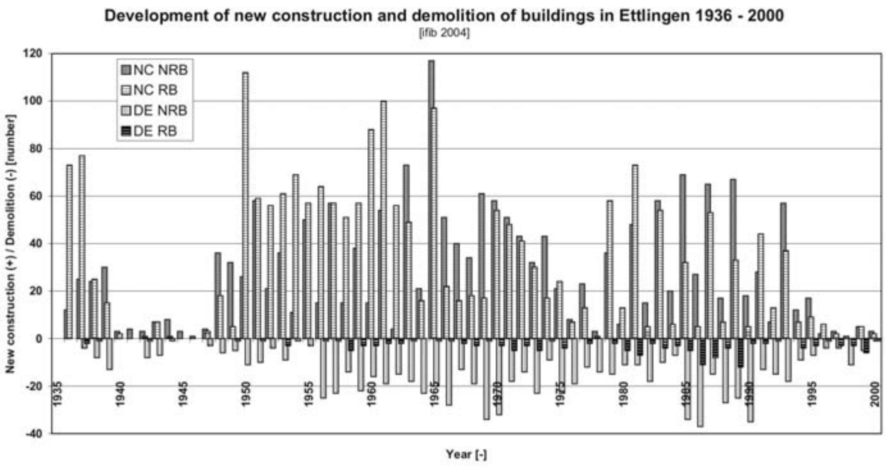

Figure 4. Development of new construction (new construction, positive values) and demolition (negative values) for residential and non-residential buildings in Ettlingen from 1936 to 2000.

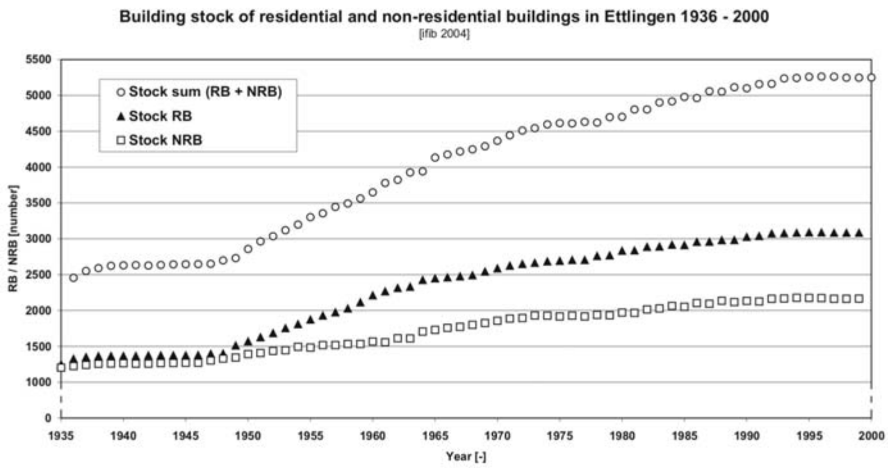

Figure 5. Building stock of residential and non-residential buildings in Ettlingen 1936 - 2000.

For these reasons the non-residential buildings cause a much higher mass flow than the residential buildings and should be investigated in more detail. But for several reasons the residential building part of the building stock is much better known and documented than the non-residential building part. It is also more homogenous and has much less problematic materials.

5 CALCULATION OF MASS FLOW

With the collected data it was possile to create different views and to analyse the behaviour of (sample) building stock in depth. One option is to look at the mass flow of one selected year or of a longer period of single buildings or single building classes. Another view could be the mass flow of one building class during its life span, or, with a complete data set, of specific construction material (not yet terminated). The difference in waste composition in the future could be another topic.

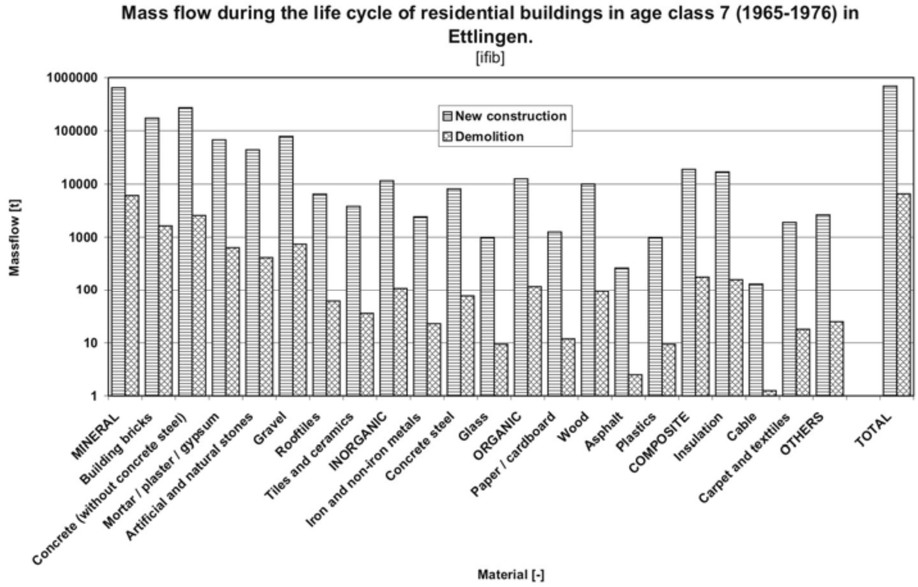

Figure 6 Mass flow of residential buildings in the age class 7 in Ettlingen at logarithmic scale.

To demonstrate the methodology a sample from the residential building age class number 7 (1965-1976) is taken. The size and dynamic data are completed by composition data from the literature [Schulze & Walther 1990]. [Figure 6] shows the result of the calculation. As expected the mass flow of new construction is orders of magnitude higher than the mass flow of demolition. This results from some reasons. The first is the higher rate of new construction. The second is the higher average gross volume of the new buildings. And the third the different composition of the building material.

The total mass flow is calculated as the sum of new construction plus operation of the stock and the demolition of buildings. As shown above for the age class 7 it is possible to calculate the mass flow for new construction and demolition. The mass flow of the operation phase of a building is much more complicated to calculate. The energy flow for heating could be calculated with some average values e.g. from Schulze Darup [Schulze Darup 2002] and the given building area. For older buildings this might lead to an overestimation, because not all rooms and therefore not the whole building area were heated in former times. But it can give a first idea of the order of magnitude. The mass flow of the operation phase of a building is the result of all processes between new construction and demolition. These are renovation, reconstruction, reuse, modernisation, modifications. These processes depends on different building use and construction properties. Their description and calculation is therefore much more complicated and could not be detailled here in total.

There is a model for the time dependence of some building and building element aging process. The data are not very precise and well defined for the moment. [Buergel-Goodwin 2004] shows the wide variation of the life span data of building components and their relation to the reality by some selected samples. As a result of this work, there is a constant overestimation of the mass flow by using the standard data from the literature.

6 RESULTS AND FUTURE PROSPECTS

Starting from the motivation to estimate – predominantly - the mass flows in building stocks, the relevant procedure is described. The analysis of the production sectors and the waste occurrence in Germany pointed out that the building sector and the building stock play an important role in the societal mass flows and they influence considerably all attempts versus a more sustainable environment.

During this project a data base for building and building stock data was developed and implemented. Data for all phases of a building life (new construction, operation, demolition) with different data depth were acquired and analysed. The results are stratified by age and use class.

Some selected results from the analysis show a higher dynamic in the non-residential building class and therefore a higher energy and mass flow. The non-residential building class is more diversified and has been less well known as the residential building class. This makes data acquisition more difficult.

To present the methodology a sample calculation for an age and use class is carried out. By solving the obstacles of missing data the presented methodology seems to be a practicable approach to estimate mass flows in whole building stocks today and to extrapolate future trends which can be compared with bottom up long term estimations of the evolution of the building stock [Kohler *et al.* 1999].

7 ACKNOWLEDGMENTS

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8 REFERENCES

TT6-64, Estimating the mass flow in building stocks. C. Ferrara.

Bader, G., Schwaiger, B. & Ferrara, C. 2001, *Methode zur Analyse von Gebäudebeständen, Alterungsmodelle und Prognose von Stoffströmen mit Anwendung auf den Gebäudebestand der Stadt Ettlingen,* Research Working Paper in the version of Aug. 15, 2001, Institut für Industrielle Bauproduktion, Universität Karlsruhe.
Behnisch, M. 2004, *Klassifizierung von Gemeinden nach bestandsorientierten Merkmalen*, Diplomarbeit, Universität Karlsruhe.
Bradley, P.E. 2004, *Validierung eines integrierten, dynamischen Modells des deutschen Gebäudebestandes: Statistischer Teil,* Technical Report 2004-01, Institut für Industrielle Bauproduktion, Universität Karlsruhe.
Bringezu, S., 2000, *Ressourcennutzung in Wirtschaftsräumen: Stoffstromanalysen für eine nachhaltige Raumentwicklung* ; Springer-Verlag, Berlin.
Buergel-Goodwin, E. 2004, *Vergleichende Studie zu Erneuerung, Unterhalt und Betrieb von Bestandsgebäuden auf Bauteilebene*, Diplomarbeit, Universität Karlsruhe.
Ferrara, C. 2004, *Validierung eines integrierten, dynamischen Modells des deutschen Gebäudebestandes: Allgemeiner Teil,* Technical Report 2004-04, Institut für Industrielle Bauproduktion, Universität Karlsruhe.
Fleckenstein, K., Hochstrate, K., Knoll, A. 1997, *Prognose der langfristigen Nachfrage nach mineralischen Baurohstoffen*, Endbericht, Mannheim.
Görg, H. 1997, *Entwicklung eines Prognosemodells für Bauabfälle als Baustein von Stoffstrombetrachtungen zur Kreislaufwirtschaft im Bauwesen*, Schriftenreihe WAR, Darmstadt.
Johnstone, I. M. 2001, *Energy and mass flows of housing: A model and example*, Building and Enviroment 36, p.27 – 41, Pergamon Press.
Kloft, H. 1998, *Untersuchungen zu den Material- und Energieströmen im Wohnungsbau*, Eigenverlag, Technische Universität Darmstadt.
Kohler, N., Hassler, U., Paschen, H. 1999, *Stoffströme und Kosten in den Bereichen Bauen und Wohnen*, ed.: Enquete-Kommission "Schutz des Menschen und der Umwelt" des 13. Deutschen Bundestages, Springer, Berlin.
Kohler, N., Hassler, U. 2002, *The building stock as a research object.* Building Research and Information (2002) 30(4), 226-236
Kohler, N., Lützkendorf, TH. (2002), *Integrated Life Cycle Analysis.* Building Research & Information (2002) 30(5), 338–348
Schulze H.-J., Walther, P. 1990, *Gebäudeatlas - Mehrfamilienwohngebäude der Baujahre 1880 bis 1980* / Bauakademie der Deutschen Demokratischen Republik, Berlin
Schulze Darup, B. 2002, *Passivhaus-Projektbericht: Energie & Raumluftqualität*, AnBUS e.V, Fürth.
Statistisches Bundesamt (Hrsg.), 1978, *Systematik der Bauwerke*, Kohlhammer Verlag, Stuttgart.
Statistisches Bundesamt (Hrsg.), 1997, *Physische Input-output-Tabellen 1990*, Metzler-Poeschel, Stuttgart.
Statistisches Bundesamt (Hrsg.), 2003, *„Umwelt — Produktivität, Bodennutzung, Wasser, Abfall“*, Wiesbaden, , p.63 ff..
Wedler, B., Hummel, A. 1947, *Trümmerverwertung – Technische Möglichkeiten und wirtschaftliche Bedeutung*, Verlag Ernst & Sohn, Berlin.

Survey on Actual Service Lives for North American Buildings

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TT6-70

ABSTRACT

This paper presents results of a demolition survey in a major North American city that captured building age, building type, structural material and reason for demolition for 227 buildings. The findings challenge many common assumptions about building longevity, and, in particular, the relationship between structural material and service life. Although it is often believed that "durable" structural materials such as steel and concrete will provide the longest service lives for their buildings, our results suggest there is no significant relationship between the structural system and the actual useful life of the building. Reasons for demolition were instead related to changing land values, lack of suitability of the building for current needs, and lack of maintenance of various non-structural components. Only eight buildings identified a specific structural failure. Indeed, the service lives of most buildings are probably far shorter than their theoretical maximum lives; the majority of demolished steel and concrete buildings in our study were less than 50 years old. In the context of sustainable construction, this raises some interesting questions and shifts the spotlight away from durability of materials and on to 1) flexibility of design to allow future changes; 2) deconstructability; and 3) the use of more accurate life span predictions in life cycle assessment calculations on whole buildings.

KEYWORDS

Demolition, sustainable design, adaptability, durability.

1 INTRODUCTION

Very little published, statistical data exists regarding actual service lives of buildings. This data is important to know as it may influence decisions in several areas, including financial aspects of buildings such as insurance and tax depreciation, maintenance and renovation activities, and initial selection of building materials.

It is generally understood that, within a fairly short time frame, a building will become functionally obsolete, or neighborhood characteristics and land values change such that the building is no longer delivering the highest value for the land. Nonetheless, many practitioners in the building industry believe that buildings last a long time, and that longevity is related to structural material [Gaston *et al.* 2001; O'Connor *et al.* 2004], as shown in Fig. 1. Accordingly, there is an increasing tendency to make assumptions or claims about the relative durability of different structural materials, particularly for the purpose of highlighting presumed environmental characteristics of materials.

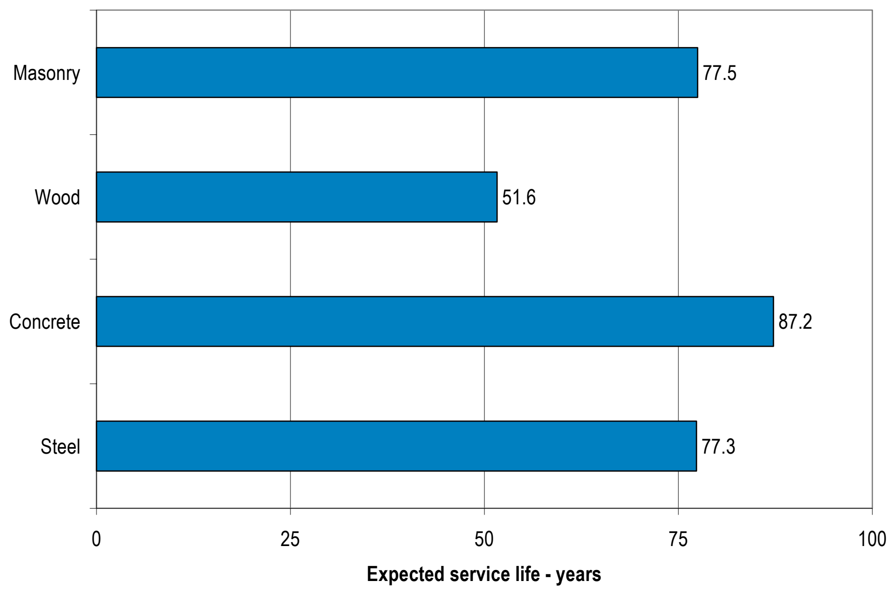

Figure 1. Average presumed service life for non-residential buildings by primary structural material. From surveys of architects, structural engineers, builders and developers in the United States and Canada. N=683.

The hypothesis behind this survey was that no relationship exists between structural material and service life of a building, and that buildings are most likely demolished far before the end of the useful life of the structural systems. Building industry beliefs that some structural materials last longer than others are most likely confusing how long a building *could* last with how long it is *actually* kept in service. In fact, a few previous studies indicate that service lives of most buildings are probably far shorter than their theoretical maximum lives. For example, a large study of U.K. residential buildings found 46% of demolished structures fell in the 11-32 year age class [DTZ Pieda Consulting 2000]. Another large study, of office buildings in Japan, found the typical life span to be between 23 and 41 years [Yashiro *et al.* 1990].

While recorded data on age of buildings at demolition is scarce, there are some data on the average age of buildings still standing. Although not as useful as demolition information, which tells us the age at the *end* of the building's life, average ages for existing buildings could be a rough indicator of the average service life midpoint. Sources for this information are organizations with large real estate holdings, such as government agencies or school districts. For example, the U.S. Department of Energy had 10,707 buildings in 2002, with an average age of 31 years [U.S. General Accounting Office 2003]. Public schools in the U.S. tend to undergo substantial renovations or additions to extend their service lives, thus the average age of the approximately 78,000 public schools in 1998 was 42 years; however, most schools are abandoned by the age of 60 [U.S. Dept. of Education 1999]. Other sources of data are agencies responsible for national statistics such as the U.S. Census Bureau. For example, in 2001 the United States had 119,117,000 residential buildings, with an average age of 32 years [U.S. Census Bureau 2002]. Statistics Canada reports that the average age of all non-residential buildings in Canada in 2003 was 17.9 years [Statistics Canada 2004].

In this era of increasing interest in sustainable construction, building practitioners are beginning to evaluate environmental impacts of design decisions over the full life cycle of a building. This process, known as life cycle assessment (LCA), requires an estimate of a building's useful life span. More statistical data on actual service lives will assist in keeping LCA results meaningful.

2 METHOD

A survey of buildings demolished between 2000 and 2003 in a major North American city was performed [Horst and Argeles 2004]. The city of Minneapolis/St. Paul was selected for two reasons: it was likely to have a mix of structural materials in its non-residential buildings, and it is located within a state currently undergoing a large building database project with coordination potential in the future.

We obtained city demolition permit records for the period in question, giving us contact information for the building owners. A short written survey was mailed, asking for a few details on each building. This included the age class of the building at demolition (0-25 years, 26-50 years, etc.), the primary structural material of the building (concrete, steel or wood), and reasons for demolition (area redevelopment, too expensive to maintain, etc.). When the building condition was cited as a reason, the survey probed for details.

We collected survey information for a total of 227 buildings, representing 75% of the appropriate demolition records for the period in question. Of these, 105 of the buildings were commercial, industrial or institutional and 122 were residential.

3 RESULTS

About two-thirds of the buildings were wood, a quarter were concrete, and the remainder were steel or various combinations of wood, steel and concrete (Fig. 2). The largest concentration of buildings fell into the 76-100 year age group (Fig. 3). At this point, it is necessary to separate the analysis into residential and non-residential buildings.

Over half of the sample is houses, which explains the high prevalence of wood; wood is widely used in the United States, particularly in houses. The longevity of houses is likely driven by different factors than those which affect the service lives of non-residential buildings. For example, older homes have characteristics valued by many people, and older homes are often more affordable. A substantial number of American homes were built before 1920 – 8.3% in 2001, in other words, about 10 million units [U.S. Census Bureau 2004]. To avoid some of the complex social factors that are unique to housing, we removed the residential buildings from some of the analysis. Wood is still well-represented among the non-residential buildings (Fig. 4).

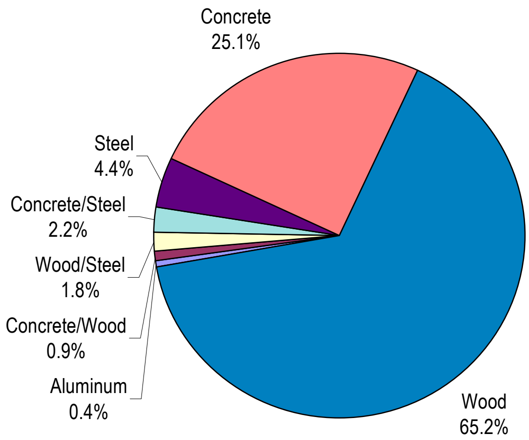

Figure 2. Proportion of all 227 demolished buildings by primary structural material.

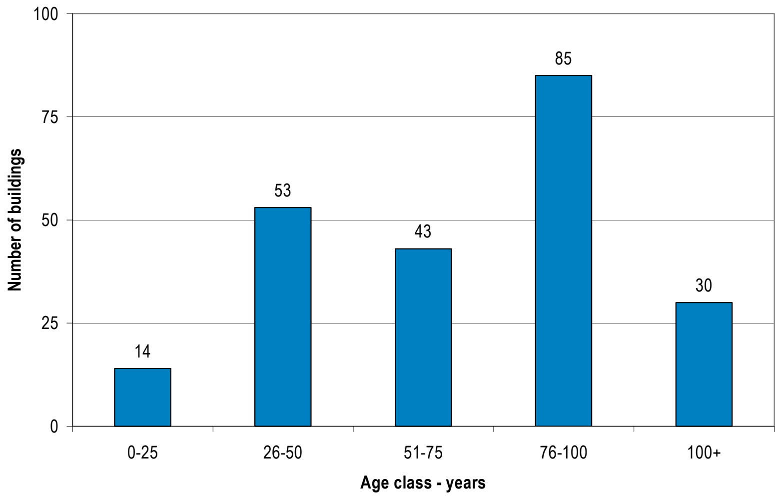

Figure 3. Distribution of all 227 buildings by age class.

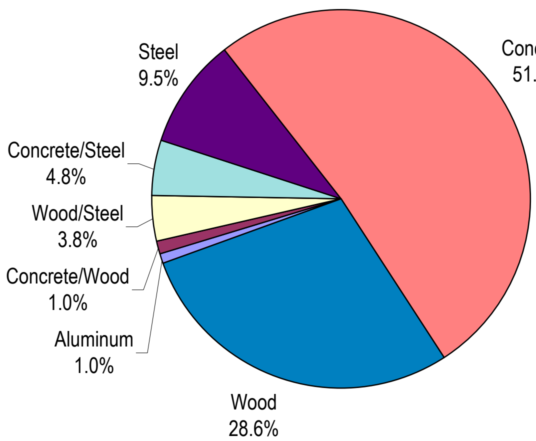

Figure 4. Proportion of the 105 non-residential buildings by primary structural material.

Looking at the age of the non-residential buildings only (Fig. 5), there is a clear concentration in the 26-50 year group. In Fig. 6, this information is shown segmented by material. Removing the buildings with material combinations, we show the distribution of the 54 non-residential concrete buildings, the 10 non-residential steel buildings and the 30 non-residential wood buildings over the age classes. The results present an interesting contrast with the impressions held by design professionals shown in Fig. 1. About 56% of the concrete buildings were 26-50 years old at demolition, while 63% of the wood buildings were older than 50 years, and the largest group of those fell in the 76-100 year age class. Meanwhile, 80% of the steel buildings were 50 years or younger, with half of those no more than 25 years old.

Comparing age with structural material helps illustrate possible correlations, but we also directly investigated whether or not the structural system was the primary motivation for the demolition. Looking once again at the entire data set of 227 buildings, Fig. 7 shows the distribution of respondents' reasons for demolition, which they selected from a list. About 38% (70 respondents) selected "building's physical condition" as the main reason. Those respondents were then asked to elaborate. The vast majority (54 of them) selected "lack of maintenance," presumably with respect to various non-structural elements. Only 8 selected "specific problem with structural or other material or system" (3.5% of all buildings). Of those, seven mentioned foundation problems and one mentioned wood decay. Six of these buildings were older than 75 years, one was in the 51-75 year group, and one was of unknown age.

Figure 8 segments the distribution over the top four demolition reasons by structural material. Surprisingly, given that they are the youngest in the data set, most of the steel buildings were demolished because they were no longer suitable for their intended use or due to their physical condition. For the wood buildings, physical condition was the dominant reason. This almost certainly is due to an age effect, since "physical condition" as the main reason for demolition predictably correlates with age of the building (75% of buildings citing "physical condition" were at least 76 years old), and the wood buildings are the oldest in the data set.

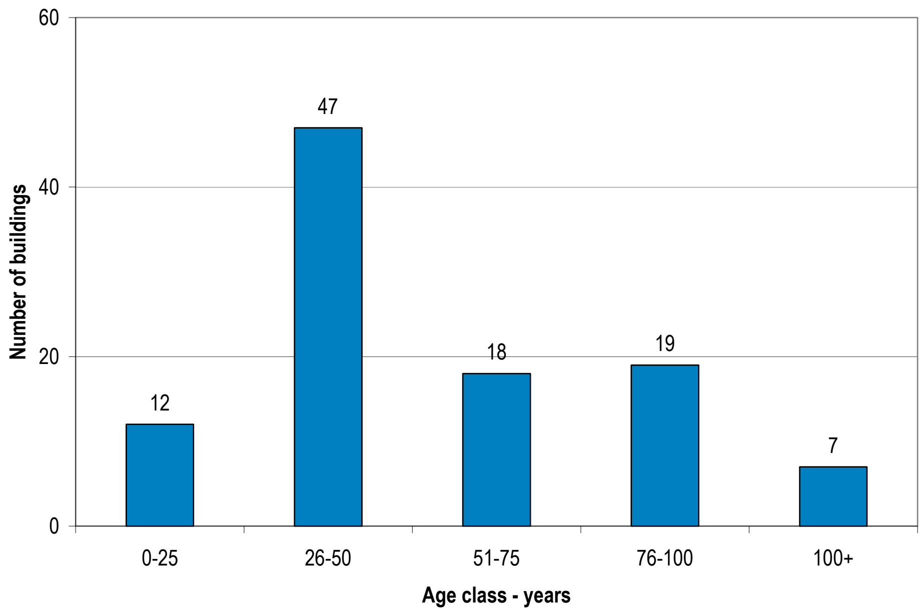

Figure 5. Distribution of the 105 non-residential buildings by age class.

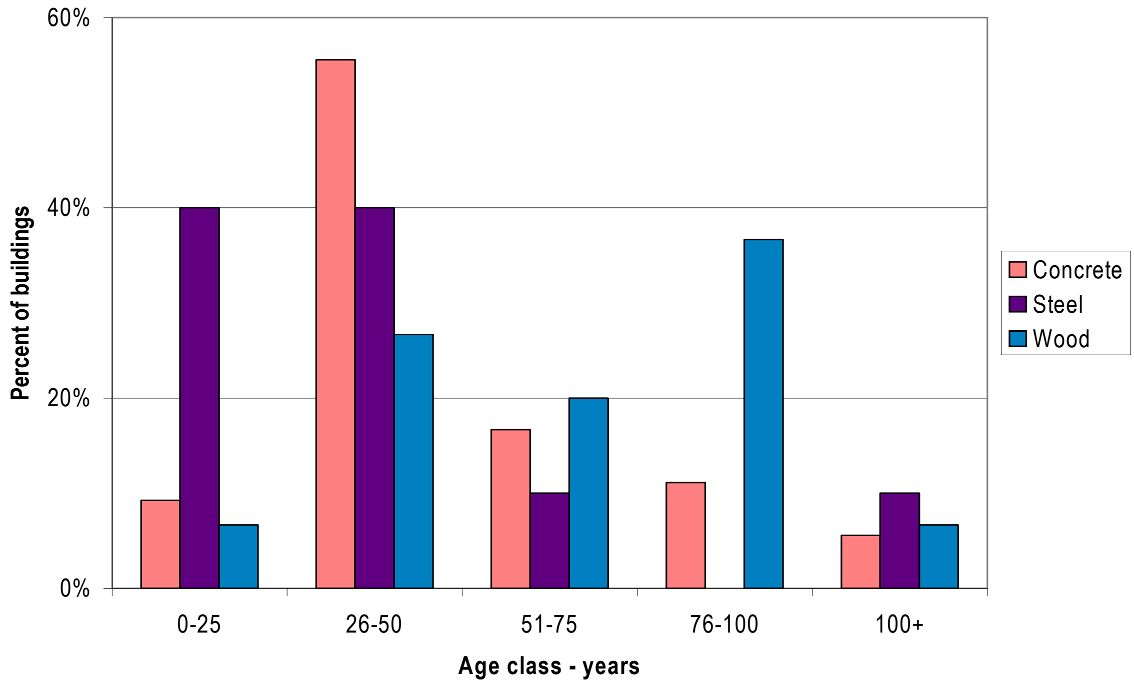

Figure 6. Distribution of 94 non-residential buildings by age class and by structural material (buildings with combinations of materials excluded).

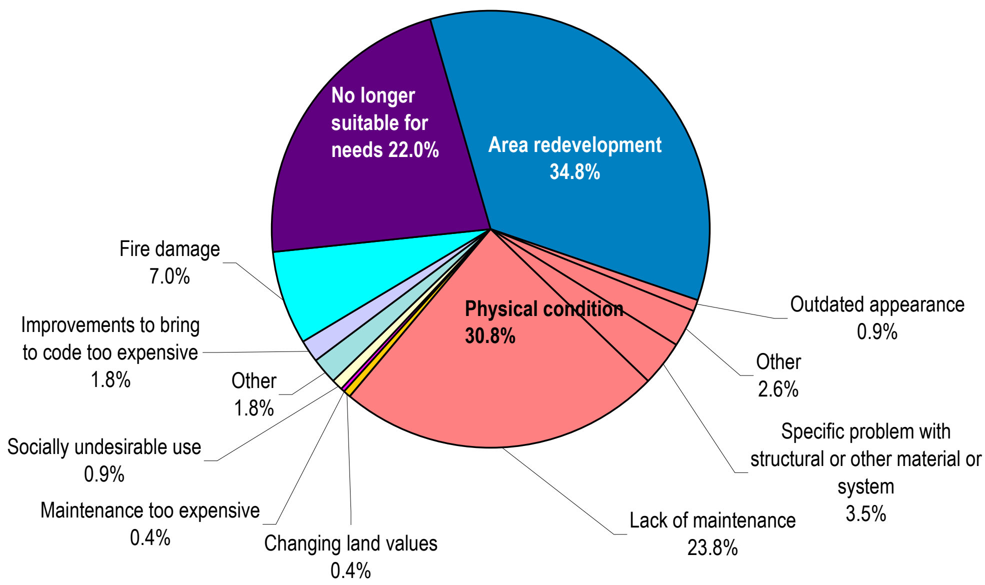

Figure 7. Distribution of "reason for demolition" responses for all 227 buildings.

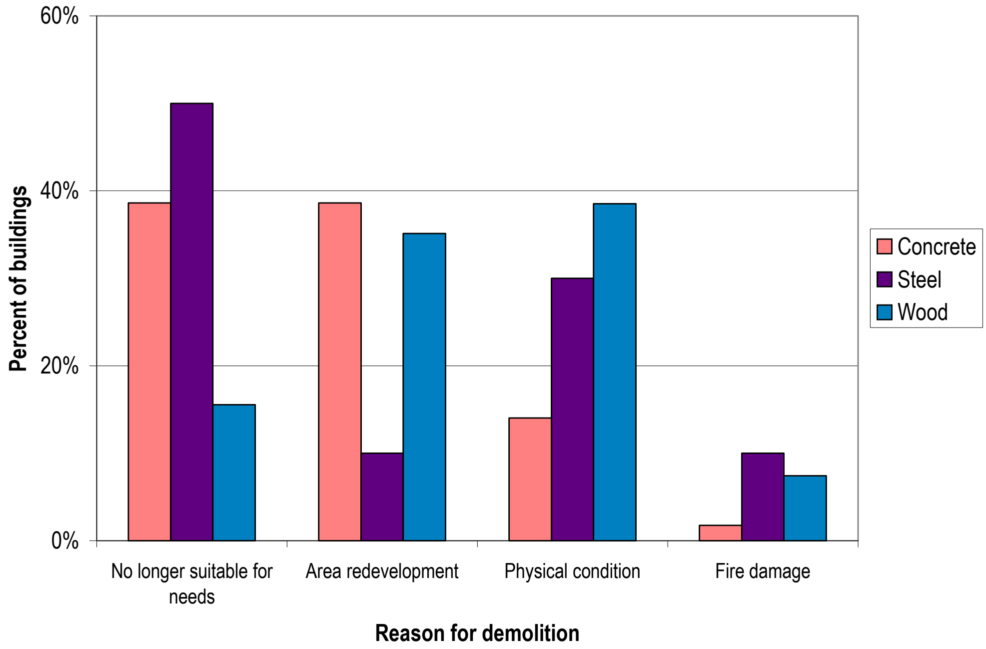

Figure 8. Distribution of buildings by material, over the top four reasons for demolition.

4 DISCUSSION

The purpose of this study was to determine if a relationship exists between structural material and building service life. It is tempting to view Fig. 6 as an indication that the two are correlated, but with an opposite relationship than many presume: the wood buildings appear to last the longest time, not the shortest. In our opinion, this conclusion is too simplistic without further investigation as to potential mitigating factors. Instead, our conclusion is that no meaningful relationship exists between structural material and average service life, and that most buildings are demolished for reasons that have nothing to do with the physical state of the structural systems.

The vast majority of buildings in this sample fell into just three categories of reasons for demolition: area redevelopment (34%), lack of maintenance (24%), and building no longer suitable for intended use (22%). The most common reason, redevelopment, is completely unconnected to the physical components of the building; this is a change to the use of the land, for example, converting an industrial site to housing.

For buildings identified as no longer suitable, most were likely too small and were to be replaced with a larger version of the same type of building, as is often the case with houses. Some of the buildings in this category may have been considered unsuitable due to technical obsolescence of some components or systems, and an upgrade was deemed too costly. However, in none of these buildings was a specific failure of a component identified.

The buildings which were demolished due to lack of maintenance (24%) are of most interest in the field of service life prediction for building components. While the structural system was intact in all of these, failures in the other components led to a shortened service life for the entire building. Our study did not probe for details about those failures; this is an area for future study.

In sustainable design, "durability" is increasingly being included on priority lists under the assumption that designing for longevity is an environmental imperative. However, this is unsupported in the absence of life cycle assessment and accurate lifespan predictions. In the worst case, designing for longevity can lead to design choices that are well-intentioned but, in fact, yield poor environmental results. For example, a building component with low embodied environmental effects, such as wood cladding, can be replaced many times before totaling the high embodied effects of a material such as brick. If the brick cladding ends up in landfill after 40 years of use, it was a poor choice on an environmental basis. The best environmental scenario for that brick is recovery at year 40, for re-use in another project. Rather than attempt to predict the future and design permanent structures with an infinite lifespan, we are probably better off in acknowledging our inability to make such predictions and instead design for easy adaptation and material recovery.

4 ACKNOWLEDGMENTS

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5 REFERENCES

Gaston, C., Kozak, R., O'Connor, J., and Fell, D. 2001, "Potential for Increased Wood-use in North American Non-residential Markets," Forintek Canada Corp.

O'Connor, J., Fell, D., and Kozak, R. 2004, "Potential for Increased Wood-use in North American Non-residential Markets – Part II," Forintek Canada Corp.

DTZ Pieda Consulting 2000, "Demolition and New Building on Local Authority Estates," Department of the Environment, Transport and the Regions.

Yashiro, T., Kato, H., Yoshida, T., Komatsu, Y. 1990, "Survey on real life span of office buildings in Japan," Proc. CIB 90 W55/65 Symposium, 1990, pp. 215-226.

U.S. General Accounting Office 2003, "Major Management Challenges and Program Risks, Department of Energy," Rpt. # GAO-03-100, January 2003.

U.S. Department of Education, National Center for Education Statistics 1999. *How old are America's public schools*. NCES 1999-048. Washington, DC: Office of Educational Research and Improvement.

U.S. Census Bureau 2002, Current Housing Reports, Series H150/01, "American Housing Survey for the United States: 2001," U.S. Government Printing Office, Washington, D.C., October 2002.

Statistics Canada personal communication, August 2004.

Horst, S. and Argeles, C., "Minnesota Demolition Survey: Phase Two Report", 2004, Athena Sustainable Materials Institute.

U.S. Census Bureau, Current Housing Reports, Series H121/04-1, "These Old Houses: 2001," U.S. Government Printing Office, Washington, D.C., February 2004

Speeding up Sustainable Development using the cost advantage of low cost products

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ABSTRACT

The three pillar model of Sustainable Development (SD) looks for a joint positive ecological, economical and social development. Whereas in most SD activities this three pillar model is accepted as a starting point, in practice only ecological measures to support SD are mentioned and economic criteria are neglected. But low economic Life Cycle Cost (LCC) is the most important economical criterion and can be used to strongly support SD.

To compare product systems we expand them in such a way that they have the same Life Cycle Cost (LCC). The expansion is chosen in a way that ecological advantage is realised and then these expanded systems are assessed via Life Cycle Impact (LCI) studies. This is called the Vinnolit Optimisation Method (VOM). A quantitative example for drinking and waste water pipes achieves huge ecological gains by integrating the important economical criterion "low cost".

This strategy is important for all working in the building area like architects, public and private purchasers etc., if they want to contribute to SD in a fast and effective way.

Other points discussed:

- So called "positive" and "negative rebound effects", internalisation of external cost etc..
- The pipe example shows the very high importance of economical advantage compared to ecological advantage.
- Using low cost products, even if made from Non Renewable Resources (NRR), can save more NRR compared to using higher cost products made from Renewable Resources (RR).
- A sustainable marketing concept "Energy Neutral Products (ENP)" is proposed, which can make consumption much more sustainable.

KEYWORDS

Efficiency, sustainable development, economic advantage, Vinnolit optimisation method, "energy neutral products", life cycle cost, life cycle impact, three pillar model of sustainable development

1 INTRODUCTION

Since the conferences of Rio de Janeiro and others Sustainable Development (SD) has been chosen as our guide line for future development. SD intends to create acceptable conditions for the people living today all over the world, without diminishing the chances for future generations. To achieve this target, ecological, economical and social development must be optimised. This “three pillar model of SD” is generally accepted by all societal groups, such as citizens, politicians, NGO’s, industry etc.; only the relative importance of the different pillars is a source of debate between these groups.

It is strange to observe that whereas in most SD activities this three pillar model is accepted as a starting point, in practice only ecological measures to support SD are discussed, worked out and proposed. In this way the economical pillar most often is not considered, perhaps a reaction to the past, where the economical pillar was often the most important criterion. This contribution does not debate the ranking of the three pillars. It shows that integration of economic aspects does not decrease but can increase the speed of SD considerably.

To speed up SD the economical advantage of lower cost products is partly or wholly used to finance ecological sensitive optimisations (Vinnolit Optimisation Method). A quantitative drinking and waste water pipe example from the building sector shows that the ecological gain from such a procedure is much higher than the gain from just choosing between comparable products on basis of ecological impact alone. In this sense we state that taking economical cost into account one can speed up SD considerably. A sustainable marketing concept “energy neutral products” can overcome the problem, that volume growth at the macroeconomic level could override ecoefficient improvements on the product level.

2 CONVERSION OF ECONOMICAL INTO ECOLOGICAL ADVANTAGES

2.1 Quantitative economical assessment

The most important economical impact is cost, which is taken as Life Cycle Cost (LCC) paid by the consumer. In some cases not all cost types arise at the same time; e.g. cost for recycling/disposal for long lasting products arises many years after the purchase of the product. In this case discounting methods are used to calculate future expenditures back to the time of purchase.

2.2 Quantitative ecological assessment

Ecological impacts are quantified by Life Cycle Impact (LCI) analysis. There are two principal possibilities to quantitatively and jointly assess ecological and economical impacts:

- Ecological impacts are monetised. There are different methods to do it. Damage repair cost calculates the cost which is necessary to repair the damage produced by the impact in question. Avoidance cost calculates the cost which is necessary to avoid the impact in question. This monetised ecological cost is added to the normal LCC and products are compared on a pure cost basis. These methods are called “monetising methods”. They are developing now but still show very big variations in specific cost; they are not discussed further in this contribution.

- Economical advantages are used to finance ecologically sensible activities. This method is called Vinnolit Optimisation Method and is explained in greater detail.

2.3 Vinnolit Optimisation Method

The Vinnolit Optimisation Method (VOM) [Spindler 1999] calculates ecological benefits which can be financed with a certain amount of money invested into an ecologically sensible optimisation, like investment into better thermal insulation of houses. This amount of money can be the cost advantage of a lower compared to a higher LCC product. An expanded product system A' is then created which consists of the lower LCC product system A plus the optimisation financed by the cost advantage. The expanded system A' is ecologically analysed by LCI and compared with the higher LCC product system B. By definition both product groups A' and B cost the same, but differ with regard to LCI. In some way VOM is an expansion of LCI work to systems which cost the same.

2.4 Ecologically sensible optimisations

Above a better thermal insulation of houses was mentioned. Better thermal insulation saves heating energy and saves thus energetic resources, mostly fossil, non renewable fuels and emissions connected to their incineration. Better thermal insulation is chosen here because of it's huge influence on important ecological impacts, like energy demand, Global Warming (GW) emissions etc..

2.5 VOM: A pipe example

In the most recent LCI study of Swiss EMPA on pipes [Reusser 1998] a settlement of 21 houses is equipped with drinking water and waste water pipes. The LCI study does also include digging of pipe ditches, laying the pipes and connecting them. The pipes used in this study were PVC, PE, iron and stoneware pipes. The LCI results do not differ widely. PVC pipes score rather well, especially when one includes the latest ecoprofile data for PVC and PE [APME 2004]. Pipe laying does not influence the result very much, since it is more or less the same for all pipes. The much lower weight of plastic pipes which alleviates the laying work was not assessed in this LCI study.

A parallel cost study was undertaken. It showed a 22% cost advantage in favour of the PVC pipe system compared to the next lowest cost system. Other cost calculations originating from different sources show similar advantages for PVC pipe systems. Other LCI studies for pipe systems can come to different classifications for the pipe materials as shows e.g. the latest review ordered by the European Commission [PE Europe 2004]. But these small differences do not influence our calculations.

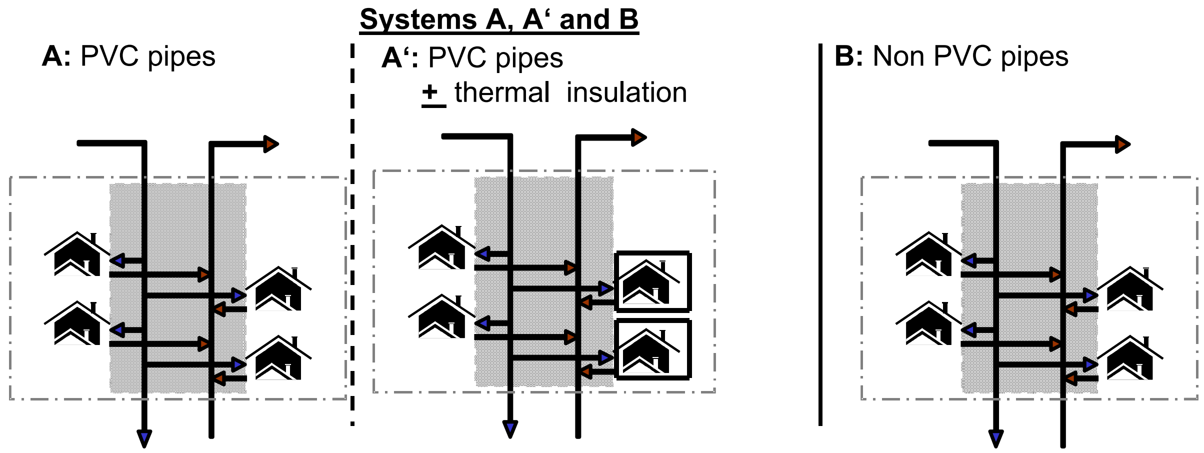

Figure 1: 21 houses are managed for drinking water supply and waste water disposal.

With the 22% cost difference an optimised PVC pipe system A' is created, consisting of PVC pipes together with better thermal insulation of some house walls. This is shown qualitatively in fig. 1.

Table 1 shows quantitative LCI and LCC results of these three different systems. LCI results are shown only for energy demand and Global Warming Potential (GWP). 2nd row shows results for the PVC pipe system A, 4th row for the alternative pipe systems B indicating the lowest and highest impacts, and 3rd row for the optimised PVC pipe system A'. The quantity of thermal insulation has been chosen so that the cost of A' is equal to the lowest cost alternative pipe system B. LCC and LCI results for the thermal wall insulating system have been taken from an eco-efficiency study based on specific work in a German town [Kicherer 2000]. Negative numbers describe cost, energy demands, emissions, positive numbers describe saved energy demand, saved emissions.

	Product system A	Product system A'	Product system B
	PVC pipe systems (lowest cost)	(cost adjusted to B)	Non PVC pipe systems (higher cost)
	Only pipes	Pipes + 340 m^2 thermal insulation	Only pipes
Energy demand (GJ)	- 495	+ 3 543	- 523, ..., 676
Fossil feedstock (GJ)	- 143	+ 3 000	- 10, ..., 281
GWP (t CO_2 equiv.)	- 24	+ 206	- 24, ..., 34
Cost (€)	- 68 000	- 83 100	- 83 100, ..., 91 000

Table 1: LCI and LCC results for different pipe materials, i.e. systems A, A' and B.

The main result from tab. 1 shows the huge ecological potential in terms of energy and greenhouse gas savings if one uses the low cost pipe system and invests the saved cost into thermal insulation of a house as one example. The ecological impact from both non-optimised products A and B are negative, whereas the ecological impact of the optimised product A' is many times higher and positive!

3 DISCUSSION AND CONSEQUENCES

3.1 Positive correlation of low cost with the three pillars of SD

The economical quality of low LCC products is connected to all three pillars of SD in a positive way:

- A low LCC is beneficial for an economical SD, because economical resources are scarce (like ecological ones)!

- A low LCC is beneficial for a socially SD, because many people in industrialised and even more so in 3rd world countries can better afford lower LCC products than higher ones. This is most important for products serving essential needs, such as medical products or drinking water pipes.

- The use of products with low LCC saves money, which can be invested for positive socially SD (e.g. financing more teachers to better educate people) and/or a positive ecologically SD (e.g. financing better thermal insulation of buildings to save heating energy).
  Both possibilities can be seen as positive rebound effects.

3.2 Negative correlation of low cost with the three pillars of SD

Saving money can be a chance, since positive rebound effects can be created by investing this money into socially and ecologically sensible optimisations. But some fear, that these positive effects could potentially be neutralised or even overcompensated by negative rebound effects. People who save money with low LCC products can spend it on non-ecological and non-social purchases. Positive and negative rebound effects have to be discussed together. Interestingly, calculations show that negative rebound effects are several times more expensive than positive ones, at least for many important areas.

3.3 External effects

LCC in general has to include or internalise external cost (EC). This is obvious for comparisons of different products where LCC can be distorted by EC for one or all of the products. There are many different kinds of EC to be discussed, like subsidies and waste disposal cost paid by the public, greenhouse gas emissions paid by nobody etc.. There are no subsidies paid in the example discussed here, and end of life costs have to be included in the LCC data. Therefore only the influence of ecological EC has to be discussed, like for greenhouse gases.

Doing so e.g. for GWP we find that even the highest cost figures found in literature (200 €/t CO_2) hardly influence the LCC of the products studied here. It must be stressed that this cost of 200 €/t is an avoidance cost (see 2.2) and therefor includes not only the reduction of CO_2 emissions, but also the reduction of emissions other than CO_2 and of the energy demand.

3.4 "Energy Neutral Products", a sustainable marketing concept

"Energy Neutral Products" is a concept how to use the economic quality of products in order to support SD. We choose here the ecological criterion "energy demand", since consumption of energy is a very important ecological topic connected to production and use of products; directly connected to this is another important topic, namely emissions with a GWP, like CO_2.

In this concept of "Energy Neutral Products" products are sold on the market in two parallel ways: They are sold at the same price as today as "normal products" and in addition as "Energy Neutral Products" with a higher price; this higher price is paid voluntarily and is used to finance energy saving activities, like financing thermal insulation in the example given above. The price increase is high enough as to save the whole energy demand needed to produce the product in question. For products studied so far like pipe systems, windows, packaging etc. this price increase is in the order of just 3% of the normal price.

An PVC pipe system as shown in tab. 1 would cost in it's "Energy Neutral Products" version some 70 000 € instead of some 68 000 € in it's normal version. The "Energy Neutral Products" pipe system would have a zero energy demand (and a slightly positive GWP score)!

Table 2 gives some further examples for the low cost increase of "Energy Neutral Products" compared to "normal" both for some PVC products as for their alternatives.

The price difference between "normal" and "Energy Neutral Products" is managed by a neutral, trustworthy organisation. It gives a receipt for the "Energy Neutral Products", uses the money for ecologically sensible optimisations etc.. The procedure could be similar to what has been realised in connection with " CO_2 neutral flights" [MYCLIMATE 2000].

	Cost (€)			
	PVC		PVC-alternatives	
	Normal	ENP	Normal	ENP
Window	300	+ 10	550 up to 650	+ 5 up to + 20
Pipe system	68 000	+ 2 000	83 100 up to 91 000	+ 2 000 up to + 2 500
Infusion bag (1 kg)	5 - 10	+ 0.3	?	>= 0.3

Table 2: Some examples for cost increases to provide "Energy Neutral Products" (ENP).

Some advantages of this concept:

- "Energy Neutral Products" are with some 3% price increase not considerably more expensive compared to normal products. This is a price increase many consumers would accept to pay in order to compensate an essential part of the negative ecological impacts of their consumption.

- Companies offering both normal products and "Energy Neutral Products" help to support SD in a very efficient way.

- "Energy Neutral Products" still have small uncompensated ecological impacts but to a much lower content than normal products. Thus "Energy Neutral Products" are an important step to more sustainable consumption. Choosing products only on an ecological basis does not lead so far.

Other variants of this concept can be "NRR Neutral Products" or "Fossil Carbon Neutral Products", "GWP Neutral Products" etc.. As with "Energy Neutral Products" in these cases the price increase is used to compensate for demand of NRR or fossil carbon, for GWP etc.. It is clear that all these variants do contribute to savings not only in all these ecological criteria. Instead of just compensating these demands or emissions one could also overcompensate them of course.

3.5 Some general results

- Quantitative results (tab. 1, 2nd and 4th row) show, that in the pipe example an ecological optimisation can achieve savings in the order of 40% maximum. This is in the same order of magnitude, which is deduced from other LCI studies.
- Quantitative results (tab. 1, 2nd and 3rd row) show, that with a small economical advantage of 22% relative to the cost of the system A one can finance ecological savings in the order of 800 to 900% relative to the ecological impacts of the system A. In this sense economical advantages have a huge ecological optimisation potential.
- These ecological savings are realised by saving mostly NRR. Therefor low LCC products even if produced from NRR like PVC pipes can save these NRR much more efficient than higher LCC products made from RR. This is also shown in 2nd result line in tab. 1.
- Eco-efficiency is criticised by some as not sufficient to sustain SD, since improvements on the product level (some 40% in the pipe example between the "best" and the "worst" pipe system) can be overridden by volume growth effects. With ENP even growth effects can be compensated, since the positive rebound effects (see 3.1 and 3.2) would grow as well.

ABREVIATIONS

CO_2 Carbon dioxide
EC External Cost
ENP Energy neutral products

GW(P) Global Warming (Potential)
LCC Life Cycle Cost
LCI Life Cycle Impact
NGO Non Governmental Organisations
NRR Non Renewable Resources
PE Polyethylene
PVC Polyvinylchloride
RR Renewable Resources
SD Sustainable Development
VOM Vinnolit Optimisation Method

REFERENCES

APME (Association of Plastics Manufacturers in Europe) 1993/2004, Eco-profiles of the European polymer industry, Report 1 to 16, Brussels.
Kicherer, A. 2000, Ökoeffizienz-Analyse: Wärmedämm-Verbundsystem mit Neopor Dämmung bei der
Brunckviertelsanierung; Ludwigshafen. Study is found in the internet under http://corporate.basf.com/file/15768.file1?id=1ox425mH2bcp.Cw
MYCLIMATE, an organisation to compensate global warming emissions created in flights, see e.g. www.myclimate.org
PE Europe 2004, Life Cycle Assessment of PVC and of principal competing materials, commissioned by the European Commission, Brussels. Project coordination PE Europe GmbH, Stuttgart (M. Baitz, J. Kreißig, E. Byrne).
Reusser, L. 1999, Ökobilanz von Rohrleitungssystemen, EMPA, St. Gallen.
Spindler, E-J. 1999, Integration der monetären Kosten in Ökobilanzen, UWSF – Z. Umweltchem. Ökotox. 11 (5) 299-302

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Achieving Reliability and Durability in Green Roofs


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ABSTRACT

Green roofs can contribute significantly to reducing the temperature of urban "heat islands," protect waterproofing membranes, improve the roof of a building as heat insulator, and reduce the intensity of stormwater drainage. To obtain all these advantages, the quality of the component materials must be assured. Even more important is the efficiency of the assembly of the whole design, to make the roof function without frequent repairs or even redesign. The future of green roofs depends on a technically valid and efficient assembly of its components. This paper assembles lessons learned from roofing and waterproofing design learned from past projects.

KEYWORDS

Flashing, Drainage, Durability, Green Roofs, Waterproofing

1. INTRODUCTION

1.1 Green Roofs: A "New" Technology at Risk

Green roofing systems are being installed with increasing frequency on roofs all over the world. They are becoming more and more popular as building owners and architects seek to capitalize on their benefits. However, the rise in interest in green roofing may peak too quickly and green roofs may never achieve their full potential in the marketplace, if they are not designed and installed to provide reliable, long term service. This paper uses lessons learned from past roofing systems, green roofing systems, and similar construction such as plaza waterproofing, to show how green roofing systems can be installed to provide reliable, long-term roofing service.

1.2 Benefits of Green Roofs

Green roofs, i.e., roofs covered with plantings, have existed for a long time, although mostly as a result of slow growth of wind-borne plantings on organic roofs. The efficiency of such roofs is unknown, as in prehistoric days, the expectations for protection from the elements were significantly more modest as they are today. The present move towards green roofs is based not so much on sheltering requirements, but on the realization that our whole environment must be changed for the following reasons:

- Counteract the "heat island" syndrome, in which roofs absorb solar radiation and re-radiate heat to produce significantly higher general temperatures, including the compounding (iterative) effects of reduced air conditioning.
- Reduce rainfall intensity, allowing reduced stormwater capacity in public drains.
- Reducing temperature extremes on roofing materials, increasing their durability.

A cross section of a green roof is shown on Fig. 1. The sequence of components can vary.

1.3 Target: Durability and Low Maintenance

A new system, such as a green roof, should not lead to leakage problems and/or loss of insulation value (which would exchange one problem for another). The roofing industry has struggled with the physical problems of materials, construction methods and leakage for years. Green roofs need appropriate technology, to assure that environmental gains are not at the expense of durability and reliability.

2. DESIGN FOR SUCCESS

2.1 Durability

Materials in a green roof must be durable. Lessons learned from investigating failed plaza waterproofing system illustrate the expected performance of the materials in green roof systems. Components can fail individually, or in combination.

The sequence of the components (Fig. 1) has not been settled. What comes first? Will there be two drainage layers? How does water pass through a root barrier? How does the layering affect drainage? Will the plant soil clog drainage filters? Is the membrane needed to protect the thermal insulation against water absorption?

2.2 Structural Dead Weight and Water Weight

The structural deck must have adequate capacity to support the obvious additional weight of the plants and soil, as well as the water retained in the soil. The total weight must be assumed to include a water level as high as overflow drainage pipes or scuppers. Structural decks must be adequately sloped to drain (to prevent ponding), continuous (to avoid movements that could affect the membrane), and durable (resistant to deterioration).

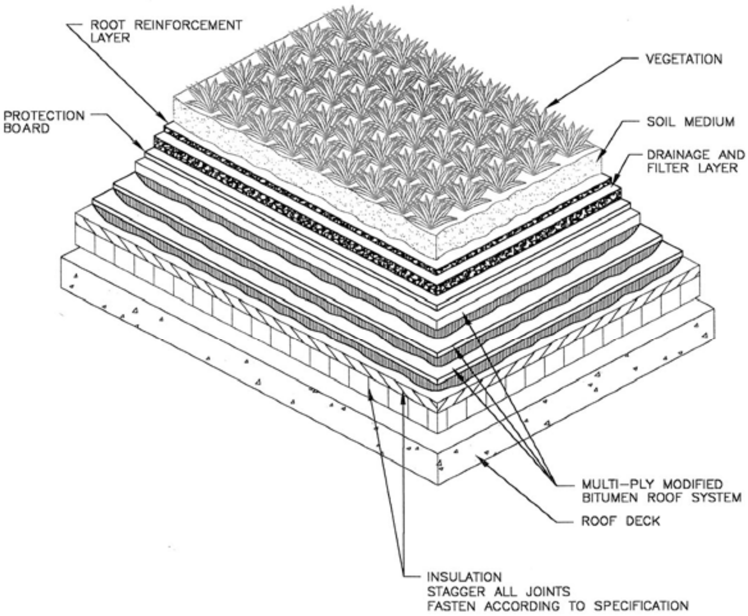

Fig. 1 - GREEN ROOF - TYPICAL SECTION
(Credit: The Garland Company, Inc.)

2.3 Root Barriers

Root barriers have a contradictory purpose. They must prevent root penetration, but must allow water drainage. Tight geotextiles seeded with chemicals to prevent roof penetration may be able to serve both purposes, but the effect of the chemical will, at best, last only a short time and can not be considered adequate for long-term durability. Roots are highly efficient in their search for water, and only strong and continuously sealed membranes can resist their attacks. An effective root barrier, consisting of welded thermoplastics or high-density polyethylene, defeats the purpose of the (necessary) drainage of the planting. Reliance on plants without vertical roots is best, but their efficacy is uncertain. Root barriers should have the following characteristics:

- Easy to apply for continuous protection.
- Durable, i.e., not subject to organic deterioration or movement.
- System to be suitable for “flashings” to prevent perimeter root penetrations.
- Compatibility with components above and below.
- Suitable for a range of potential rooftop plants.

2.4 Drainage Layer

In recent years, effective drainage layers have been developed that can serve as part of a green roof. Drainage layers should have the following characteristics (Table 1 below):

- Provide both capacity and easy flow of the water to drains.
- Maintain its dimensions to avoid loss of drainage capacity.
- Constructed of components that will not corrode or disintegrate.

Material	Advantage	Disadvantage	Typical Trade Names
Gravel wrapped in filter fabric	Does not deteriorate when exposed to moisture. Generally does not require ballast during installation to keep in place.	Thicker than most drainage layers. Heavier (structural deck must be capable of supporting the weight). Requires retaining structures to keep in from migrating around the roof. Limited volume for water flow.	Generic
Plastic high strength "honeycomb" core with filter fabric	Flexible, interlocking to cover entire area	Loose laps in the filter fabric allow debris to enter, contributing to clogging and slower drainage. Debris enters open edges not wrapped with filter fabric.	Miradrain, Hydroduct
Non-woven plastic mesh with filter fabric	High capacity	Can get clogged with fines	Enkadrain
Rigid plastic cells	Interlocking rigid pieces, higher capacity than most. Open cell structure less likely to clog.	Thicker than most drainage composites.	Versicell
Dimpled Plastic Fabric with Filter Fabric	High Compressive Strength; Durable. Less expensive.	Low flow rate compared with other drainage composites. Clogs easily, slowing drainage.	J-Drain 300

Table 1 – DRAINAGE LAYER CHOICES FOR GREEN ROOFS

2.5 Membrane Protection

There are many forces on green roofs that could damage the roof membrane. For example, as in conventional plaza waterproofing systems, the membrane could be damaged by construction operations. On green roofs, increased plant root action can also damage the membrane. The protection layer resists impact puncture and deterioration (low water absorption), and should provide reliable jointing to achieve continuity.

2.6 Membrane

The waterproofing membrane (Table 2) must be designed for long-term durability, because it is below other layers of a green roof; it must be durable, the method of joining seams in the field must be reliable (such as heat-welding), and the membrane must not be vulnerable to splitting from movement in the deck (such as some unreinforced liquid applied systems). Membranes should have the following characteristics:

- Durability – no deterioration such as friability, hardening, softening, shrinkage.
- Low moisture absorption. No wicking into reinforcement at cut edge.
- Allow some moisture vapor diffusion to minimize risk of blistering.

- Permanence and reliability of bonding method – adhesive, bonding tape, welding (for thermoplastic membrane), continuous mopping materials.
- Toughness – resistance to impact damage in application and service.
- Repairability and ability to accept changes and reconstruction.

Material	**Form**	**Advantage**	**Disadvantage**	**Typical Trade Names**
HDPE	Hard/solid	Strong, rigid	Unreliable weld joints. High expansion coefficient. Difficult to obtain continuity. Unwieldy	
EPDM	Flexible	Large sheets good performance record	Unreliable adhesive joints. Tape joints are less risky.	Firestone, Carlisle
TPO, reinforced	Flexible	Weldable	Shorter track record of performance.	Firestone, Versico
PVC, reinforced	Flexible	Weldable joints show good durability	Some brands had unstable plasticizer, causing shrinkage and cracking.	Sarnafil Firestone
CSPE, reinforced	Flexible	Weldable (limited)	Wicking of reinforcement at cut joints; history of coat debonding. Difficult to repair aged membrane.	J. P. Stevens
Hot-applied rubberized asphalt	Hot-applied liquid	Mopping is continuous	Risk of too-thin application.	American Hydrotech
Modified Bitumen	Sheet	Strong, tough	Workmanship dependent.	Garland, Siplast
Asphaltic BUR	Sheet	Mopping is continuous	May be penetrable by roots if not perfect.	Generic
Coal Tar Pitch BUR	Sheet	Mopping is continuous	Health risk during application.	Koppers, Allied
Polyester resin/fleece	Liquid applied	No joints	Can get reflected cracks at joints and new cracks in the substrate. Depends on installer for quality.	Kemper
Self-adhered modified Bitumen (composite)	Self-adhesive sheet	Uniform thickness, low odor, Self-sticking, good bond	Risk of wrinkled application, bridging. Risk of tunneling at laps. Relies on sealants as pentrations.	Carlisle CW 701, Bituthene/ W.R Grace
Two-part latex rubber	Liquid applied	Simple application	Risk of holidays, irregularities, and thin spots. Vulnerable to substrate movement. Workmanship sensitive.	Procor
Reinforced Urethane	Liquid applied	Strong	Workmanship is critical, potential for reversion. Reflected cracking. Curing problems.	Liquid Plastics

Table 2 – MEMBRANE MATERIALS FOR GREEN ROOFS

2.7 Insulation

Insulation is needed for green roofs as much as for conventional roofs. The materials available (Table 3) all have limitations that can make them ineffective when buried. As an example, extruded polystyrene, unless it is exposed to air, can absorb, over time, as much as 200 to 300% by weight when used under a conventional roofing membrane.

Foamed glass, especially when frozen, tends to disintegrate when wet. Other materials perform even worse in the presence of water.

Material	Form	Advantage	Disadvantage	Typical Trade Names
Extruded Polystyrene	Rigid Board	Available with high compressive strength. Close-cell structure makes it more moisture resistant than other boards.	Absorbs moisture when left submerged for long periods of time.	Dow Styrofoam Foamular
Expanded Polystryrene (Bead Board)	Rigid Board	Less expensive than extruded polystyrene.	Absorbs moisture and deteriorates when left submerged (more quickly than extruded polystyrene). Limited compressive strength.	Generics
Polyisocyanurate	Rigid Board	Good thermal insulation value. Available tapered.	Limited compressive strength. Absorbs moisture – do not use above the membrane.	Manville Hunter ACFoam
Cellular Glass	Rigid Board	High compressive strength. Less moisture sensitive than other boards.	Difficult to work with: friable, will dissolve when frozen while wet.	Foamglas
High Density Wood Fiberboard	Rigid Board	Inexpensive. Reasonably strong.	Absorbs moisture. Deteriorates if exposed to moisture. Must be below the membrane. Limited availability in different thicknesses.	Celolex Hubert Generic

Table 3 – TABLE OF INSULATION MATERIALS FOR GREEN ROOFS

Insulation materials must be dimensionally stable when applied. Some require aging to a constant dimension. The membrane should be above the insulation to prevent breakdown from water absorption. Insulation should have the following function:

- High and stable thermal resistance.
- Good impact resistance and adequate compressive strength.
- Dimensional stability for durability and to prevent damage to other components.
- Attenuate movement between different levels of the roofing system.
- Acceptable fire resistance.
- Resistance to deterioration from insects and biological growth.
- No health risks – free of hazardous components.

3. EXPERIENCE

3.1 Do All Green Roofs Leak? – Greater Risk of Leakage from Green Roofs?

The cost of resolving the leaks in green roofs is much higher than for conventional roofing. Locating the source of the leak is more difficult (because water travels laterally within the layers of the roof system), and because significant overburden must be removed to investigate (and repair) the leakage problem. Many of the leak problems of green roofing systems are the same as those encountered in conventional waterproofing systems:

- **Flashing height.** At all perimeters and penetrations the roof membrane must extend at least 200 mm (8 in.) above the finished surface of the soil, paving, or other overburden. The base flashing should be protected from damage by a metal skirt flashing that is integrated with the penetration or wall system above.

- **Flashing construction.** Flashing must be constructed of durable materials that can be integrated reliably with the field membrane. The flashing must be installed over a solid substrate with no bridged or unsupported areas. For example, leakage at building expansion joints is a frequent problem on many of the roofs and decks we investigate because the waterproofing is difficult to seal watertight over gap. A flexible substrate for the membrane should be installed over the gap (such as a rubber bellows), and the membrane should be installed continuous over the joint on this substrate.
- **Penetrations.** Roof penetrations are best covered and surrounded by removable components such as stones or precast concrete for easier access.
- **Leakage from Walls that Rise Above the Roof.** A continuous through-wall flashing system should be constructed at the base of any wall that rises above a roof, to collect water in the wall system and direct it back to the exterior.

3.2 Design of Drainage and Water Retention

The surface of the roof membrane must be designed to slope-to-drain in a four-way "inverted diamond" pattern with a minimum 2% slope (1/4 in per ft) wherever possible. Improving drainage will improve the performance and durability of paving systems by allowing them to dry out more readily, particularly in northern climates exposed to freeze-thaw cycles. The leakage volume at a defect in the membrane will be more severe where water is ponded over the defect, rather than if water flows over and around the defect.

3.3 How to Select and Maintain Planting

Plants can have excessive root growth. Roots will inevitably find a path through open laps in a root barrier and attack the roofing membrane. Many root barriers are installed by simply lapping the adjacent sheets; adjacent sheets are often not sealed together. Roots tend to follow the path of the drainage water, and depending upon the porosity of the root barrier; water may accumulate on the root barrier and drain through unsealed laps.

3.4 Efficacy of Insulation and Its Location in the Roofing Assembly

On the basis of many years of experience, we know there is no perfect insulation. To some degree, all current thermal insulations absorb water if exposed to it, and gradually lose their thermal resistance. Below are two random examples of our findings:

One green roof we investigated in Cambridge, Massachusetts, USA, included large blocks of expanded polystyrene insulation ("bead board") above the membrane and buried beneath organic soils and plants. The polystyrene insulation was completely saturated with water and was so deteriorated that it no longer resembled the large blocks it was originally installed as. During the demolition of this roof, the insulation broke up into small pieces, and the debris had to be removed totally.

Several years ago, we investigated many built-up roofs with extruded polystyrene insulation. Because of leakage, the insulation was exposed to some water. We found the insulation of the roofs of the Olympic Village in Grenoble, France, for example, to contain more than 200% of water by weight.

Green roofs are especially vulnerable because they are, by definition, a walking and working surface. The typical composition of such roofs starts with a structural concrete surface that is covered by the waterproofing membrane. On top are the (needed) thermal insulation and the balance of the green roof components. This exposes the insulation directly to water penetration. If the insulation is placed under the waterproofing membrane, the membrane would lack a solid base and be subject physical damage.

Material research on durable and water-resistant insulation is needed to improve the durability of green roofs.

4. FINANCIAL ISSUES

4.1 Installation Costs – What are the additional expenses?

Green roofs cost more than conventional roofing systems because:

- Material cost of additional layers including protection, drainage, soil, and plants.
- Labor to install these additional layers and materials.
- Equipment (such as a crane) to hoist the green roofing materials to the roof.
- Details in green roofing systems are more expensive to construct.
- The structural capacity of the roof deck and framing generally needs to be increased to support the increased weight of green roofing and retained water.

4.2 Maintenance Costs – What are the Additional Expenses?

Water that leaks through a defect in the waterproofing can travel laterally under the membrane and across the deck until it finds a point to pass through the deck. Once the possible location of a membrane defect has been identified, significant amounts of soil and planting have to be moved to expose the underlying layers. Layers of filter fabric, drainage composite, insulation, and protection board must peeled back to expose the roof membrane. Loose pieces of fabric and insulation are prone to blowing around or off the roof, and they must be held down by ballast or other (means without damaging them) if they are to be reused. The seams in these components do not fall directly over one another, and therefore, these components are often cut to limit the size of the excavation rather than excavating further to expose and existing joint.

Once the membrane is exposed, it has to be cleaned, inspected, and water tested. Water testing may be inconclusive however, because it can travel underneath the membrane and across the deck until it finds a point to pass through the deck and into the building. A leak inside the building may be caused by more than one membrane defect.

5. CONCLUSIONS

5.1 What is the Present State of Technology?

The roofing industry has always sought to develop new methods of installing roofing systems; not all have been successful. Roofing consumers should invest in green roofing technology with the most durable and reliable system. The durability of the green roofing system can be compromised by repeated repairs or premature replacement of a designed or poorly installed system. The costs to repair or replace green roofing systems are so high, that they must be properly designed and constructed to justify the benefits claimed by the green roofing industry.

5.2 What New Technology is Needed?

For most roofing systems, the details required to construct the roof membrane are well established. The membrane details for specific roofing membranes are provided by manufacturers and trade groups such as the National Roofing Contractors Association (NRCA), promulgating standard details and specifications.

Details showing how the green roof membrane is terminated at perimeters and how it is integrated with the other components of the green roof system (such as drainage layers, soils, paving, and other features) are newer. These details have not undergone years of refinement (reflecting improved durability) as standard roofing system details have.

Green roofing system manufacturers have developed some standard details for these new systems, but as in the case of generic details for conventional roofs, these green roof details need to be *applied* to specific projects and conditions

To achieve reliability and durability, green roofing systems must be developed for typical conditions, such as drains, roof curbs, parapets, and planters. The increased risks and complexity of a green roofing system should require building owners and contractors to give more attention to the roofing system than they otherwise would. If roofs last longer, this contributes to sustainability by producing less waste and reducing the unnecessary use of raw materials and energy that go into producing and installing a new roofing system.

5.3 How to Design Green Roofs for Durability?

Some of the conditions to be observed to design a green roof to be durable and reliable:

- **Membrane Selection.** Select a durable and reliable roofing membrane with a track record of successful performance in green roofing applications.
- **Develop Project Specific Details.** Do not rely on the membrane manufacturer's standard details alone; the manufacturer's details can be useful to understanding the installation of a particular membrane system, but standard details may not apply necessarily to a particular roofing project. Retain a qualified roofing professional to assist with the design of specific details for the project.
- **Detail Filter Fabric Around Drainage Layer.** Roof details should clearly show the filter fabric wrapped around the edges of the drainage layer, and must show a supplemental piece of filter fabric along all edges. .
- **Provide for Ease of Maintenance.** Design roofing systems, overburden, and details that allow maintenance personnel to access the roofing membrane, flashing, and drains relatively easily without having to rebuild the roof. One method of improving maintenance access is installing the plants and soil layers in "trays" set on pedestals above the roof membrane. Penetrations should be surrounded by removable pavers.
- **Monitor the Construction.** Retain a qualified, full-time, roofing monitor to observe the installation of the green roofing system, not just the membrane.
- **Water Test.** Water test the completed membrane before the balance of the system is installed, it is best to delay the planting for one season. .

6. SUMMARY

The long term success of the green roofing industry depends on the ability of designers, manufacturers, contractors, and owners to install these systems for many years of reliable and durable performance. If the industry installs green roofs that have problems and require frequent repairs or premature replacement, "green roofing" will acquire a bad reputation and building owners will be reluctant to install these systems.

If the roofing industry applies the fundamental lessons learned from the last 200 years of roofing practice to today's green roofs, green roofing appears to be here to stay.

This paper summarizes some of the issues that will contribute to the long-term reliability and success of a green roofing system, but more research is needed some areas including the durability of waterproofing membranes and the water resistance of the thermal insulation. Information about the long-term performance of installed membranes should be collected and analyzed to determine which systems are providing the best performance. This has been done to some degree with traditional low-slope roofing systems, but a more systematic collection of the experience of green roofs and plaza waterproofing would benefit the industry.

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On Reduced Energy Performance of a Solar-Assisted Heat Pump System due to Absorber Coating Degradation

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ABSTRACT

This paper presents a literature study of the degradation of flat plate solar collectors used in Domestic Hot Water (DHW) systems. The scope of the paper is limited to the selective absorber coating of the collector and is focused on the research made in the IEA Task X study. The objective is to investigate the potentials of computer simulations of performance reduction of a solar-assisted heat pump system, due to absorber coating degradation. The Task X study used an approach of accelerated testing to access the extent of absorber coating degradation. Mathematical models describing the coating degradation were used to transform the accelerated results into in-service conditions. A mathematical model of the microclimate in a flat plate collector was also developed. Results showed that the accelerated test procedure is in fairly good agreement, although it contains a number of simplifications, when compared with specimens from in-service collectors that have been working for 3 to 15 years. By using in-service microclimate data and accelerated test results, calculations into approximated real time degradation can be made. The microclimate model that was intended for simulations gave deviating results when compared with measurements, showing that further development is needed. The methodologies from the Task X study could also be useful when assessing the effects of absorber coating degradation on other types of collector systems, e.g. a solar-assisted heat pump system. Such a system is under evaluation in the ongoing EU project named: Endothermic Technology for Energy Efficient Housing in the EU (ENDOHOUSING). The project uses solar-assisted heat pumps to provide the thermal energy to meet space heating, cooling and hot water requirements for domestic houses in different regions of the EU throughout the year. Six demonstration houses (endohouses) will be established and equipped accordingly across the EU and evaluated during the project. If these systems are to be commercially installed and used they must be economically feasible. This places emphasis on the cost, durability and performance of the system. A solar collector is exposed to various strains that will contribute to a degradation of the collector materials, which will decrease its energy performance. These changes will in turn be transposed throughout the system lowering its degree of efficiency that directly relates to the economical aspect of the system. As a result of these changes a performance over time assessment is needed for this type of system, which in turn is in line with the European Construction Products Directive (CPD).

KEYWORDS

Degradation, selective absorber coating, life performance, solar-assisted heat pump system, simulations

NOMENCLATURE

a	parameter (-)
b	parameter (-)
c	power series parameter (-)
E	activation energy (kJ/mol)
F_S	annual solar fraction (-)
k_1	constant slope parameter (-)
k_2	intercept value parameter (-)
R	ideal gas constant (kJ/K mol)
r	accelerated constant corrosion rate (g/m^2 year)
T	test temperature (K)
T_{eff}	effective mean temperature (K)
y	degradation time (time)
PC	performance criterion (-)
RH	relative humidity (%)

Greek letters

α_S	weighted solar absorptance (-)
ε	weighted thermal emittance (-)
τ	time fraction of one year (-)
Δ	difference (-)

Subscripts

A	airtight collector
B	non-airtight collector
H	humidity
0	origin
R	reference
S	service conditions
T	temperature
Zn	zinc
max	maximum value
min	minimum value

1 INTRODUCTION

The possibilities and benefits of using solar-assisted heat pump systems for heating purposes have been studied since the 1970s [Kjellsson 2004]. This type of system can be made up of either unglazed or glazed solar collectors in combination with a heat pump and energy-storage. The EU project Endothermic Technology for Energy Efficient Housing in the EU (ENDOHOUSING) Project No: NNE5-2001-00565 was launched in 2003. The project uses solar-assisted heat pumps to provide the year-round thermal energy to meet, space heating, cooling and hot water requirements for domestic houses in different regions of the EU. This endothermic technique is expected to provide a considerable amount of cost reductions and positive environmental effects. The project is innovative because it delivers higher performances; it improves the overall cost effectiveness and enhances the general public's perspective on the acceptability of solar-assisted heat pump systems by the use of dual-purpose endothermic energy collectors that integrates with the building to form its roof. The collectors (unglazed) blend into their surroundings and do not appear as "add on". Six demonstration houses (endohouses) will be established and equipped accordingly across the latitudes of the EU from 35°-62° and evaluated during the project. If these systems are to be commercially installed and used they must be economically feasible, this means that the system's cost and payback time is of great importance, which applies emphasis on the durability and performance of the system. A solar collector is exposed to various strains as a result of its outdoor placing and in-service use. This contributes to a degradation process of the collector material that will decrease its energy performance. These changes will in turn be transposed throughout the system lowering its degree of efficiency, which directly relates to the economical aspect of the system. As a result of these changes, a performance over time assessment is needed for this type of system. In turn this is in line with the European Construction Products Directive (CPD) [Council directive 89/106/ECC]. Extensive research on solar collector material durability has been made in the IEA Task X "Solar Materials Research And Development" of the International Energy Agency Solar Heating and Cooling programme [Carlsson *et al.* 1994]. The work was organised as a case study in which commercially available selective absorber coatings were studied. These coatings were used in single-glazed flat plate solar collectors for Domestic Hot Water (DHW) production. An accelerated test procedure was developed to test the life performance of the absorber coatings.

1.1 Objectives

This paper presents a literature study of the degradation of flat plate solar collectors used in DHW systems and the effect it has on their energy performance. The scope of the paper is limited to the selective absorber coating of the collector and is focused on the research made in the IEA Task X study. The objective is to investigate the potential of computer simulations when assessing the performance reduction of a solar-assisted heat pump system, due to absorber coating degradation. By simulating the microclimate in the collector at varying weather conditions throughout several years, the amount of strain that affects the coating could be estimated. If the degradation mechanisms that act on an absorber coating and influence the system performance are known, the extent of approximated energy performance reduction could be calculated.

2 SELECTIVE ABSORBER COATING

Solar collectors can have different designs; the most common type used for DHW purposes are flat plate collectors. An important part of all collectors is the absorber coating. It should be designed to absorb as much as possible from the incoming solar radiation. To improve the thermal performance, flat plate solar collectors use a spectrally selective solar absorber coating. This type of coating is important regarding the performance and its durability is vital. The concept of a selective surface/coating is to have a high solar absorptance and low long wave emittance. Degradation studies of selective absorber coatings within the Task X study were focused on four commercially available products based on two types of coatings: black chromium on stainless steel and copper sheets, and nickel pigmented anodised aluminium. Black chrome coatings consist of small metallic chromium particles and chromium oxide. The metallic chromium particles are small and are embedded in the oxide phase. Black chrome coatings can be electrodeposited on various substrates. The high solar absorptance of these coatings is due to the metallic chromium particles and a surface roughness effect. Nickel pigmented anodised aluminium coatings are produced by anodising aluminium sheets in a phosphoric acid solution to generate a pours layer of aluminium oxide on the substrate surface. This layer is then used as a matrix for small metallic nickel particles, which are made to precipitate in the pores of the aluminium oxide layer in a process of electrochemical reduction of nickel from a solution containing $NiSO_4$. The high solar absorptance of these coatings is due to metallic nickel particles, which are located at the bottom of the pores of the aluminium oxide.

3 DEGRADATION OF SELECTIVE ABSORBER COATINGS

There can be many environmental factors that might cause degradation of an absorber coating. To be able to identify these factors it is important to have an understanding of what the dominating mechanisms of degradation may be. For the absorber coatings tested in the Task X case study [Carlsson *et al.* 1994], the environmental factors that may cause loss in optical performance were of interest. These were identified as high temperature, high humidity and moisture, airborne pollutants and solar radiation. The effects of these environmental parameters, which were considered to be the dominating degradation factors, were investigated by using an accelerated ageing test program that was set up by Task X participants. The program consisted of four different tests related to: high-temperature degradation, degradation due to condensed water and high humidity, degradation caused by sulphur dioxide as an airborne pollutant and UV degradation, *see* [Carlsson *et al.* 1994]. For interpretation of the ageing test results, the analysis of spectral, build-up and compositional changes induced by artificial and natural ageing where performed, in order to identify mechanisms of degradation. By using theoretical tolls and techniques for surface analysis, dominating mechanisms of degradation of the studied absorber coatings could be identified, *see Table 1*. The tests concluded that UV radiation is not a serious degradation factor for the investigated coatings.

Black chrome coating	High-temperature degradation in absorptance (α_s)	Oxidation of metallic chromium to chromium oxide
Nickel pigmented anodised aluminium coatings	High-temperature degradation in absorptance (α_s)	Oxidation of metallic nickel to nickel oxide
	Degradation in emittance (ε) during condensation and at high humidities	Hydratization of Al_2O_3 to Pseudoboehmite and to Boehmite
	Degradation in absorptance (α_s) at SO_2 exposures and at high humidities	Electrochemical oxidation of metallic nickel to nickel sulphate

Table 1. Dominant mechanisms of degradation of absorber coatings in the Task X case study.

3.1 Microclimate in a flat plate solar collector

In order to determine the expected environmental stress on the absorber coating, which relates to the environmental factors of interest, measurements of microclimate data representing typical service conditions for absorber coatings in single glazed flat plate collectors used for DHW production, was performed in the Task X study. The measurements were carried out in Rapperswil, Switzerland for a time period of three years, *see* [Carlsson *et al.* 1994]. The obtained measurements were also used for calculations and formulations of theoretical models of the microclimate in the collector. The Task X group found it rather straightforward to calculate the mean solar absorber plate temperature by using existing mathematical models for flat plate solar collectors, *see* [Duffie & Beckman 1990]. But to calculate humidity conditions on the absorber plate surface under in-use conditions was more complicated. A heat- and mass-transfer model was developed by Van den Linden *et al.* [1990]. The model takes the following processes in account:

- Ventilation caused by density differences between the air inside and outside of the collector, expansion/compression of the air in the collector due to heating up/cooling down of the air during the considered time step.
- Condensation as a consequence of the fact that the dew point of air inside the collector exceeds the temperature of the cover or absorber.
- Evaporation of condensed water on absorber (or cover plate) when the dew point of the air inside the collector is less then the temperature of the absorber (or cover plate).

The heat- and mass-transfer model by Van den Linden *et al.* [1990] was found to deviate when compared with measurements. The measured RH was about 5-10 % higher than the calculated in the air gap during night conditions. Despite these deviations the correspondence between measured and calculated humidities was seen as fairly accurate. The model by Van den Linden *et al.* [1990] does not take into account the transportation of airborne pollutants into the collector due to high complexity, and the ventilation rate through the collector is considered as independent of the ambient wind conditions. The model is based on non-hygroscopic effects in the collector, but it could be extended so that it is included. A heat- and mass-transfer model, which is based on the Van den Linden *et al.* [1990] model, was also developed by Holck *et al.* [2003]. The model takes in account the influence of wind and the hygroscopic effects of the insulation materials on the humidity level inside the collector. The modelling of the collector ventilation, caused by wind, was derived by empirical correlation and the moisture buffering effects of the insulation was expressed with a transient model of the heat- and mass-transfer in the insulation. Using microclimate measurements performed in test collectors located throughout Europe validated the model. The model showed a good correlation between simulated and measured humidities in the air gap, although the simulated humidity values were slightly lower during night conditions. The deviations become much larger when the influence of wind was excluded from the model. The model is currently used in a computer program named: Modelling Of Microclimate In

Collectors (MOMIC), and is meant to be used by designers to study the effects of modifications to the collector design with respect to the microclimate.

3.2 Mathematical modelling

The results from the different accelerated tests performed in the Task X study were used to mathematically model the life performance of absorber coatings. When applying the mathematical model, time transformation functions in terms of Arrhenius relations are used. Therefore the following simplifications are introduced:

- When calculating the extent of degradation, only the time period at a certain state of environmental influence is important, not the history of changes in the environmental stress factors with time.
- Degradation processes taking place under service conditions are proceed by the same mechanism as those conducted at accelerated aging tests.
- Different processes of degradation proceed in parallel and independently of each other. The contributions of the different processes to the changes in solar absorptance and thermal emittance are additive.

The results from the accelerated high-temperature degradation tests in the Task X study were modelled by applying the Arrhenius equation for the temperature dependence expressed as:

$$\ln y = E_T / R \cdot \left(T^{-1} - T_R^{-1}\right) + \ln\left(\sum_{n=0}^{n} c_n \left(-\Delta\alpha_S\right)^n\right) \tag{1}$$

Values for E_T and the parameters c_0-c_n are determined by way of regression analysis from experimental data. To transform the life data from the high temperature accelerated tests into time under in-service conditions, the effective mean temperature, $T_{eff,T}$, is introduced and expressed as:

$$T_{eff,T} = \frac{E_T}{R} \cdot \left[\ln\left(\left[\int_{T_{T,\min}}^{T_{T,\max}} \frac{f(T)}{e^{\left(\frac{E_T}{R\cdot T}\right)}}\, dT\right]^{-1}\right)\right]^{-1} \tag{2}$$

$f(T)$ is the yearly-based frequency function for the absorber in-service temperature. The effective mean temperature is defined as the constant temperature resulting in the same degradation in the same time period as the fluctuating load. $T_{eff,T}$ is then applied to the Arrhenius equation for a time transformation into in-service degradation: $y_S = y_R \cdot e^{\frac{E_T}{R}\left(T_{eff,T}^{-1} - T_R^{-1}\right)}$ (3)

High humidity and condensation degradation is expressed with the same analogy as Eq. (1).

$$\ln y = E_{H,T} / R \cdot \left(T^{-1} - T_R^{-1}\right) + \ln\left(\sum_{n=0}^{n} d_n \left(\Delta\varepsilon\right)^n\right) \tag{4}$$

The effective mean absorber temperature when the RH >99 %, $T_{eff,H}$, is expressed as:

$$T_{eff,H} = \frac{E_{H,T}}{R} \cdot \left[\ln\left(\left[\int_{T_{H,\min}}^{T_{H,\max}} \frac{f_H(T)}{e^{\left(\frac{E_{H,T}}{R\cdot T}\right)}}\, dT\right]^{-1}\right)\right]^{-1} \tag{5}$$

$f_H(T)$ is the yearly based frequency function for the absorber in-service temperature when the RH inside the collector is >99 %, $T_{H,\min}$ is at 0°C. The time transformation function for the high RH and condensation degradation into in-service degradation is expressed as:

$$y_S = y_R \cdot e^{\frac{E_{H,T}}{R}\left(T_{eff,H}^{-1} - T_R^{-1}\right)} \cdot \tau_H^{-1} \tag{6}$$

τ_H is the time fraction of the year when the RH is ≥99 %. The humidity interval of RH >99 % is used as lower values of RH are considered to contribute to a negligible rate of degradation. Due to the complexity of airborne pollutant degradation, metal coupons were use in the Task X study to determine the atmospheric corrosivity inside a collector. The corrosion rate of zinc was taken as a measure to determine the stress on the absorber surface as sulphur dioxide was considered to be the dominating airborne pollutant. Results from the study shows that the collector design is a crucial factor for the corrosion rate. Two classes of collectors are considered.

- Type A: Airtight collectors with a controlled low ventilation rate. The atmospheric corrosivity level corresponding to a corrosion rate of zinc of 0.1 g/m^2 per year.
- Type B: Non-airtight collectors with essentially uncontrolled high ventilation rate. The atmospheric corrosivity level corresponding to a corrosion rate of zinc of 0.3 g/m^2 per year.

The time transformation function for degradation of the optical performance of the absorber coating was assumed to be the same as the corrosion rate of zinc, i.e.

$$y_{S,A} = \frac{y_{R,A} \cdot r_{Zn}}{0.1} \quad \text{and} \quad y_{S,B} = \frac{y_{R,B} \cdot r_{Zn}}{0.3} \tag{7}$$

3.3 Performance analysis of a DHW collector system

In order to investigate the effects of selective absorber coating degradation on the annual performance of a DHW collector system, computer simulations were made by Hollands *et. al* [1992] within the Task X study. The simulations where meant to determine the relationship between the change in solar absorptance, $\Delta\alpha_S$, and thermal emittance, $\Delta\varepsilon$, on a range of different system parameters: geographical location, collector area, hot water set point temperature and collector flow rate, as the decrease in annual solar fraction of the DHW collector system, $\frac{F_S}{F_{S,0}}$, is in the order of 5 % and 10 %.

The results showed that the relationship between $\Delta\alpha_S$ and $\Delta\varepsilon$ has a linear dependence that can be expressed as: $-\Delta\alpha_S = a - \frac{a}{b} \cdot \Delta\varepsilon$ (8)

The parameters a and b depend on the various system parameters. The simulation results showed a nearly constant value of the linear slope, $k_1 = \frac{a}{b} \approx 0.27$, but the intercept value, $k_2 = a$, varied. To investigate the variation in parameter $k_2 = a$ the ratio $\frac{a/\alpha_{S,0}}{\Delta F_S / F_{S,0}}$ was plotted against the initial annual solar fraction, $F_{S,0}$, at different geographical locations. The plots showed that at low solar fractions, $F_{S,O} \leq 0.50$, the ratio $\frac{a/\alpha_{S,0}}{\Delta F_S / F_{S,0}} \approx 1{,}1$ is fairly constant. By defining failure as a decrease in optical performance of the absorber coating, which corresponds to an annual loss in solar fraction of a DHW collector system, the performance criterion *PC* for low fraction systems, less then 50 %, was formulated as: $PC = -\Delta\alpha_S + k_1 \Delta\varepsilon \leq k_2$ (9)

In the Task X case study the performance criterion for the selective absorber coatings was formulated, with a slight modification to $k_1 = 0.25$, as: $PC = -\Delta\alpha_S + 0.25\Delta\varepsilon \leq 0.05$ (10)

3.4 Validation

To validate the results from the accelerated tests in the Task X study, absorber specimens from in-service collectors that were tested for three years in an outdoor studied at Rapperswil, Switzerland were used. The results from the comparison between the accelerated tests and actually observed degradation showed a rather good agreement, although three years of testing time is too short to accurately judge the predicted service live [Carlsson *et al*. 1994]. To better validate the accelerated tests with observed degradation of nickel pigmented anodised aluminium absorber coatings, a study by Carlsson *et al*. [2000, *a*] was performed where coating samples from DHW collector systems that had been operating for up to 15 years were analysed. The results showed a rather high degree of correspondence with accelerated tests. For the anodised aluminium coating in the Task X study and in [Carlsson *et al*. 2000, *a*] the design of the solar collector with respect to ventilation proved to be a crucial factor for the service life. To validate the reproducibility of the accelerated test procedure, three different laboratories were involved in a round robin test. The participating laboratories showed a very similar change in the optical properties of the absorber coatings [Brunold *et al.* 2000]. The test procedure, which is based on the IEA Task X procedure, has been proposed as a standard to the ISO/TC 180 "Solar Energy" and a draft standard has been prepared, ISO/DIS 12952, [Carlsson *et al*. 2000, *b*]. The draft standard is meant to be applicable for the qualification of all kinds of absorber surface materials designed for flat plate collectors used in DHW systems, but it is however particularly suited for the qualification of electroplated and sputtered selective absorber coatings.

4 DISCUSSION

The IEA Task X study is an extensive research of the degradation of selective absorber coatings used in DHW collector systems. Many aspects have been analysed and tested. The study contains a number of simplifications regarding the absorber coating degradation. The simplifications can be summed up as: only a number of specific conditions contribute to coating degradation, and that these conditions can be reproduced and enhanced by accelerated tests. When calculating the extent of degradation, additional simplifications must be made as explained under section 3.2. Although these assumptions are made, comparison between accelerated and observed degradation is in rather close correspondence. When applying the methodologies from the Task X study for testing selective absorber coatings, the results can be used to assess an approximated in-service degradation and the effect it has on the system performance. By simulating the microclimate in a flat plate solar collector, the yearly-based frequency function could be attained and used to recalculate the accelerated degradation into in-service results. It is difficult to obtain accurate simulated values. This problem has been presented by Van den Linden *et al.* [1990], as deviations were found between simulated and measured RH values during night conditions. These values are crucial, because it is during these conditions that high RH values are likely to accrue. The model presented by Holck *et al.* [2003] showed good accuracy, although it gave slight deviations during night conditions, because it takes both wind and hygroscopic effects into account. To be able to simulate the effects of humidity on the absorber coating, accurate calculations are needed, as the humidity level of RH >99 % contributes to a significant rate of absorber coating degradation. The yearly based frequency function used for calculations in the Task X study was based on measured values. The methodologies from the Task X study could also be useful when assessing the effects of absorber coating degradation on other collector systems, e.g. solar-assisted heat pump systems.

5 R&D NEEDS AND FUTURE WORK

Although the degradation models of the Task X study are approximations, they can be useful for making rough estimations of service lives from accelerated ageing data. If a proper heat- and mass-transfer model is used, the yearly-based frequency function could be simulated for various collector- and system-types at different geographical locations. These results can then be used in time transformation calculations of accelerated absorber coating degradation. As a starting point, the heat- and mass-transfer models by Van den Linden *et al.* [1990] and Holck *et al.* [2003] can be used for further studies for attaining an accurate model for computer simulations of humidity conditions in a flat plate solar collector. The model presented by Holck *et al.* [2003] achieved good accuracy, but it still needs to be evaluated to ensure its sufficiency for simulating a proper frequency function. Further studies are therefore needed for evaluating the available models and attaining one that is adequate. The use of Computational Fluid Dynamics (CFD) simulations could be a useful tool when assessing or improving the microclimate modelling. Using simplified models for microclimate simulations results in low computing time, an advantage compared to an entirely based CFD model. This makes it easier to perform annual calculations and parameter studies. The methodologies developed in the Task X study could also be applicable for assessing the effects of absorber coating degradation of solar-assisted heat pump systems. If different changes in absorber coating properties are simulated for a solar-assisted heat pump system, the effect this has on the system performance can be investigated. This might lead to a new formulation of the absorber coating performance criterion. It is anticipated that this type of parameter study will be performed on the ENDOHOUSING system.

REFERENCES

Brunold, S., Frei. U., Carlsson. B., Möller. M., Köhl. M. 2000, "Round robin on accelerated life testing of solar absorber surface durability", *Solar Energy Materials and Solar Cells 61, pp 239-253.*

Carlsson, B., Frei. U., Köhl. M., Möller. K. 1994, "Accelerated life testing of solar energy materials – Case study of some selective solar absorber coating materials for DHW systems", *A technical Report of Task X of the International energy Agency programme on Solar Heating and Cooling, February 1994, SP-Report 1994:13, ISBN 91-7848-472-3.*

Carlsson, B., Möller. K., Frei. U., Brunold. S., Köhl. M. 2000, *a*, "Comparison between predicted and actually observed in-service degradation of a nickel pigmented anodized aluminium absorber coating for solar DHW systems", *Solar Energy Materials and Solar Cells 61, pp 223-238.*

Carlsson, B., Möller. K., Köhl. M., Frei. U., Brunold. S. 2000, *b*, "Qualification test procedure for solar absorber surface durability", *Solar Energy Materials and Solar Cells 61, pp 255-275.*

Duffie, J.A., Beckman. W.A. 1991, "Solar Engineering of Thermal Processes 2^{ed}", *Wiley/Interscience, New York, ISBN: 0-471-51056-4.*

Holck, O., Svendsen. S., Brunold. S., Frei. U., Köhl. M., Heck. M., Oversloot. H. 2003, "Solar collector design with respect to moisture problems", *Solar Energy 75, pp 269-276.*

Hollands, K.G.T., Karagiozis. A., Brunger. A.P. 1992, "Effect of selective surface degradation on the performance of solar water heating systems", *Solar Energy Materials and Solar Cells 25, pp 125-141.*

Kjellsson, E. 2004, "Solvärme i bostäder med analys av kombinationen solfångare och bergvärmepump" (in Swedish), *Licentiate Thesis, Division of Building Physics, Lund Institute of Technology, Lund, Sweden, Report TVBH-3047, ISBN 91-88722-32-5.*

The Council of the European Communities 1988, "Council directive of 21 December 1988 on the approximation of laws, regulations and administrative provisions of the Member States relating to construction products", *The Council of the European Communities, 1988, Brussels. 98/106/EEC.*

Van den Linden, J., Havinga. J., Kowalczyk. W. 1990, "Microclimate in Solar Collectors", *Reports TPD No. 914.012, TPD-HWM-RPT-90-052, TNO-TU Delft, Report for IEA SHC Task X.*

IBO Passivhaus Bauteilkatalog – a catalogue of building elements specified for Passivhaus standard

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ABSTRACT

We present the IBO Passivhaus Bauteilkatalog, a catalogue of construction details which will be published in 2005 in German and English langugage. IBO, the Austrian Institute for Healthy and Ecological Building has a record in this field as author of Ökologischer Bauteilkatalog – Bewertete gängige Konstruktionen [Catalogue of ecologically assessed building elements] . Wien, New York: Springer 1999.

Contents

Collection of building elements, with several variants, specified for Passivhaus standard and low-energy building design ,

- with technical description,
- physical parameters and
- ecological life cycle analysis.

Technical descriptions focus on airtight building component connections without thermal bridges, and on technical safety, amended with information on production processes, on prefabrication and on demands on building site management.
The physical discussion treats thermal-, noise-, and fire protection, diffusion of vapour and thermal storage characteristics.
Topics of the ecological analysis are ecological impact categories of building materials – Global Warming Potential (GWP 100), Acidification (AP), Primary Energy Input (non renewable) (PEI nr) – durability and maintenance needs of constructions as well as demolition, recycling and disposal.

Technical advice on avoiding hazardous influences (fibres, dusts, toxic exhalations) from building materials during construction and during use of the building is given for each building element.

Attached are cost assessments for Austria for all building elements described.

KEYWORDS

passive house standard, building elements, building element connections, physical analysis, ecological impact analysis

INTRODUCTION

Catalogues of building elements are a useful source of reference for students, planners, construction companies and their clients. They demonstrate the state of art by giving examples and they provide all critical data.
What can be regarded as state of art in building construction changes these days with the development of highly energy-saving building envelopes. And what can be regarded as complete set of data to be provided to describe all relevant aspects of a building element changes as well. Data on the environmental impact of producing building material and on the impact of (separable or inseparable) connections of the components of a building element on their recycling properties become available. Ecological data emerge as a third dimension to consider – along with technical and economic data.
The IBO Passivhaus catalogue of construction elements integrates expertise for building according to the Passivhaus standard – probably the most advanced building standard for energy-conscious construction – and ecological analysis and optimisation of building construction.

PASSIV HAUS STANDARD

The Passivhaus standard has been developed by Wolfgang Feist [1998] and collaborators. The standard defines an building envelope specified to provide thermal comfort with

- an annual maximum energy requirement ≤ 15 kWh/m2 (residential surface area)
- an annual maximum primary energy input for all services (heating, ventilation, warm water supply, domestic electricity) of ≤ 120 kWh/m2, or ≤ 40 kWh/m2 if domestic electicity is not considered [Passivhaus Insititut 2004]
- a maximum heating capacity ≤10W/m2.

To achieve such little energy requirements under Central European climate conditions, passive houses need a.o.

- airtight building envelopes (n50 ≤ 0,6/h)),
- controlled ventilation with efficient heat recovery (recovery rate > 75 %).
- thermal losses through the envelope ≤ 0,15 W/m2.K for opaque elements, ≤ 0,7 W/m2.K for window glazing, and ≤ 0,8 W/m2.K for complete windows (frame and glass).
- Linear thermal bridges (outside) ≥ 0,01 W/m.K should be avoided [Feist 1998].

So far, several thousand passive houses have been built in Europe at building costs that not necessarily exceed costs of standard reference buildings by more than 5% [Kaufmann, B. et al. 2002] . A EU-funded THERMIE project, CEPHEUS, built and monitored building projects comprising 221 residential units in Austria, France, Germany, Sweden, and Switzerland [Krapmeier & Drössler 2001, www.cepheus.de].
In Austria, passive house construction has become a demand-driven market. A reference book seems helpful to guide planners and construction companies, especially those who are about to start their first passive house project.

OBJECTIVES

To create

- a reference for architects and building engineers
- information material for clients
- a reference on ecological criteria for public subsidies for housing (Wohnbauförderungen)
- Information and knowledge transfer for planners and building contractors who are on the threshold to building "ecological passive houses".

General objectives are:

- Reducing costs für ecological building construction: Deficient information forces up prices!
- Increasing quality levels in building construction
- Dispelling misconceptions about costs, comfort and reliability.

The book is structured as shown below

Part 1: Introduction, Methodology, Reference
Part 2: Construction elements
Part 3: Connections
Part 4: Functional units
Part 5: Building materials
Part 6: Glossary, Literature, Index
Part 7: Cost assessments

ON: PART 2: CONSTRUCTION ELEMENTS

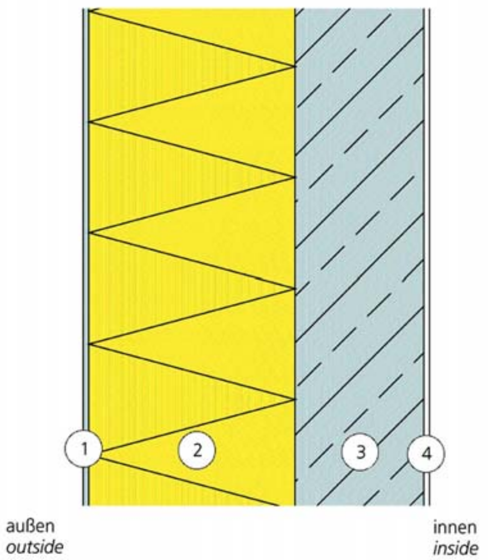


Figure 1. Typical cross sections of an external wall: organic cover coat or silicate-based plaster (1) , 40 cm expanded polystyrene or cork (2), 18 cm reinforced concrete (3), 0,5 cm plaster or clay-based plaster (4).

Typical cross sections are technically described with

- a drawing (see Fig. 1),
- a table specifiying the layers (see caption of Fig. 1),
- a table of physical parameters: heat transition coefficient [W/m2.K], sound reduction index [dB], moisture behaviour [kg/m2.a], effective storage mass [kg/m2],
- a text considering suitability criteria, giving technical advice for construction and maintenance and discussing the functionality of the layers and their proper sequence in the construction.

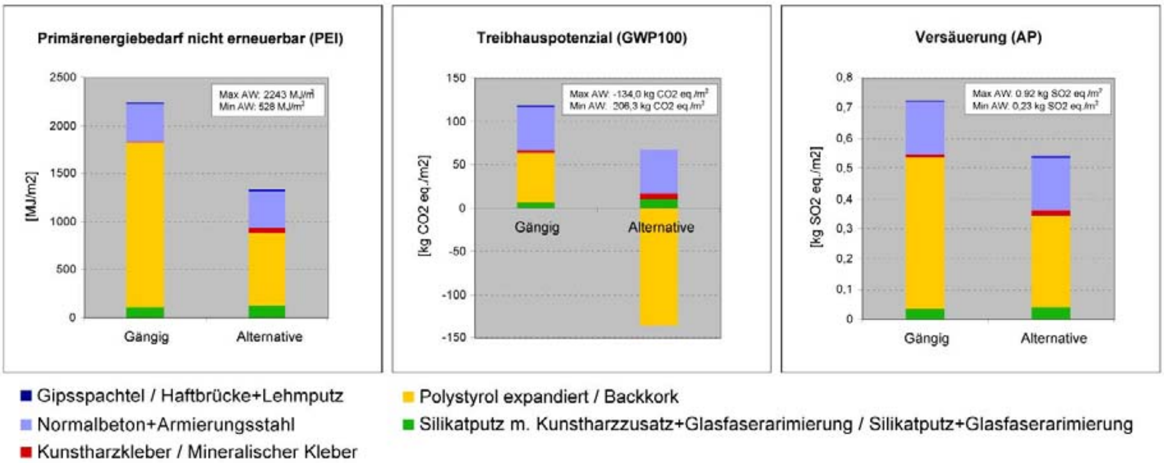


Figure 2. Ecological impact of the production of building materials that make up a square meter of the typical cross section shown in Fig 1.

The ecological evaluation features primary energy input, global warming potential and acidification that has to be accounted to the production of the building element, per square meter and per layer (Fig. 2). Material choices make quite a difference here.

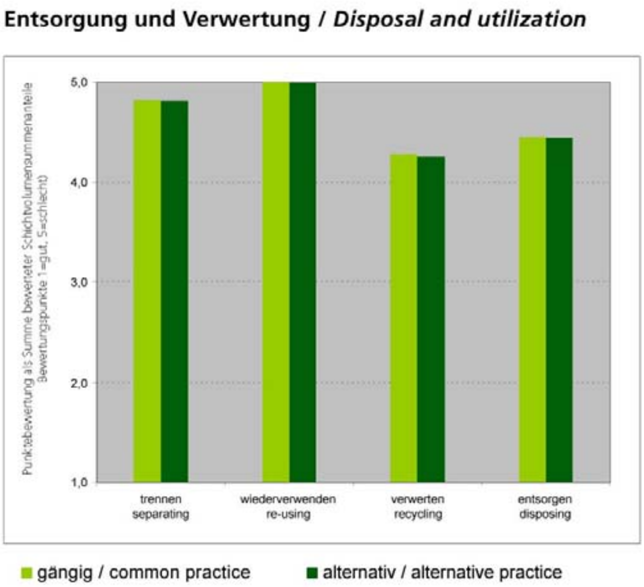


Figure 3. Rating of the potential for separation, re-use, recycling and deposition at the end of the life cycle.

Another chart (Fig.3) rates the potential of a construction at the end of its life cycle: can layers be separated, re-used, recycled or disposed ? Material choices make no difference (in the case shown).

ON PART 3: CONNECTIONS

Connections of building elements in passive houses must be specified with attention on avoiding thermal bridges and on assuring an uninterrupted airtight layer. A detailled drawing (Fig. 4) is supplemented with charts about the spatial distribution of isothermic lines and temperature values along the inner surface of a corner (Fig. 5). Where necessary, also a chart on a dynamic simulation of moisture distribution within the construction will be provided.

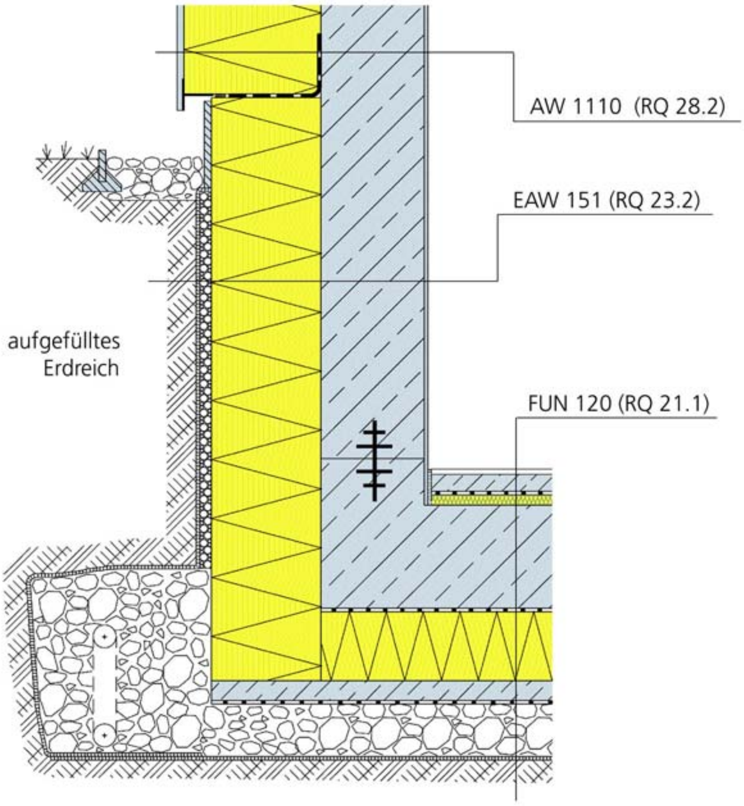
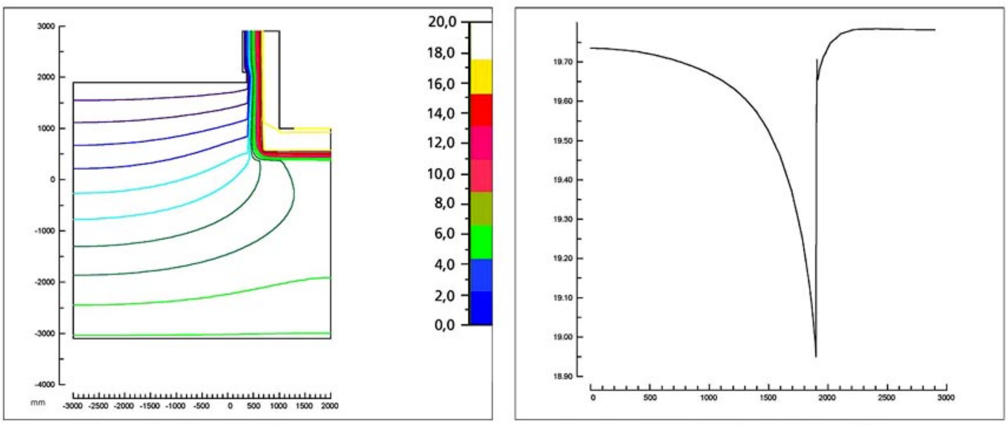


Figure 4. Connection detail. Foundation slab without continuous footing, connected to an external wall (below ground level).

Bauphysik / *Building Physics*

Bauphysik - Baukonstruktion	Einheit	
Linearer Wärmebrückenkoeffizient Ψ	W/mK	- 0,0502



Isothermen (Isothermenabstand entspricht 1°C)

Temperatur entlang der Innenoberfläche, oben beginnend [°C]

Figure 5. Spatial distribution of isothermic lines (left) and temperature values along the inner surface of the corner shown in Fig. 4 (right).

ON PART 4: FUNCTIONAL UNITIES

Functional unities are defined as "connected layers in a building element providing, together, a certain technical service".

Examples are

- composite thermal insulation systems
- insulation between lightweight construction elements
- interior plastering

Functional unities that serve the same function are being compared in the catalogue for their ecological impact of their production and for their recycling potential.

ON PART 5: BUILDING MATERIALS

Building materials are described in detail: ecological life cycle, toxicology (during production, in the construction process, during use, when deposed), supplemented with quantitative data:

- physical values: bulk density [kg/m3], thermal conductivity [W/m.K], vapour diffusion resistance [m], specific thermal capacity c [kJ/kg.K],
- ecological impact per kg mass: global warming potential in (GWP100) [kg CO_2 eq]., acidification AP [kg SO_2 eq.], primary energy input PEI non renewable [MJ].

ON PART 7: COST ASSESSMENTS

Cost assessments are calculated according to the Austrian norm ÖNORM B 2061 for all typical cross sections and will be published separately, on the internet only.

CONCLUSIONS

Allmost all common construction elements can be specified to comply with the Passivhaus standard.
The environmental impact of building material production, that have to be assigned to a given building element can often be diminished considerably by choosing suitable materials.
Also, whether a given construction element can be separated, its parts be used again or recycled, and whether it can be disposed without problems or with difficulties, can be influenced to a remarkable extent by material choice.
The IBO Passivhaus Bauteilkatalog can be expected to become an important reference in the years to come.

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REFERENCES

Feist, W., 1998, Stand der Baupraxis Passivhäuser – Einführung [State of the Art in Passive House Construction – Introduction], in „Das Passivhaus – Baustandard der Zukunft?“ Protokollband Nr. 12, Arbeitskreis kostengünstige Passivhäuser, Ed. W. Feist, Passivhaus Institut Darmstadt, pp. I/1–I/16

IBO (Ed.) 1999, Ökologischer Bauteilkatalog – Bewertete gängige Konstruktionen [Catalogue of ecologically assessed building elements] . Springer, Wien, New York.

Krapmeier, H. & Drössler, E. 2001, CEPHEUS Living Comfort Without Heating, Springer, Wien, New York

Kaufmann, B., Feist, W., John, M., Nagel, M., 2002, Kosten, in holzbau handbuch, Reihe 1, Teil 3, Folge 10 „Das passivhaus – Energie-Effizientes Bauen, DGfH Innovations- und Service GmbH (Ed.), München, p.23

Passivhaus Institut 2004, PHPP 2004, („Passivhaus Projektierung-Paket“, a software package that supports any calculations necessary for planning a building according to the Passsiv Haus standard). Passivhaus Institut, Darmstadt (www.passiv.de)

www.cepheus.at, Information website about Austrian CEPHEUS (= Cost Efficient Passive Houses as EUropean Standards) projects

www.cepheus.de, Information website about non-Austrian CEPHEUS projects (i.e. in France, Germany, Sweden, Switzerland)

Introducing environmental and health criteria when choosing building products

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ABSTRACT

The choice of construction products is essential to building design. Any product chosen for a building has to respect some environmental and health quality, as the third essential requirement of the CPD (Construction Products Directive) notifies it. The choice of building products will also be all the more critical as it will result in a wide compromise among more and more criteria including technical, social, economic, environmental and health ones. Despite the development of construction products standards concerning environmental and health aspects, such as the French NF P01-010, there is no tool that can help building professionals in choosing their products minding environmental and health criteria. That is why such a decision-aid tool is developed in this study. It aims at comparing several sets of building materials and products – building components – by using data from the Life Cycle Assessment (LCA) methodology and the NF P01-010. This tool also includes multi-criteria decision analysis methods.

In this paper, the principles of the existing multi-criteria decision analysis (MCDA) methods are first presented. These principles are then applied in order to perform the framework of the decision-aid tool. All the stages of the presented framework are next detailed: the choice of a component, the determination of the component building options, the assessments of these options, and the comparison of these evaluations. The comparison needs aggregation that demands to select a well-adapted MCDA method. The way of choosing a decision-aid method relevant to the input data – the Life Cycle Inventory (LCI) results of a set of construction products – and some constraints – risk of compensatory results – is finally explained.
In conclusion, the developed decision-aid tool will allow building actors to take into account Sustainable Development by introducing environmental and health quality in their choice of building systems.

KEYWORDS

Sustainable Development, Environment, Building products and materials, Building components, multi-criteria decision aid methods.

1 INTRODUCTION

Nowadays buildings have to integrate more and more principles of Sustainable Development in all stages of their life cycle. In particular, they have to reduce their consumption of raw materials and energy, and to limit their emissions to the environment. Some of these impacts are related to the materials and products that buildings are made of. The choice of building materials and products is consequently important. This choice has now to take into account environmental and health characteristics of these materials and products in addition to their technical and economical features. There is no tool that can help building actors to make such an integrated choice at this time.

Some recommendations about the environmental and health minimum characteristics of the building products are developed in the European Construction Products Directive (CPD), in particular in the third essential requirement. The Integrated Product Policy (IPP) also deals with the environmental and health importance in the design and the choice of all the products, and consequently all the building products. But these recommendations dwell rather on the importance of the choice than on the way of making this choice.

The environmental and health impacts of building products are more considered in the French standard NF P01-010. This standard provides a common format – the French EPD (Environmental Product Declaration) format – to declare the environmental and health characteristics of building products. Consequently this French standard allows building products manufacturers to provide environmental and health characteristics of their products. Nevertheless these characteristics are numerous, and sometimes redundant. As a consequence, building products may neither be correctly compared, nor easily chosen according to their environmental and health characteristics. Moreover the link between environmental and health building impacts related to building products is not clearly feasible. That is the purpose of this study. It consists in introducing environmental and health characteristics of the components – sets of building materials and products that have a particular functional unit – in a decision-aid tool to choose them.

This decision-aid tool is based on the principles of Multi-Criteria Decision Analysis (MCDA) methods and partly uses the Life Cycle Assessment methodology. It evaluates each action – several building options of a building component – against each criterion – environmental and health impacts – and offers a way of aggregating the results.

2 FRAMEWORK OF THE DECISION-AID TOOL

The objective of the decision-aid tool will first be presented. The principles of the MCDA methods will then be described. Finally the main guidelines of the decision-aid tool will be provided.

2.1 Objective of the decision-aid tool

The objective of the tool is to help building actors to compare and then to choose the set of building products for a component that will best suit their environmental and health priorities. To compare different building options of a building component regarding their environmental and health characteristics, the principles of MCDA methods are used, as said before.

2.2 Principles of the multi-criteria decision analysis methods

The objective of multi-criteria decision analysis methods is to provide an overview of a complex issue that a decision-maker has to 'solve'. The MCDA methods are usually composed of five main parts. Firstly, the MCDA methods consist in identifying the set of alternatives or options foreseen to solve the problem. Then it is necessary to determine a set of objectives and the associated set of criteria that will assess the achievement of the objectives for each alternative. The set of criteria has to be coherent [Roy 1985], [Roy, 1993]. Criteria have indeed to be carefully chosen.

The MCDA methods go on by building a performance matrix [table 1], in which each row describes an alternative (i) and each column describes the performance, or the score (C_{ij}), of the alternatives against each criterion (j). Then weights may be assigned to each criterion to reflect their relative importance to the decision.

	Criterion 1	Criterion 2	Criterion 3	Criterion 4	Criterion 5
Alternative 1	C_{11}	C_{12}	C_{13}	C_{14}	C_{15}
Alternative 2	C_{21}	C_{22}	C_{23}	C_{24}	C_{25}
Alternative 3	C_{31}	C_{32}	C_{33}	C_{34}	C_{35}
Alternative 4	C_{41}	C_{42}	C_{43}	C_{44}	C_{45}

Table 1. Description of a performance matrix of a MCDA method

These considerations are common to all MCDA methods. MCDA procedures are indeed distinguished from each other principally in terms of how they process the basic information in the performance matrix. There are three kinds of processing: complete aggregation, partial aggregation, and local and iterative aggregation. The MCDA methods end by the analysis of the obtained results and some recommendations are done.

2.3 Guidelines of the decision-aid tool

The decision-aid tool is based on the MCDA methodology. The MCDA methodology is indeed one of the best methodologies that offer a flexible and effective way to help decision-makers [Pictet 1996].

The tool is composed of five main parts. The first one is the choice of the component that the user wants to study and the definition of the functional unit of this component. The functional unit integrates the element, its function and its lifetime. The second part then consists in determining the different component building options: the alternatives. The third part consists in studying all the building options by assessing their environmental and health characteristics: the performance matrix. This part needs the previous construction of the set of the environmental and health criteria: the coherent set of criteria. The fourth part offers the way to compare the component variants, by using a set of weighting – that represents the users environmental and health priorities – and an aggregation method (a MCDA method). The last part allows the user to choose the building option of the component that fits his environmental and health priorities, by taking into account the analysis of the results and the recommendations of the tool. The user may then choose another component to study. Figure 1 presents the framework of the decision-aid tool.

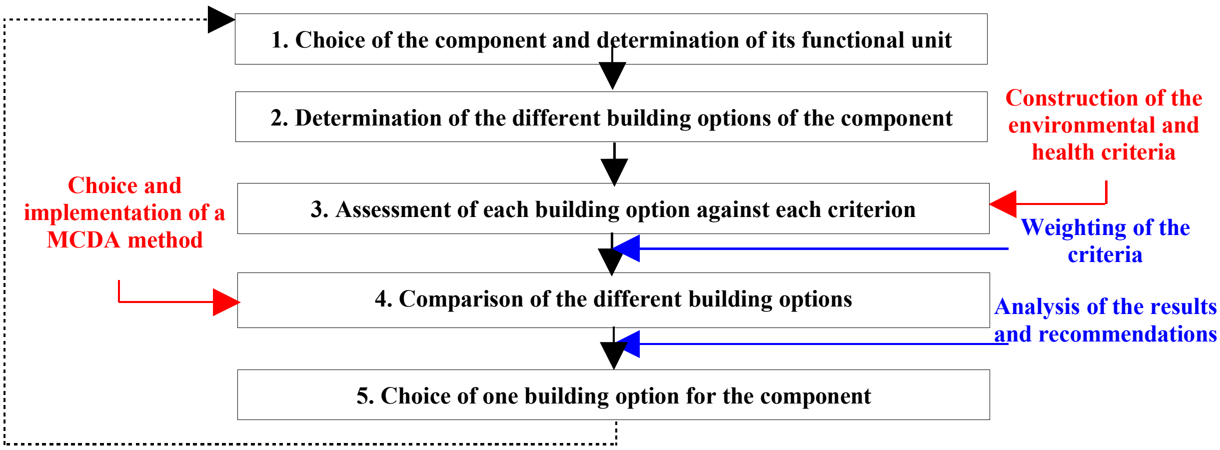

Figure 1. Framework of the developed decision-aid tool that represents all the stages and all the inputs/outputs of the tool.

The developed tool may use external data from INIES – the French reference database of building products EPD's [INIES 2004] – to choose the components and determine their building options for example.

It will use internal data such as the coherent set of criteria, the simplified set of criteria, sets of weighting, and algorithm calculations of the chosen MCDA methods.

The objective of the developed tool is to provide the building actors with a decision aid so that they can choose their building components by taking into account environment and health. The output data of the tool may then be useful for design tools of the buildings or environmental and health evaluation tools of the buildings.

3 METHODOLOGY OF THE DECISION-AID TOOL

The framework of the decision-aid tool is now determined. Each part of the methodology will then be detailed. Firstly, it is essential to determine the different component building options to compare. Secondly, the environmental and health criteria allowing the selection of the different options have to be built. Environmental and health assessments of these options against the criteria are then necessary. Aggregating these evaluations is finally recommended in order to obtain clear and simple results. The choice of the well-adapted MCDA method is then necessary to combine the evaluations of the component building options.

3.1 Determination of the component building options

The developed tool aims at comparing different building options of a same building component. A component is a set of building materials and products that has one particular functional unit. This unit may be performed with different sets of materials and products. It is necessary to clearly define all the building options of a component (the potential alternatives) in order to make possible their comparison. The data of INIES may be used to define the components and their potential building variants.

Each variant is made of several building materials and products that may have different lifetimes. The quantities of all these elements also have to be modified in order to be in accordance with the functional unit of the component. The building materials and products will be provided with their Life Cycle Inventory (LCI). LCI is the second stage of Life Cycle Assessment (LCA) and is available either in INIES or in the environmental and health declaration of the building product.

3.2 Determination of the coherent set of environmental and health criteria

The coherent set of criteria has to be built in order to allow the use of MCDA methods to aggregate the evaluations of the component building options. The construction of the environmental and health criteria was based on the NF P01-010 environmental impact categories. But these impact categories are not coherent criteria in the meaning of MCDA methodology. These impact categories were also modified and completed by the listing of the CML-SETAC [CML 2002], ATEQUE [Chatagnon 1999] and environmental and health assessment tools of buildings [GBTool 2004], [EQUER 2004]. The following tables 2 and 3 respectively represent the impact categories defined in the NF P01-010 and the criteria that will be used in the decision-aid tool.

Each criterion is provided with a name, a scale, an evaluation indicator and energy and material flows from the LCI. The construction of each criterion takes into account the recommendations of the NF P01-010.

The developed coherent set of criteria was simplified in order to provide an easier understanding of the environmental and health profile, as well as facilitate the weighting of the criteria. Criteria were also gathered into more global criteria by possibly using internal weightings. The simplified set of criteria is made of six global criteria that are presented in table 4: consumption of resources, waste production, stock production, global long-term environmental impact, local mid-term environmental impact, health risk of pollutants' emissions.

Environmental impact categories	Energy resources consumption
	Depletion resources indicator
	Water consumption
	Solid Waste
	Climate change
	Atmospheric acidification
	Stratospheric ozone depletion
	Air pollution
	Water pollution
	Photo-oxidant formation
Health impact categories	There are no health impact categories but recommendations are presented.

Table 2. Environmental and health impact categories of the NF P01-010

Environmental criteria	Energy resources consumption
	Non-energy resources consumption
	Water consumption
	Hazardous waste production
	Non-hazardous waste production
	Inert waste production
	Radioactive waste production
	Material stock production
	Energy stock production
	Climate change
	Atmospheric acidification
	Stratospheric ozone depletion
	Air pollution
	Water pollution
	Soils pollution
	Photo-oxidant formation
Health criteria	Chemical pollutants' emissions
	Physical pollutants' emissions
	Biological pollutants' emissions

Table 3. Environmental and health criteria of the developed decision-aid tool

	Complete set of criteria	Simplified set of criteria
Environmental criteria	Energy resources consumption	Consumption of resources
	Non-energy resources consumption	
	Water consumption	
	Hazardous waste production	Waste production
	Non-hazardous waste production	
	Inert waste production	
	Radioactive waste production	
	Material stock production	Stock production
	Energy stock production	
	Climate change	Global long term environmental impact
	Atmospheric acidification	
	Stratospheric ozone depletion	
	Air pollution	Local mid-term environmental impact
	Water pollution	
	Soils pollution	
	Photo-oxidant formation	
Health criteria	Chemical pollutants' emissions	Health risk of pollutants' emissions
	Physical pollutants' emissions	
	Biological pollutants' emissions	

Table 4. Presentation of the simplified set of criteria of the decision-aid tool.

3.3 Assessments of the component building options

The evaluation of each component building option is based on the LCI of the building materials and products that the option is made of. All the LCI flows of a material or a product are converted into evaluation indicators of all the criteria for the functional unit of the component. This stage needs to carefully define the limits of the LCI of the building materials and products. All the materials and products have to be counted once and once only. All the flows are then transformed into environmental and health evaluations by using the criteria evaluation indicators. A complete environmental and health profile (per functional unit of the component) is also obtained for each material or product of the component building option. This profile is then simplified by using the simplified set of criteria.

All the simplified profiles of the materials and products are finally combined by a weighted sum in order to obtain the evaluation of the component building option. These operations are repeated for each component building variant.

All the component building options are now provided with their environmental and health assessments. These evaluations may also be aggregated in order to compare the component building variants. This aggregation requires the selection of an aggregation method well-adapted to the input data and the constraints of the tool. This point will be detailed in the part 4.

3.4 Choice of one building option of the component

The aggregation of the assessments of the component building options allows users comparing these options and choosing the building variant that fits their environmental and health priorities. An analysis of the results – by changing parameters such as weights, or the aggregation method – and some recommendations will be introduced in the developed tool in order to help users to make their choice.

4 SELECTION OF A MULTI-CRITERIA DECISION ANALYSIS METHOD

First of all, the main differences between MCDA methods will be analysed in order to present a practical decision tree for these methods. This decision tree will finally allow an easier selection of the well-adapted method according to the input data and constraints.

4.1 Distinctive elements of the MCDA methods

In addition to their processes of aggregation, the MCDA methods have not the same objectives. They do not give the same kinds of results nor do they use the same inputs and hypotheses. Moreover, they do not all use weights, and the features of the criteria may also be different.

4.1.1 Aggregation

As seen before, there are three processes to combine the scores of the alternatives on the criteria: the complete aggregation, the partial aggregation and the local and iterative aggregation. Only the two first kinds of aggregation will be considered in the following study. The complete aggregation combines the scores of the alternatives into a global score by usually using sum and average. Then it compares the global score of each alternative to deduce a ranking of the alternatives. Such an aggregation is compensatory [Schärlig 1985], [Maystre *et al.* 1994]: an alternative with a good score on a very-weighted criterion may be among the first even if all its other scores are bad. This kind of aggregation is used by some environmental assessment tools, such as BREEAM for the buildings [Anderson & Shiers 2002], or BEES for the products [BEES 2004]. The partial aggregation compares each score of each alternative to each other. Then it ranks the alternatives two by two before deducing a global result. This kind of aggregation is often considered as a non-compensatory method.

4.1.2 Objectives

Roy [1985] sorts out the objectives of the MCDA into three categories. The first one allows selecting the best alternatives or options as regards their scores on all the criteria. The second one allows sorting the different alternatives in predetermined categories. And the third one allows ranking the different alternatives in preference order as regards their scores on all the criteria. The MCDA methods have often been built in order to answer one particular objective. But they may be modified in order to answer some different objectives.

4.1.3 Inputs

All the MCDA methods do not treat the same inputs, and particularly the feature of the scores and the nature of the criteria. The scores of the alternatives on each criterion may be notes, values, or qualitative judgements. Several MCDA methods preferentially use one or another kind of scores. For the complete aggregation methods, the criteria have to be given without thresholds: they are considered as “true”. So there are no various preferences between the alternatives. The partial aggregation methods may use threshold criteria. The nature of the criteria may also influence the choice of the MCDA method.

4.1.4 Nature of the MCDA method and weights

Some of the MCDA methods are ordinal: they only use the ranking of the alternatives on each criterion to deduce their global ranking. The other MCDA methods are cardinal and combine the scores of the alternatives. Several MCDA methods do not use weights to combine the scores of the alternatives. Consequently the difficulty to weight the criteria may be got round by choosing MCDA methods that do not need weights.

4.2 Practical decision tree for MCDA methods

Before using any MCDA method to solve a problem, the dilemma is to choose the more appropriate MCDA method. That is the purpose of the development of a practical decision tree for MCDA methods (Fig. 2). It aims at allowing an easier choice of MCDA methods by taking into account some of their main features. These main features have just been described. The tree was built by studying the progress of the methods.

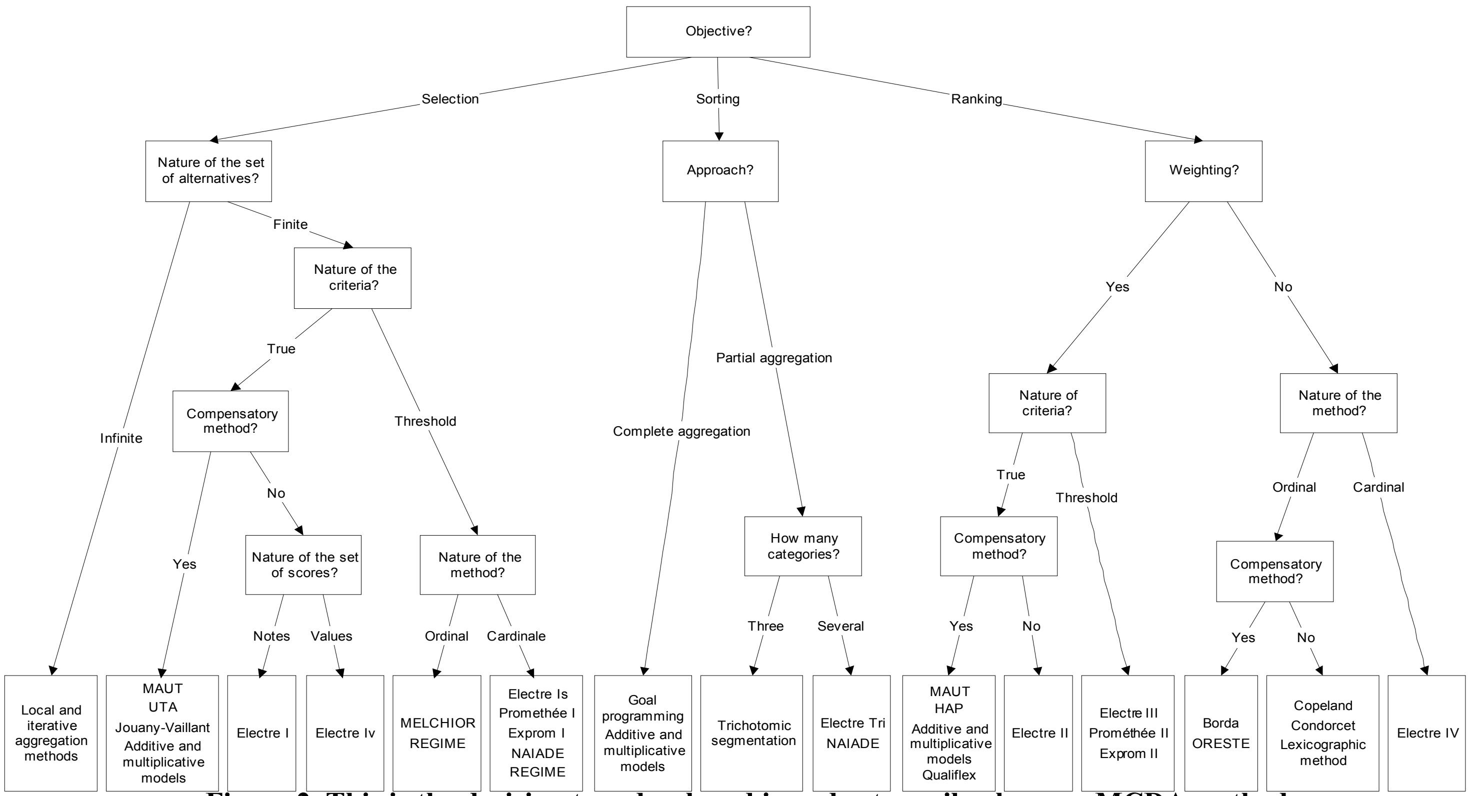

Figure 2. This is the decision tree developed in order to easily choose a MCDA method.

4.3 Comments on the decision tree

The decision tree was built so as to limit redundant questions on the same initial branch. Moreover, the maximum differentiation between the methods was looked for. Some changes may be done in order to obtain only one method at the end of each branch. At the moment, there is a lack of information on several methods. For example, the compensatory characteristic of ELECTRE III, PROMETHEE II or EXPROM II is not known in the decision tree. Nevertheless, this present decision tree offers a practical and swift way to choose between numerous methods.

4.4 Choice of a well-adapted MCDA method and aggregation of the assessments

The objective of ranking is chosen in order to offer a decision-aid and not a proper decision. Weighting have been asked by the professional users of the developed tool. Several sets of weighting will also be available in the tool. The input data of the developed tool is the LCI of the building products. Then the simplified environmental and health profiles of each component building option may be demanded. All this data is numeric or may be transformed to be numeric. Moreover values are exact. The criteria are all true: their assessments are exact. However they may be changed in the future improvements of the tool. So the criteria will be considered as threshold ones. Finally the data deals with the environment and the health. Consequently non-compensatory methods are preferred to compensatory ones.

All these elements allow the selection of MCDA methods such as ELECTRE III, PROMETHEE II or EXPROM II. Nevertheless, it may be interesting to offer the possibility of comparing results given by two kinds of methods. The developed tool will consequently include both a complete aggregation method – additive model – and a partial aggregation method – ELECTRE III.

5 CONCLUSIONS AND PERSPECTIVES

Environmental and health characteristics of the building materials and products are insufficiently taken into account when these elements are chosen for building design. However lots of environmental and health impacts of buildings are related to their own materials and products. This study also aims at compensating this lack by developing a decision-aid tool to choose building materials and products according to their environmental and health characteristics. This study has allowed the achievement of the framework of the decision-aid tool. All the elements are now defined: the input data, the output data, and the internal databases of the tool. The developed tool uses the principles of MCDA methods in all its stages, and particularly for the aggregation of the assessments of the component building options. The methods of aggregation have been chosen thanks to a decision tree for MCDA methods. This decision tree was developed in order to make easier and more practical the choice of a MCDA method, according to some input data and constraints.

This study will go on with the computer development of the decision-aid tool and its validation by processing several applications. The following components will be studied with the developed tool: walls, windows, soils surfaces, and girders. This computer development will finally allow future improvements of the tool. The family of criteria may be extended to technical, economic and comfort criteria. The uncertainty of the input data may be taken into account.

6 REFERENCES

Anderson, J. & Shiers, D. 2002, *The Green Guide to Specification*, Blackwell Science, Oxford.
BEES, 2004, http://www.bfrl.nist.gov/oae/software/bees_USDA.html
Chatagnon, N. 1999, *Développement d'une méthode d'évaluation de la qualité environnementale des bâtiments au stade de la conception*, Thèse de Doctorat, Université de Savoie.
EQUER, 2004, http://www.izuba.fr/equer.htm
GBTool, 2004, http://greenbuilding.ca/gbc2k/gbc-start.htm
INIES, 2004, http://inies.cstb.fr
Leiden University (CML), 2002, *Handbook on Life Cycle Assessment, Operational Guide to the ISO Standards*, Kuwer Academic Publishers, Dordrecht.
Maystre, L., Pictet, J. & Simos, J. 1994, *Méthodes multicritères ELECTRE*, Presses polytechniques et universitaires romandes, Lausanne.
Norme AFNOR NF P01-010, 2004, Qualité Environnementale des produits de construction, *Déclaration des caractéristiques environnementales et sanitaires des produits de construction et de leur contribution aux impacts environnementaux d'un ouvrage donné*.
Pictet, J. 1996, *Dépasser l'évaluation environnementale, Procédure d'étude et insertion dans la décision globale*, Presses polytechniques et universitaires romandes, Lausanne.
Roy, B. 1985, *Méthodologie Multicritère d'Aide à la Décision*, Economica, Paris.
Roy, B. & Bouyssou, D. 1993, *Aide Multicritère à la Décision : Méthodes et Cas*, Economica, Paris.
Schärlig, A.1985, *Décider sur plusieurs critères, Panorama de l'aide à la décision multicritère*, Presses polytechniques et universitaires romandes, Lausanne.

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Environmental Aspects of Concrete Structures in Sustainable Society

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ABSTRACT

In this study, “the environmental ethics”, “the environmental consciousness among stakeholder” and “the design strategy considered the former contents”, which are necessary for developing sustainable society is to be illustrated. The mainly results are as follows.

1) Sustainable society would be a fundamental concept of sustainable development, and in construction fields it would be necessary for considering as essential principles.

2) The environmental aspects for concrete structures are largely explained by the three-steps concepts as follows. The first is the regional and the earth-scale natural capitals such as water circulations, air circulations and material circulations. The second is the social systems categorized by technical factors, political factors and legal factors. The third is the substantial needs of individual and society based on the Masrow psychological theory regarding human needs.

3) The environmental aspects hierarchy would be useful as a material for clarifying "Projection Initiation" indicated as a diagram of technical, economic and environmental assessment in SLP and the location of project planning in the life cycle of the building/constructed asset in ISO/DIS 15686-6's.

4) In order to clarifying and developing the environmental aspects of concrete structures in sustainable society, it would be effectiveness to introduce the design strategy constituted by 8 kinds of fundamental functions and the 4 kinds of design objects for concrete structures.

KEYWORDS

Sustainability, Sustainable Society, Environmental Aspect, Social Needs, Real Function, Concrete Structure, Recycle Concrete

1. INTRODUCTION

Providing excellent performances as a structural material, concrete has long been deemed essential for modern civilization and recognized as a material that will continue to maintain and support the development of human society. It is now being seen in a new light, as recycling of concrete in a completely closed loop has become technically feasible.

In view of considerations for the global environment to be required in the future at every step of the production of concrete and concrete structures, this paper then overviews the method of identifying social needs for concrete structures, the way the production systems of structures should meet such social needs, lifecycle design techniques for structures, and techniques for expressing the environmental performance of structures with the background reasoning, as well as the importance of such techniques.

The authors finally discuss what true recycling and truly recycling-oriented society are, based on the above-mentioned discussions.

2. ENVIRONMENTAL ASPECT OF CONCRETE STRUCTURE PRODUCTION

2.1 Introduction of environmental aspect for concrete structures

Environmental aspect is defined in ISO 14050 (Environmental management – Vocabulary) as follows:

> "Element of an organization's (1.4) activities, products or services that can interact with the environment (1.1)"
>
> *1.1 environment
> Surroundings in which an organization (1.4) operates, including air, water, land, natural resources, flora, fauna, humans, and their interrelation
>
> *1.4 organization
> Company, corporation, firm, enterprise, authority or institution, or part or combination thereof, whether incorporated or not, public or private, that has its own functions and administration

Figure 1 shows the classification and targets of sustainability considerate of the global environmental aspects. According to the above definition, the "environment" is a concept to be recognized as a factor covering a wide range. As the "environmental aspect" is composed of wider concepts, possible events to be considered can increase in the future. It should be noted that the construction industry, which is responsible for the activities using products made using recycled aggregate and recycled aggregate concrete, is by nature prone to be involved in social and environmental events through industrial activities due to the properties of the materials and mass circulation systems the industry uses. In other words, extensive social responsibilities may be appointed for the production of finished articles and raw produce. Stakeholders may therefore be required to investigate the market expansion while being always conscious of the boundaries of the "environmental aspects." Such practice will ultimately lead to risk management for producing concrete structures.

Meanwhile, an essential concept for considering social activities deeply related to the environment is "sustainable development." The first comprehensive proposal was made in "Our Common Future" published in 1987, in which the following principles were included.

1. Social equity (social aspect)
2. Environmental prudent (environmental aspect)
3. Economic efficiency (economic aspect)

The environmental aspect is regarded as being composed of a social aspect, environmental aspect, and economical aspect. It suggests that it will become important for production activities to consciously consider the social aspect and environmental aspect in addition to the conventional economic aspect. It means that any development can be inappropriate for sustainability unless it is based on the three principles to ensure sustainable development.

As stated above, conventional practice of production was not sufficient in terms of the holistic strategy, specifically, the application of manufacturing strategy in consideration of the social and environmental aspects in addition to the economic aspect.

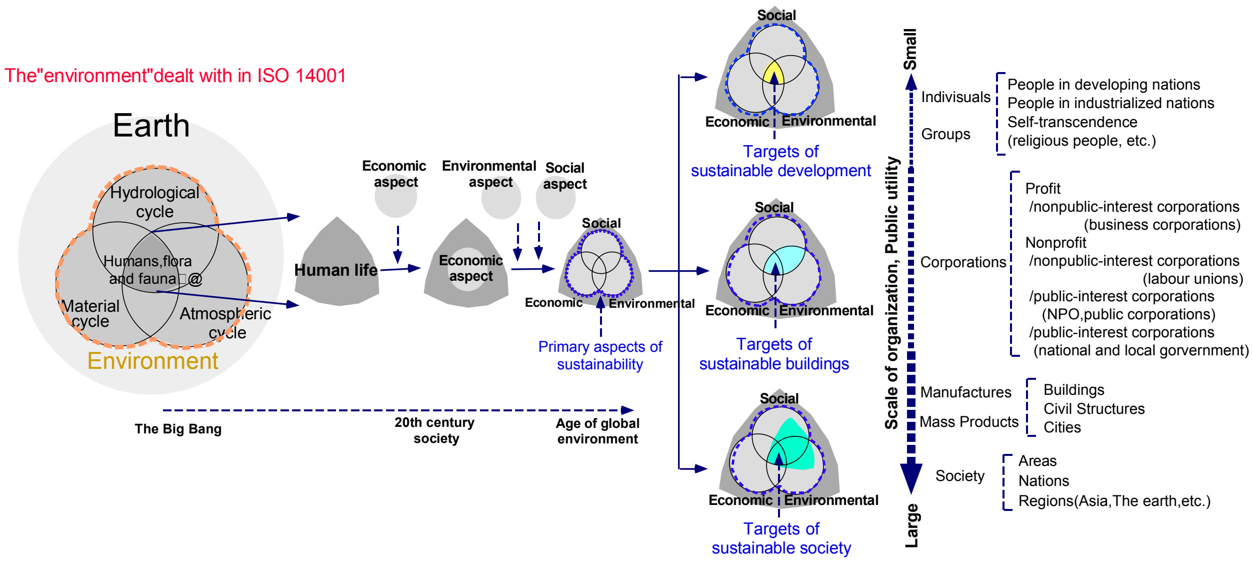

Fig. 1 Classification and objects of sustainability with consideration to global environmental aspect

2.2 Grounds for identifying social needs for concrete structures

From a broad perspective, even amid the era of the global environment, the world as a whole makes its way under the banner of the human-oriented goal of leading a sustainable social life premised on the development of economic activities. It is predicated on a theory that a system of distributing goods produced from economic activities to the market and benefiting from them is necessary for satisfying (or stabilizing) people, in order to realize the affluent social life ahead. (It has been made clear by past discussions that this cannot be a universal and optimum form of human activity considerate to the global environment.) Systems for such “stability” and “affluence” become more complicated and hard to accomplish as the range expands from the levels of individuals to organizations, regions, nations, and to the earth. It is now vital to recognize that we are, fortunately or unfortunately, at the ultimate stage of active efforts worldwide aiming for “global stability,” including measures against global warming committed in the Kyoto Protocol. For explanation, Fig. 6 shows the order of dependence for the conditions of stability of a global system. This figure does not mean a hierarchical relationship but shows an order of dependence in which the upper factor is realized depending on the support from the lower factor. Stabilization on the individual and organizational levels, for instance, is important for social stabilization in developing countries, whereas it is necessary for developed countries, for whom environmental consideration is mandated by ratified treaties, to give consideration to stabilization on the regional and global level rather than on the individual and organizational levels.

Stabilization on the global level can be achieved by maintaining the three cycles, i.e., the hydrological cycle, atmospheric cycle, and material cycle, to hold without damaging the self-cleaning action of the earth as inferred from ISO 14000 series. It is self-explanatory that stability on the global level as a superordinate concept is essential for ensuring stable conditions at a smaller region as a subordinate concept on a long-term basis while maintaining equitability between generations.
To put it plainly, an essential condition required of concrete structures for the stabilization of the earth is that a system for assuring the three cycles, i.e., the hydrological, atmospheric, and material cycles, should be inherent in such structures. Focusing on the material cycle, which is closely related to the act of production, the condition is consolidated into a commitment to ensure resource circulation of the structure and its materials beforehand. As recycled aggregate concrete is a factor that strongly affects the material cycle of structures, it is important to extend the radius of influence from individual consumers to organizations/groups and to districts/regions to give shape to the ways of resource circulation of structures and materials and to capture related needs.

2.3 Factors of social systems composing environmental aspects

The environmental aspects of concrete structures are embodied by focusing on their social and environmental aspects in addition to conventional economic aspects with respect to various types of information (international treaties, laws, rules, policies, and guidelines) that serve as factors for extracting the environmental aspects.

Figure 2 shows the order of dependence in the environmental aspects in the wide sense of the word. Though this figure classifies the environmental aspects into three factors (needs of individuals/society formative factors of social systems, and formative factors of global systems), this chapter deals with the “formative factors of social systems” with which environmental aspects can be clearly explained. Note that the “global system” composing the bottom consists of the three cycles of the earth and regional factors, but the environmental aspects may widely vary depending on the country and area. The radius of influence of the same environmental aspect can widely vary depending on the effect of factors on the regional level. It can therefore be said that the contents and degrees of the needs of individuals and society based on such global and social systems can also vary from one society to another.

The formative factors of social systems are roughly classified into three categories. The first consists of “legal factors” primarily comprising international charters/treaties, domestic laws (fundamental laws and normal laws), and local regulations. In a law-abiding society, the boundaries of acceptable social/economic activities and industrial activities are defined by these legal factors.

The second covers “policy factors” that are deeply involved in the formation of actual social order. Policy factors, which form a concept smaller than that formed by legal factors, comprise national policies/measures, principles/strategies, and guidelines.

Since these factors directly affect the boundaries of activities, they concretely shape the way the society functions.

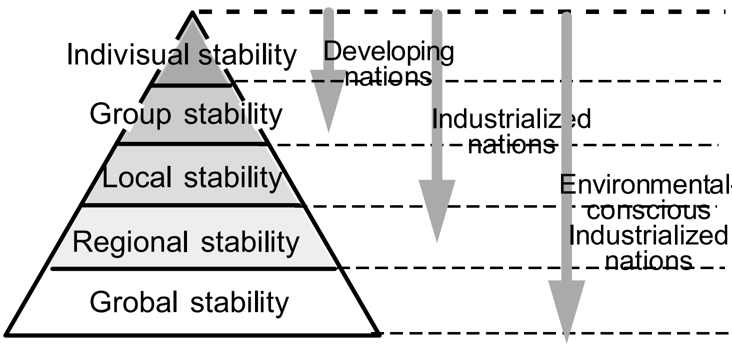

Fig. 2 Order of dependence for stability conditions on a global scale

The third comprises “technical factors” for conducting social activities safely and rationally under the influence of the legal and policy factors. Technical factors primarily comprise specifications, methods, and techniques, whose contents and spread steer the social trends. Note that information on the needs of individuals and society can be fed back to design techniques for structures through technical factors (e.g., architectural design briefing). These permit investigation in specific terms into the needs of individuals and society derived from their inherent desires.

2.4 Formative factors of individual and social needs

The factors forming the needs of individuals and society are considered to be the inherent desires of individuals and organizations. It is therefore desirable to extract various needs from the inherent desires of individuals and society to identify the needs for structures to be ultimately reflected to the essential functions of the structures. Figure 3 shows the order of dependence of inherent desires of individuals and society affecting the essential functions of structures.

Inherent desire of individuals can be objectively grasped by Maslow's theory of motivation and human needs and the concept of the need for self-transcendense to change oneself into something beyond the present state. Maslow's theory of motivation and human needs consists of levels of needs for physical safety and security, social safety and security, communication and response, self respect and acceptance, and fulfillment of goals and dreams, in which the fulfillment of the lower level is essential for fulfilling the upper level.

It is not easy to grasp inherent desires of organizations and society, but the presence of such desires is understandable. Their needs can be clarified to a certain extent by research. For instance, when an enterprise, a typical social group, is explained as "integrating production factors and continuously operating a business with the aim of production and profit making□or the subject of such activities," the fundamental desire of enterprises corresponding to the needs on the bottom level by Maslow (physical needs of enterprises) may be the desire for "conducting a business operation to be economically viable without ceasing to produce profits," and this is considered to be an essential desire of enterprises. Enterprises are therefore understood as groups that seek for their identities driven by the bottom needs by Maslow (physioligical needs of enterprises). In this case, the chances for their activities to fulfill the desire on the upper levels are considered very low. This explains the difference between individuals and organizations from the aspect of needs.

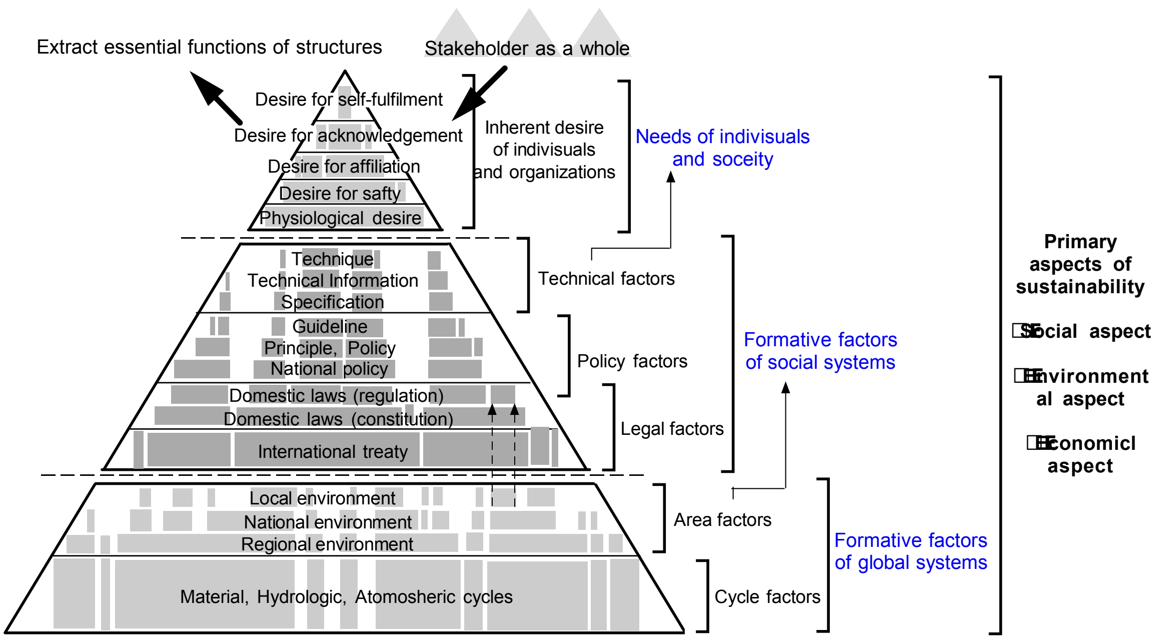

Fig. 3 Order of dependence for environmental aspects in a broad sense

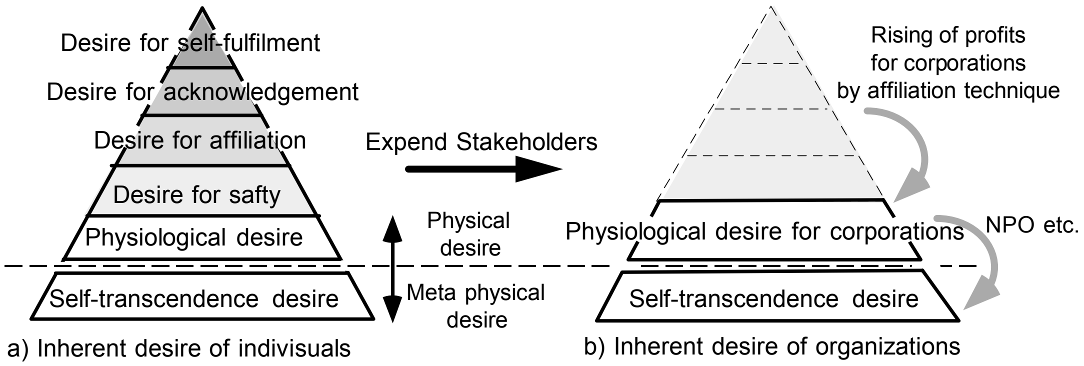

Fig. 4 Inherent desires of individuals and society affecting the essential functioning of structures

A large number of firms recently carry out their industrial activities while promoting environment protection. These can be recognized as activities for fulfilling the "desire for self-respect and acceptance," which complements the "physiological desire of enterprises." On the other hand, the activities by environmental nonprofit organizations can be recognized as those for fulfilling their "desire for self-transcendence" placing top priority on the global environment. Though the desires of organizations and societies differ from the order of dependence of individual desires as explained above, organizational activities are important, as they produce large-scale and immediate effects on actual activities for solving environmental problems.

The production of structures generally has strong impacts on society. Moreover, interests in structures include individuals as well as their relationships with society. It is therefore important to simultaneously extract the needs of individuals and society and fulfilling them in the form of essential functions of structures.

3. TOWARD A VERITABLE RECYCLE-ORIENTED SOCIETY

Promotion of recycling is essential for establishing a recycling-oriented society. However, it is difficult for low-quality products to achieve marketability simply because they are recycled.

The authors consider that recycling technology should fulfill the following principles.

1. Recycling should be of high quality.
2. Recycling should be repeatable.

The former means that recycled products cannot be marketable unless they satisfy users with their quality, or recycling cannot hold in spite of the fair name of "recycled product." Low quality of recycled products should be regarded as indicating immaturity of the recycling technology, demanding technology improvement or new technology development for achieving quality recycled products.

The latter means that, if a recycled product has to be dumped in a landfill after use with no chance of recycling, then the recycling is no better than producing waste of the following generation, contradicting the formulation of a recycling-oriented society. Such a product also burdens the purchaser with the responsibility of waste disposal. Potential purchasers hesitate to purchase the product the moment they realize the pitfall, inhibiting the recycling loop to close (the rule of Old Maid). Accordingly, recycling should aim for reproduction of the same product (to be repeatable) as the original sense of the word to form a loop.

Care should be exercised for so-called cascade recycling, mixed/compound recycling, recycling into other industries, and byproduct utilization, as these tend to be unrepeatable. When a product utilizing a byproduct is repeatedly recycled, the byproduct as a material will become useless, ending up as waste.

What is a truly recycling-oriented society? Humans live on a cycle of resource use in which they take various resources from nature into their society, obtain benefits from them for a certain period by processing and using them, and return them back to nature as waste when their utility runs down. Once taken into the human side, the resources are altered or modified to a certain extent by the time they are returned to nature for final disposal. One or two recycling phases are also a short stay of resources in human society after all, which ends up with disposal (return) to nature. True recycling should eliminate such return of resources to nature, but unrepeatable recycling merely extends their stay on the human side without contributing to the basis for truly recycling-oriented society. The only exception of this type of cycle is the use of resources into a form acceptable for nature.

Recycling-oriented society in the true sense of the word is a society that continues to use resources, once they are taken from nature into the society, without returning them unless they are acceptable for nature. In such a society, the intake of resources from nature will be a minimum required, and products and materials that cannot be recycled repeatedly are rejected. Though this may be too idealistic, the idea should be kept in mind when evaluating and developing recycling technology.

4. CONCLUDING REMARKS

Through the research and development for concrete recycling and investigation of the difficulties and problems of recycling practice for a long time, the authors learned that the above-mentioned two principles are vital for recycling to hold. This approach to recycling has long been practiced for electric steel, aluminum, and paper and has been adopted for various types of products including PET bottles recycled into PET bottles, the reuse and recycling of materials for automobiles and electrical appliances. Efforts for closed-loop recycling have also begun in the fields of glass, gypsum board, and other construction materials. This trend will prevail in all fields in the future, eventually leading to a society in which materials recyclable in closed loops or disposable in a nature-friendly form expel those that are not, as in the case of chlorofluorocarbons driven out of the market.

In this sense, the greatest challenge for mankind is the formulation of a recycling system for carbon dioxide gas and other greenhouse gases, which account for the vast majority of waste on earth.

5. REFERENCES

A. H. Maslow (1987). “Human psychology”, Sanno publication (in Japanese).

Global Responsible Initiative(2000). “Sustainability Reporting Guideline” .

Kimura F. (1999). “Life Cycle Design for Inverse Manufacturing”, Proceedings of Eco Design’99. IEEE, 995-999.

Noguchi, T. and Tamura, M. (2001). “Concrete Design toward Complete Recycling”, Journal of the Fib Structural Concrete, Vol.2, Number3, 155-167.

Pietersen, H.G. and Hendriks, C.F. (1999). “Toward Large Scale Application of Recycled Aggregate in the Construction Industry, European Research and Developments”, Proceedings of the Fourth International Conference on Eco materials, Gifu, 217-221.

Tamura, M., Noguchi, T. and Tomosawa, F. (1997). “Development of Recycle-oriented-concrete with easy to collect aggregate”, The 51st Annual meeting of Japan cement association, 218-219. (in Japanese)

Tamura, M., Noguchi, T. and Tomosawa, F.(2002). “Life Cycle Design based on Complete Recycling of Concrete”, The First fib Congress 2002 Concrete Structures in the 21st Century , Vol.2 Session.8, CD-rom.

Tamura, M.(2003). “Resource conservation strategy for concrete structure in lifecycle design” Doctoral thesis of the University of Tokyo. (in Japanese)

Tamura, M. and Noguchi, T (2003). “Resource Conservation Strategy for Concrete Structures”, Proceedings of Eco Design 2003, 3rd International Symposium on Environmentally Conscious Design & Inverse Manufacturing, 531-536.

Tamura, M., Tomosawa, F. and Noguchi, T. (2004). “Cementitious-Waste-Free-Type Completely Recyclable Concrete”, RILEM International Symposium on Environment-Conscious Materials and Systems for Sustainable Development ECM2004, 61-72.

Tamura, M. and Noguchi, T. (2004). "Resource Conservation Strategy for Concrete Structures", CONSEC'04, Fourth International Conference on Concrete under Severe Conditions: Environment & Loading, Vol.2, 2069-2073. International Symposium on Sustainable Development of Cement and Concrete, 147-160.

Tomosawa, F. and Noguchi, T.(1996). "Towards Completely Recyclable Concrete", Integrated Design and Environmental Issues in Concrete Technology, E & FN SPON, 263-272.

World Commission on Environment and Development. (1987). "Our Common Future".

Energy Performance of Switchable Glazing – IEA Solar Heating and Cooling Programme, Task 27

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ABSTRACT
Approximately 40% of the energy consumption in Europe is connected to heating, cooling and artificial lighting of buildings. Most of this consumption generates CO_2 emissions into the atmosphere, which we want to reduce. Traditionally most of the energy that goes into a building is consumed during its service lifetime and only a smaller part during its construction and destruction. As energy efficiency is improved, this may change and by investing more energy during the construction phase, the total energy load can be reduced. Unless they are carefully designed and appropriate glazing is installed, buildings with large glazed areas may need more energy for heating during the winter and also need electricity to run air-conditioning systems. Windows using modern coated glazing products with either high or low values of the total solar energy transmittance in combination with a low thermal emissivity can reduce these problems considerably and save large amounts of energy both for heating and cooling in our built environment A new emerging window technology for energy efficient windows makes use of switchable glazing products. These are coated glass or polymer sheets, for which the light and solar transmittance can be varied by exposing the active coating to a modified gas fill or to a low electrical voltage. Windows with a light transmittance that can be varied between around 60 and 10 percent have been produced and evaluated within the International Energy Agency, Solar Heating and Cooling Programme, Task 27. These windows have been tested for durability and the energy performance has been simulated using building simulation software. By having them in the dark state whenever cooling is needed and in the light state whenever heating is needed passive solar energy is utilised for heating and the cooling load is reduced by blocking solar irradiation. Control strategies are in general more complex and daylighting, glare and thermal comfort are aspects that need to be taken into account, as well as personal preferences. In this presentation some of these aspects are discussed, and some results of building simulation of the energy performance are presented. For a standard reference office block, the energy saving potential in three different climates has been simulated. The results indicate large savings in cooling load and, perhaps more important, a reduction of the cooling power needed. If installed air-conditioning power can be reduced, or even eliminated, the investment cost of switchable glazing can be justified. Sensitivity studies using building simulation also play an important role in assessing the effect of product durability on building energy performance over the entire product lifetime. During Task 27 tests have been performed to attempt to relate outdoor durability tests with accelerated ageing tests under controlled switching conditions. It is important to note that the emphasis was on developing the testing procedure, and that the results obtained from prototypes are not representative of technically mature products.

KEYWORDS
Switchable glazing, Energy performance, Durability testing

1 INTRODUCTION

Modern advanced glazing materials include products with optimised optical properties for different applications. In particular, recent developments in glass coating technology have led to a large number of coated glazing products with high light transmittance and low emittance of thermal radiation for energy efficient window applications. A special group of advanced windows are the so-called switchable windows. These are windows that can switch between high and low transmittance of solar and/or visible radiation. Thus, they can adapt to varying solar irradiation conditions and maintain a comfortable level of light and thermal transmittance. Owing to this ability to switch, these window have a further energy saving capacity compared to static windows.

The switching can be between high and low transmittance or between transparent and translucent states. This short presentation is limited to the high/low transparent case. Presently there are two competing technologies to achieve the switching. Either the switching is controlled by a low electrical voltage between transparent electrodes (electrochromic) or by the addition of hydrogen in the gas phase to a coated surface (gasochromic). In both cases the coatings become absorbing upon reduction. An oxidation process leads to bleaching. In the electrochromic case hydrogen can be replaced by lithium. For a detailed description of the function of these surfaces the readers are referred to the literature. [Granqvist, 1995, Wilson et al, 2002].

This presentation is a summary of the activities carried out within the International Energy Agency, Solar Heating and Cooling Programme - Task 27. Switchable glazing was studied with respect to its energy performance and long term durability. The performance and life time of these products depend on a number of external parameters and many of the results are still only preliminary. A very important goal of the task is thus the identification of recommended procedures to evaluate switchable glazing.

2 TESTED WINDOWS

Prototypes of switchable glazing from three European manufacturers have been tested and evaluated within the task. The object has not been to compare products, but instead to evaluate energy performance and durability test procedures. The switchable prototypes are therefore not identified in this report, and the results are simply referred to windows A, B or C. Neither of the switchable coatings provides a surface with low thermal emittance. In order to get a window with a low U-value, the switchable panes have to be combined with a separate low-e coated pane in a double or triple glazed unit. A list of the relevant optical and thermal parameters of the tested windows is shown in table 1. The task has lasted for 5 years and product specification has changed over these years. Building simulations performed by the different participants presented in this report may therefore use slightly different input data.

Window	A	B	C	Ref
TSET (g)	0.30-0.10	0.48-0.18	0.40-0.16	0.62
T_{vis}	0.47-0.10	0.60-0.15	0.52-0.16	0.77
U value (W/m^2K)	1.2	0.93	1.2	1.3

Table 1. Solar and thermal parameters for window simulations. TSET=g=Total solar energy transmittance (including absorbed and re-radiated energy), Tvis = light transmittance, U-value=thermal conductivity of window. A, B and C correspond to switchable windows and ref is a standard double glazed low-e window.

3 ENERGY PERFORMANCE

The performance of a window depends on a large number of parameters and is in most cases difficult to evaluate. In the winter when it is much colder outside than inside, the net energy flow through the window is outward and the room needs to be heated. In the summer it is often warmer outside than inside and then the net energy flow is inward. In addition to the thermal flow driven by a temperature difference, there is also the solar irradiation transmitted through the window and this is where switchable glazing can help to control the energy flow. Inside the building, heat is generated by people, lighting and various electrical appliances as well as by the heating radiators. It is desirable to maintain an indoor temperature between 20 and 26 degrees. To achieve this we generally need both a heating and a cooling system. Since more than one third of the total energy consumption in Europe today is associated with temperature control of our buildings, it is easy to understand the importance of keeping the consumption down. In later years cooling has become more common and the summer of 2003 was very hot in Europe with temperatures reaching 40 degrees in many places. The trend with an increasing use of air conditioning is alarming since it inevitably increases the consumption of electricity as well as the peak demand.

In order to calculate the total energy balance of buildings, simulation tools have been developed. They make it possible to simulate the temperature inside the rooms of a building hour by hour throughout the year. The physical properties of all building elements need to be known, together with the meteorological data for the location of the building and occupant behaviour. All simulation tools are based on models and simplifications of the building structure. The simulation tools so far used in Task 27 are based on the software tools TRNSYS [Platzer, 2003], CLIM2000 [Covalet and Greiffier, 2003], BSIM [Nielsen, 2002] and DEROB [Kvist, 1999]

The simulations were performed for three different locations in Europe, representing Mediterranean, central European and northern European climate zones, Rome, Brussels and Stockholm, respectively. Meteorological data were generated by the program Meteonorm.

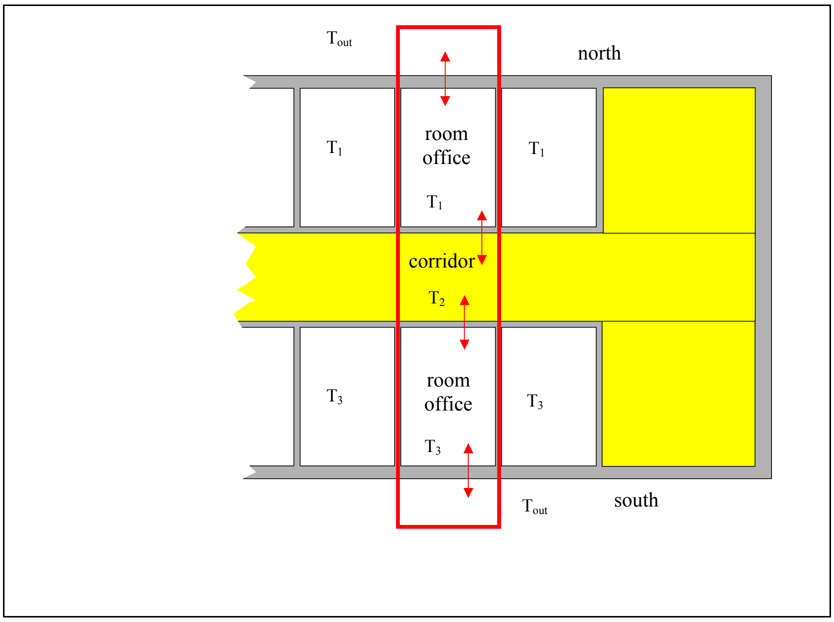

Fig 1 Standard reference office for building energy simulation within IEA SHC Task 27.

The simulations were applied to a “standard reference office”, defined in Fig. 1. This office consists of identical office modules facing north and south separated by a corridor. The offices can be seen as independent units unaffected by adjacent offices, but interacting with the corridor and the opposite office. Lighting, heating, cooling, office equipment, occupancy and several other parameters are specified, so that different simulation methods can be compared using identical input parameters.

Switchable glazing needs a control strategy for when it should be in the dark state and when it should be in the bleached state. This strategy can be based on a number of parameters, one of which is the unpredictable preference of the occupants. Minimising power consumption is not necessarily what the occupants want. Glare reduction may be the prime factor for the staff who sit in front of computer screens, while temperature is more important to others. Physical parameters that can be used for the control are solar irradiation, outside temperature, indoor temperature, light level inside the office, occupancy, time of day and time of year. For each one of these parameters, different set points and different control algorithms can be used. The switching time is of the order of several minutes so a time constant must be used to avoid too frequent switching. All this means that the simulation of the energy and power consumption of the office block is complex.

4 SIMULATION RESULTS

Only a small selection of the obtained results is presented in this summary report. These can be seen as examples of how data can be presented and give an indication of the results. In table 2 the results from the different simulation tools can be seen for some of the glazing options for heating and cooling and for the south and the north office in Fig. 1. All results can be separated into, for instance, monthly values and for different control. For simplicity the presented results are only calculated for the switchable glazing either in a permanent bleached state or in a permanent dark state.

Heating BRUSSELS

	< Heating - south office			>< Heating - north office			>
Window	clim	bsim	derob	trnsys	clim	bsim	derob	trnsys
Low-e	499		277	374	596		389	614
A(bleached)	619	589	367		696	678	449	
A(coloured)	750	725	452		797	776	505	
B(bleached)	486	448	259	400	584	547	355	604
B(coloured)	709	651	395	627	763	707	452	729

Cooling BRUSSELS

	< Cooling - south office			>< Cooling - north office			>
Window	clim	bsim	derob	trnsys	clim	bsim	derob	trnsys
Low-e	328		703	410	226		351	171
A(bleached)	103	216	248		79	144	167	
A(coloured)	35	99	120		32	83	89	
B(bleached)	271	393	576	269	179	235	305	124
B(coloured)	46	118	152	63	40	95	109	42

Table 2. Annual heating and cooling energy demand (kWh/m^{2}year) for different glazing options. Simulation tool from Section 2 indicated in table head.

For optimum energy performance, seasonal switching between these two states is probably close to what is the best choice, since cooling is only required in the summer and heating in the winter. Thus the windows can be kept in a permanently dark state during the summer months and vice versa for the winter. As can be seen from the simulated values in the table there are considerable differences in the energy demand as obtained from the different simulation tools. The exact reasons for these differences have not yet been fully evaluated. What is interesting to see is the low values for cooling for the case when the switchable glazing is left in the dark state and that the energy needed for heating is of the same order as for the low-e reference glazing. In the simplest possible control strategy the window is set in the dark state when cooling is needed and in the bleached state otherwise.

4 DURABILITY OF CHROMOGENIC GLAZING

As chromogenic glazing is a very new product that is just entering the market, the data base on its durability is very limited. In an attempt to gain information that can be used in simulation sensitivity studies, two approaches have been followed in parallel: newly defined accelerated ageing test procedures for chromogenic glazing, and outdoor exposure in test stands. It must be emphasised that because the research field is so new, the results presented here were obtained with prototypes that were still in the process of development.

4.1 Accelerated Ageing Tests

A number of accelerated ageing tests for chromogenic glazing have been defined during the duration of IEA Task 27, in the U.S.A. as ASTM standards or drafts, and in Europe as exploratory tests within the EU-funded SWIFT project [Wilson, 2003]. A series of tests has been defined, in which the samples cycle continuously between the bleached and coloured states (representing one form of acceleration), and are subjected to different temperature and/or radiation conditions.

The ASTM tests and the SWIFT tests differ significantly in the specified number of cycles. If the criterion is the number of cycles expected during a 20-year lifetime, 3600 is probably too low (less than one cycle per two days - SWIFT). However, 50000 cycles (ASTM), corresponding to more than 6 cycles per day, is much too high. The experience of one industrial partner with chromogenic windows installed in offices was that they were switched for one or two cycles per day, or even less frequently. This corresponds with surveys of office workers, which indicated that in most cases, lighting conditions were changed (by switching on lights or operating mechanical blinds) only when the occupant entered the room in the morning and afternoon, or if there were disturbing glare. On this basis, it is suggested that 7000 cycles would be a more suitable limit.

However, it may be more appropriate to choose a still lower number of cycles, so that the total testing time remains with acceptable limits, but the duration of the cycling period is long enough to ensure cycling over at least e.g. 80 % of the maximum transmittance switching range. Depending on the size of the sample, up to 60 minutes may be appropriate for a complete switching cycle. It is probable that 3500 cycles between transmittance values corresponding to a given change in optical density represents a greater stress to the active material in the chromogenic unit than 7000 cycles covering only half the change in optical density.

Investigations by one industrial partner have indicated the importance of using samples of different areas for the cycling tests. A minimum area of 400 mm x 400 mm was recommended (preferably 600 mm x 600 mm), although a longer time is needed for one complete cycle than with smaller samples. This is because the number of cycles successfully completed by larger samples was found to be larger than that for smaller samples which were otherwise identical in composition, i.e. the smaller samples did not adequately represent the performance of the larger ones.

Two thermal cycling tests were also defined analogously to existing standards such as prEN 1279-2, which was specified to test the durability of the seal in conventional insulating glazing units. In accordance with the aim of testing the complete system, the scientists in the SWIFT project propose that the chromogenic glazing unit remain connected to the supply and control units during the test, although the latter need not be subjected to the test conditions if they are to be installed indoors.

4.2 Outdoor testing

Site conditions

Measurements were made at two different sites, with two different types of samples, C and B. One site was the Fraunhofer Institute for Solar Energy Systems in Freiburg, Germany (48°00' N, 7°51' E, altitude: 269 m). The measurement period is from 1st December, 2001 to 31st August, 2003. The other site was the Centre Scientifique et Technique du Bâtiment (CSTB) in Grenoble, France (45°11' N, 5°43' E, altitude: 212 m). Results are presented from 1.1.02 to 31.8.03. for this site. As the statistics for the average temperature and total radiation values indicated, the second year included an unusually hot and dry summer, resulting in a high frequency of high radiation values at both sites.

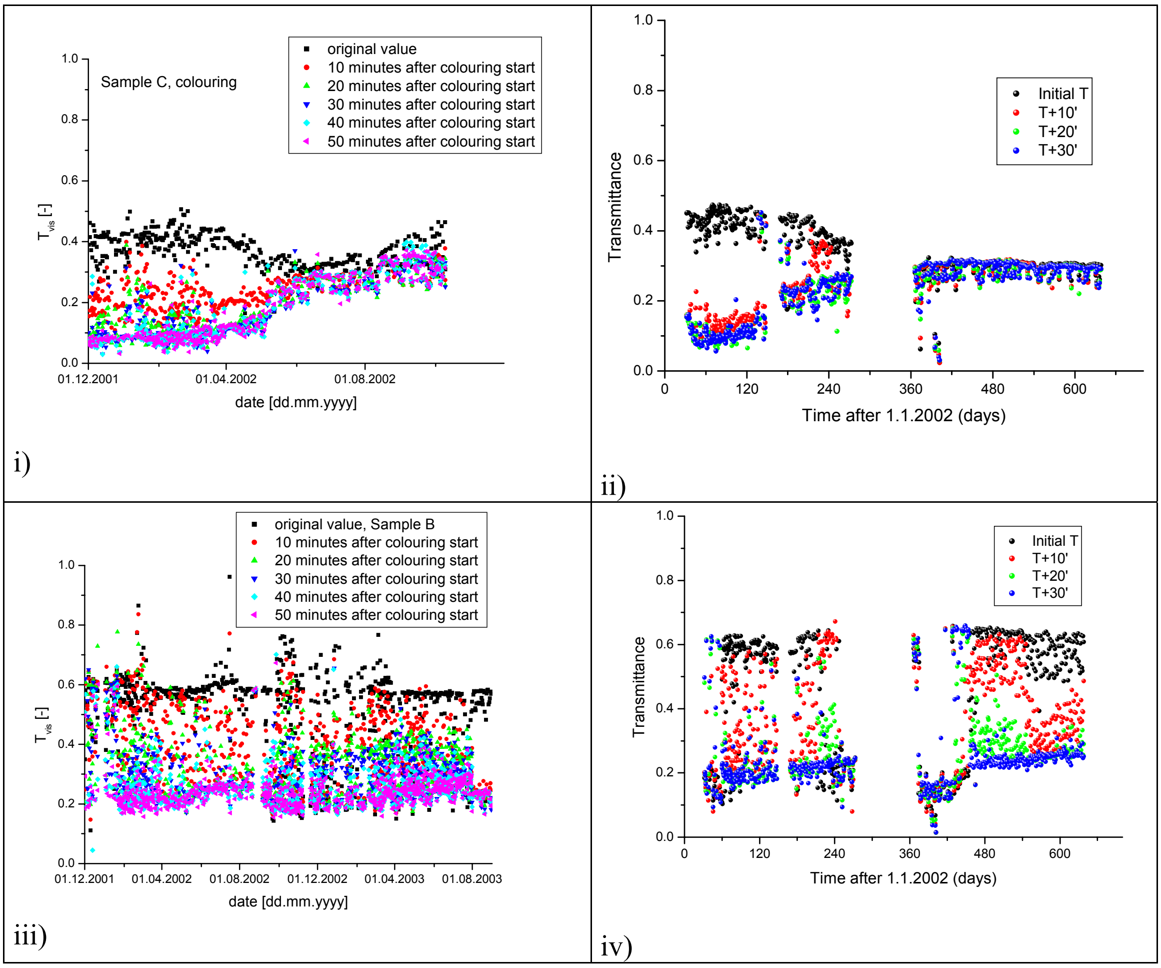

Fig. 2: Visible transmittance values at 10-minute intervals after the beginning of colouring or bleaching for the prototype chromogenic glazing samples C and B, exposed outdoors in Freiburg and Grenoble. i) Window C, Freiburg, ii) Window C, Grenoble, iii) Window B, Freiburg, iv) Window B, Grenoble

Test Stand

At both test sites, the chromogenic glazing system prototypes were exposed outdoors in a test stand which is tilted 45 ° and orientated due south. The total glazing area for each type of sample, each represented by two IGUs, was 1.08 m^2. The glazing is cycled twice a day during the daylight hours. In addition to standard meteorological data, the glazing surface temperature, the UV radiation, solar radiation and daylight incident on the plane parallel to the glazing are monitored continuously. The visible transmittance is used as the performance parameter, and is monitored with luxmeters located outside and behind the glazing units. As the solar incidence angle varies throughout the day and year, the transmittance values originally calculated from the measured luxmeter ratios reflect the angular dependence of the glazing transmittance. The transmittance values presented here from Freiburg were subsequently corrected to the values for near-normal incidence on the basis of laboratory measurements. To reduce extrapolation errors, the transmittance values used for analysis were restricted to those measured with angles of incidence less than 65° and reference illuminance values exceeding 4500 lux. The transmittance values from Grenoble still include the variation due to the angle of incidence.

Figure 2 gives a representation of each switching process by showing the transmittance value at different times after the start of bleaching or colouring processes. Despite filtering the data to remove points corresponding to the lowest light levels and highest incidence angles, the prevalence of low light levels and high incidence angles clearly reduces the accuracy of the transmittance values determined in winter, causing wider scatter of the points then. However, qualitative trends can be recognised, including the similarity in behaviour shown at the two different sites for samples of type C or B.

4.3 Comparison of results from accelerated ageing and outdoor exposure

It is important to note that the results presented here are from prototypes in the development process, not from technically mature products. The aim of the comparison is to see whether the information from the accelerated tests correlates adequately with that from outdoor exposure. On the basis of the graphs for sample C at both sites in figure 2, it is clear that the transmittance values for the coloured and bleached states have converged and the switching rate has decreased so much that the chromogenic glazing system prototype has failed. This agrees qualitatively with the results from the accelerated SWIFT tests 1 and 3 at 5 °C and 23 °C. However, the high-temperature accelerated test, SWIFT 2, on this sample did not reveal the same type of marked degradation as had been seen in the outdoor test after high temperatures in spring and summer. By contrast, the severe SWIFT 2 test caused catastrophic failure of sample B, although the outdoor performance is good. This raises the question of an appropriate temperature and radiation level for the accelerated ageing test. Obviously, the temperature should not be so high that the activation energy threshold for new degradation mechanisms is exceeded. However, even if a temperature is chosen which is lower than the peak which can be experienced in outdoor exposure, it can be questioned whether the continuous cumulative load on the switching system only accelerates degradation or whether it enables a sequence of mechanism steps that would be terminated and may even reverse during the nocturnal conditions of natural exposure.

In choosing testing conditions for architectural chromogenic glazing systems, there may be a case for distinguishing the requirements according to the application. For example, safety standards differentiate between vertical and overhead glazing - for absorptive glazing such as coloured chromogenic glazing, the difference in maximum possible incident solar radiation intensity could be significant. Similarly, a classification according to climatic zones, as is the case for energy rating systems, could also be considered. Finally, the flexibility associated with the control unit, which is an integral part of a chromogenic glazing system, means that precautionary measures can be taken to prevent the glazing from reaching a temperature known to be harmful. A test specifying a higher temperature then may become irrelevant.

5 CONCLUSIONS

The results of the simulation work performed within Task 27 clearly indicate a large saving potential for buildings, especially in warm climates. An important result not specifically shown by the results here is that the cooling *power* is considerably reduced. This can help to reduce peak loads for the local energy system, and also in many cases make expensive and energy consuming air conditioning units obsolete. In central and northern European climates, it is likely that buildings designed with proper solar shading can maintain a comfortable indoor climate without air conditioning. The results of these investigations are not fully evaluated. It is, however, of interest to compare different tools having different degrees of built-in approximations and simplifications, using the same input parameters. The correct answers will have to wait until switchable glazing is more readily available and can be installed in more than just a few test offices.

Based on observations of architecturally dimensioned, chromogenic glazing systems exposed to natural weathering, some questions are raised on the appropriate choice of loads, switching cycle duration and number of cycles in current and proposed accelerated ageing tests. Definitive answers to these questions will require further time-consuming accelerated ageing and natural exposure tests, and careful analysis and comparison of their results. Although it is clear that the results presented here are still of a provisional nature, it is suggested that they provide a useful basis for sensitivity studies on the effect of changing technical specifications on building energy performance. Given the lack of clear correlation between accelerated ageing and outdoor exposure results, at present it seems more appropriate to draw on the results gained in outdoor exposure.

6 REFERENCES

Covalet, D. and Greffier, V., Modelling a reference office building proposed in T27 of IEA - Application to chromogenics, Task 27 report, EDF HE-14/03/006 (2003)

Granqvist, C.G., Handbook of inorganic electrochromic Materials, Elsevier, Amsterdam, 1995

Nielsen, T.R., Simulation results for reference office with electrochromic and gasochromic glazings, IEA Task 27 report (Sept 2002)

Kvist, H., User Manual for DEROB-LTH, Lund University, Lund, Sweden, 1999

Platzer, W.J., Switchable Façade Technology - Energy Efficient Office Buildings with Smart Facades, Proceedings ISES Solar World Congress, Göteborg, Sweden, 2003

Wilson, H.R., Blessing, R., Hagenström, H., Hutchins, M.G., Dvorjetski, D., Platzer, W.J., 2002, 'The optical properties of gasochromic glazing', Conf. Proc. 4th ICCG, Braunschweig, Germany, 3rd -7th November, 2002, pp. 649-657

Wilson, H.R., 2003, 'Steps toward Appropriate Accelerated Ageing Tests for Architectural Chromogenic Glazing', Proc. IEA SHC Task 27 Dissemination Workshop, Freiburg, Germany, 6th October, 2003

Effects of Weathering on Fatigue Resistance of Epoxy Injection Repairs of Concrete

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ABSTRACT

Epoxy injection repairs of alive cracks in concrete are prone to deterioration by fatigue resulting from the expansion/contraction of concrete. Also, the fatigue resistance of epoxy resin can decreases over time with their properties by weathering. The evaluation of epoxy repairs by fatigue in consideration of weather resistance is essential for achieving long-term performance of such repairs. In this paper, the effect of weathering on fatigue resistance of epoxy repairs of concrete was investigated.

A fatigue test apparatus that reproduces the same level of movements observed in actual crack was first developed. A dog bone shaped mortar specimen that epoxy resin was injected into 1mm gap between the two half pieces of mortar was exposed outdoors in Yokohama for three years. The typical two kinds of epoxy resin such as high modulus type and low modulus one were selected as injection materials for repair. After outdoor exposure, fatigue tests were carried out at the amplitude of 0.002mm for the high modulus type and 0.03mm for the low modulus one respectively.

The repetition number of cycles to failure was noticeably decreased for exposed specimens compared to the unexposed ones. The relation between weather during exposure and the fatigue tests results of exposed specimens, and the main affective elements are presumed to be heat and moisture. An accelerated deterioration test was finally discussed, and it was found out that the equivalent test condition to degradation of three years' exposure was immersion in warm water of 50□ for 50-250 hours.

KEYWORDS

Epoxy resin, Repair, Crack, Fatigue, Movement

1 INTRODUCTION

Epoxy injection repairs of alive cracks in concrete are prone to deterioration by fatigue resulting from the expansion/contraction of concrete. Also, the fatigue resistance of epoxy can decrease over time like other organic materials, with their physical properties being altered while in use outdoors. The evaluation of epoxy repairs by fatigue in consideration of weathering resistance is therefore essential for achieving long-term performance of such repairs.

The deterioration factors relevant to epoxy repairs are basically the same as those that deteriorate normal polymeric materials. In view of the fact that epoxy is injected into concrete cracks, the deterioration factors can primarily include the heat from the air temperature and sunlight and water from rain and humidity. Solar ultraviolet radiation can also adversely affect the surfaces of epoxy, which are exposed to weather at the concrete cracking.

Another concern relates to the deterioration factors affecting the bond between the resin and concrete in addition to the resin body. The evaluation should therefore include the adherend. With this as a background, this study aims to make clear the effect of weathering-related deterioration on the fatigue resistance of epoxy injected in concrete cracking.

2 PAST STUDIES

Epoxy injection has a long history of application to concrete repair, with a large number of studies having been reported. However, most of them have reported only the effect of repair using unexposed specimens based on simple mechanical testing. In regard to the deterioration of epoxy injection repairs, L. Tu et al., M. Hawary et al., and M. Y. Al-Mandil and K. E. Hassan investigated the adverse effects of water, seawater, and heat, respectively. Motohashi investigated the influence of outdoor exposure using specimens exposed for 5 years. However, all these studies deal with the deterioration of resin injection repairs by static testing. There have been few studies that investigated the effect of meteorological factors on the epoxy repairs of cracking subjected to concrete movement.

3 TYPES OF REPAIR SPECIMENS AND THEIR PROPERTIES

3.1 Fabrication of specimens

Two types of epoxy repair materials commercially available were used: high-modulus and low-modulus epoxies, each comprising modified epoxy as the base and modified aliphatic as the hardener. These were used for repairing dog-bone-shaped mortar specimens as shown in Figure 1. Though concrete is used in actual applications, mortar was used for this experiment to facilitate the testing.

Mortar specimens were made as follows: Place mortar made by mixing 514 kg/m^3 of normal portland cement, 277 kg/m^3 of water, and 1,337 kg/m^3 of sand in the specified forms. Water-cure the demolded specimens for 3 months. Cut each specimen into two at midlength with a concrete cutter and allow the cut pieces to dry for 1 month in the laboratory conditions. Fix the two pieces with a clearance of 1 mm in between and fill the space with epoxy. Cure the repaired specimens for 24 h at 20°C and for another 24 h at 50°C to fully harden the epoxy.

3.2 Tensile properties of epoxy-repaired specimens

Tensile tests were conducted using an Instron-type testing machine to investigate the basic mechanical properties of epoxy-repaired specimens. The elongation rate was 1 mm/min, and the test temperature was 20°C. Note that the displacement of the repair width with the high-modulus epoxy was so small that it was directly measured using a digital microscope (×800). That of the repair width with the low-modulus epoxy was measured by fixing a differential transformer at the critical area adjacent to the repair of each specimen.

Figure 2 shows the tensile stress-elongation relations as averages of five specimens. When comparing high-modulus epoxy-repaired and low-modulus epoxy-repaired specimens, the tensile stress of the former is high but the amount of deformation to failure is small, whereas the tensile stress of the latter is not so high but the amount of deformation is large. Moreover, the stress of the latter shows a peak and then diminishes. The failure modes of the two types also differed. The high-modulus epoxy type failed in the mortar near the repair, whereas the low-modulus epoxy type showed both cohesive failure of the resin and the interfacial failure.

3.3 Fatigue properties

3.3.1 Fatigue test conditions

The movements for the high-modulus and low-modulus epoxy-repaired specimens were 0-0.002 mm and 0-0.03 mm, respectively, based on the measurement of crack movements in actual walls. The period of movement was set at 30 sec to shorten the test period, though the period of movement of actual structures is generally 1 day. The test temperature was 20°C.

3.3.2 Test apparatus and procedure

A fatigue apparatus shown in Photo 1 was used. This apparatus, a prototype designed and produced for this study, is furnished with a mechanism that converts the fine rotational movement obtained via a transmission to linear movement. A load cell is fixed at the end of this apparatus to

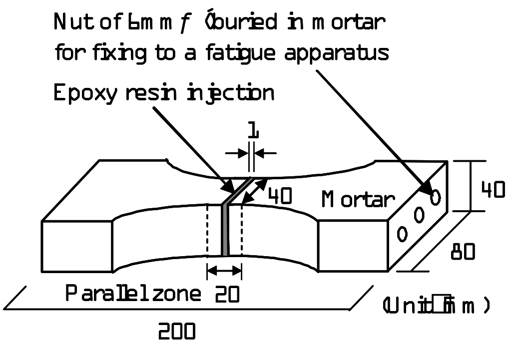

Figure 1 Test specimen

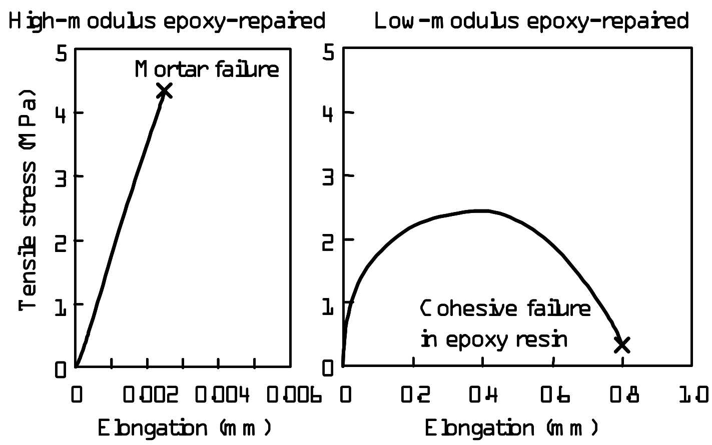

Figure 2 Mechanical property of epoxy-repaired specimen

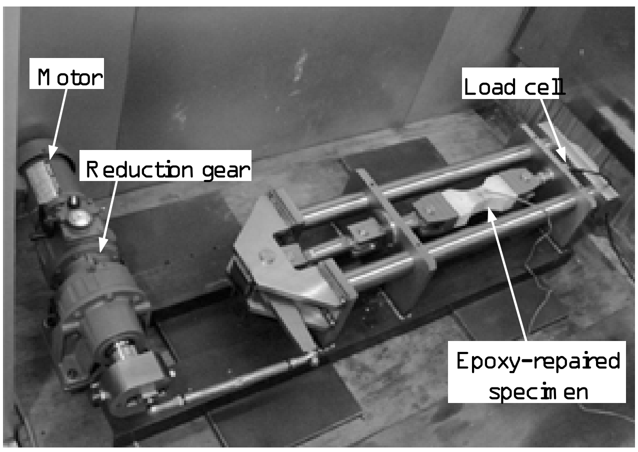

Photo. 1 Fatigue apparatus

measure the changes in the loading during fatigue testing. The body of the apparatus is installed in a room at 20°C. During testing, changes in the loading resulting from the movement were measured. The state of rupture and failure of specimens was also visually observed.

3.3.3 Results

Figure 3 shows the movement of high-modulus epoxy-repaired specimens and typical changes in the resulting stress during fatigue testing. The initial stress amplitude under the movement decreased as the number of cycles increased, though only tensile stress is generated due to the cyclic motion on one side only. Such reductions are considered to indicate the progress of fatigue in the specimens. Figure 4 focuses on the stress amplitude of unexposed specimens repaired with high- and low-modulus epoxies, showing the state of stress reduction related to the number of cycles. The peak stress decreases as failure approaches and rapidly decreases to failure in both cases. The high-modulus epoxy type failed in the mortar, whereas the low-modulus epoxy type specimens underwent both interfacial failure and cohesive failure in the resin.

4 OUTDOOR EXPOSURE TEST

4.1 Procedure of exposure

Each epoxy-repaired specimen was horizontally placed with a flat surface up on racks 1.5 m in height as shown Photo.2. All surfaces of specimens were therefore exposed to the air temperature and humidity, while mostly the top surfaces were exposed to sunlight and rainwater. Note that the bottoms of specimens were open to the air, allowing free airflow.

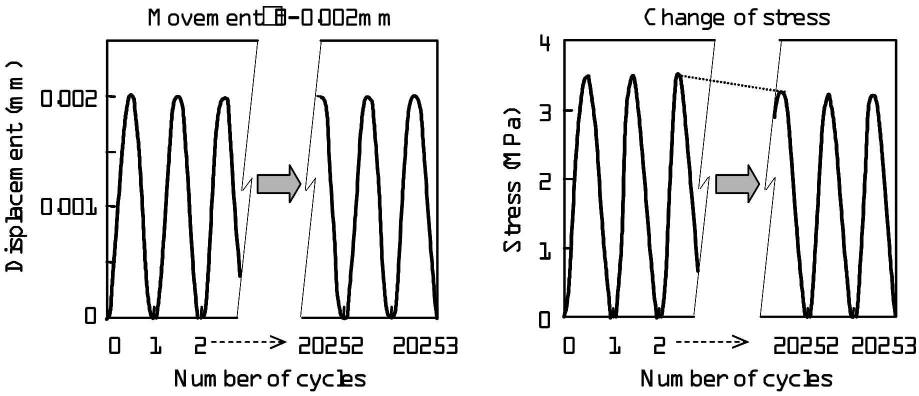

Figure3 Typical change in the resulting stress of high-modulus epoxy-repaired specimen during fatigue test

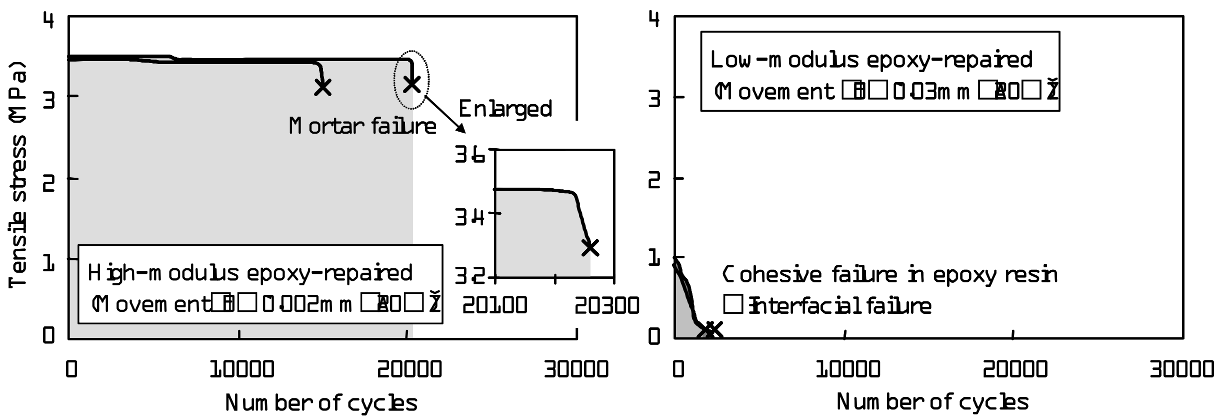

Figure 4 Stress change related to repetition in fatigue test

The exposure site was the roof of a 9-story building in a campus of Tokyo Institute of Technology in Yokohama. The exposure period was 3 years from January 2001 to December 2003, during which the weather conditions were as summarized in Table 1 based on the monthly report released by the Japan Meteorological Agency.

4.2 Results

4.2.1 Changes in appearance

The top surfaces of epoxy repairs exposed to sunlight appeared to be roughened. This was considered to be deterioration by sunlight. Since the depth of such roughening was of concern with respect to epoxy repair, the vertical cross sections of epoxy repairs were therefore microscopically observed. A typical cross section is shown in Photo 3. The depth of resin alteration appeared to be limited to around 0.05 mm from the top surface, with the part deeper down being scarcely affected.

4.2.2 Tensile properties

The tensile test results of exposed specimens are shown in Figure 5. The results of unexposed specimens are also shown in the figure for comparison. After the exposure, the maximum tensile stresses of both high- and low-modulus epoxy types tended to be slightly higher, and elongation at failure tended to be lower than the values before exposure, but the changes were both marginal. Though the failure was limited to the mortar portion in unexposed specimens repaired with the high-

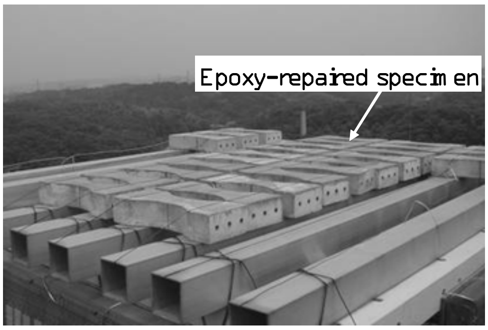

Photo. 2 Outdoor exposure

Table 1 Weather conditions during outdoor exposure (Yokohama Jan.2001-Dec.2003)

Mean air temperature[℃]	15.9
Daily maximum temperature [℃]	36.9
Sunshine duration [hour]	5995
Rainy days [day]	331
Amount of precipitation [mm]	5400

* From the monthly report of Japan Metrological Agency

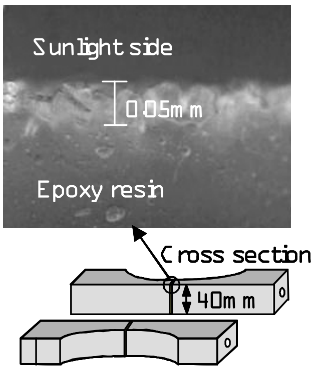

Photo. 3 Deterioration in a cross section of epoxy resin

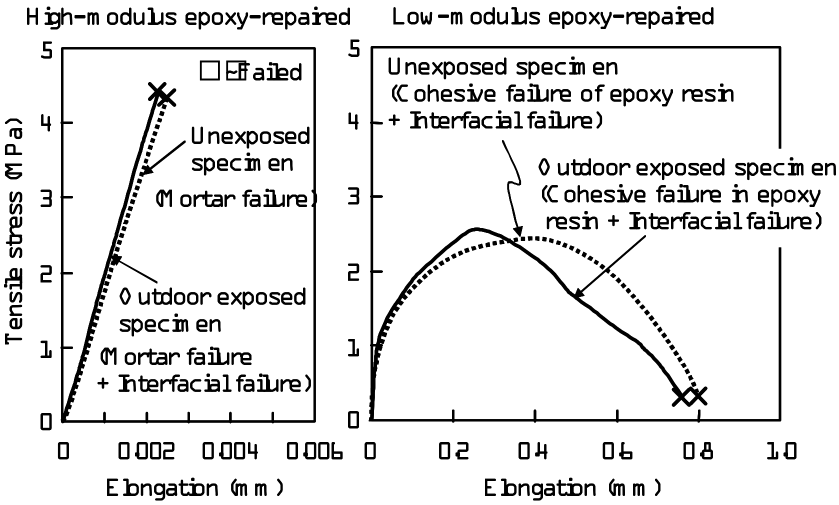

Figure 5 Tensile test results of the specimens exposed in outdoor for 3 years

modulus epoxy, interfacial failure was partially observed after exposure in addition to mortar failure.

On the other hand, no difference in the failure mode was observed between the unexposed and exposed specimens repaired with the low-modulus epoxy, both showing cohesive failure in the resin and interfacial failure.

4.2.3 Fatigue resistance of outdoor exposed specimens

Figures 6 and 7 show the changes in the peak stress to failure during fatigue testing of the high- and low-modulus type specimens, respectively. Though the initial stress of the high-modulus-type exposed specimens is nearly the same as that of unexposed specimens, the stress decreases as the number of fatigue cycles increases, ending up in failure at as early as several hundred cycles. Thus the fatigue resistance is substantially reduced by the exposure. In regard to the failure mode of such specimens, the failure was limited to the mortar part in the unexposed specimens, but interfacial failure was added after exposure as shown in Photo 4. As for the low-modulus type, the number of fatigue cycles to failure was not significantly reduced by exposure. However, the failure mode changed from the mixture of interfacial failure and cohesive failure in the epoxy resin in unexposed repair specimens to interfacial failure after exposure as shown in Photo 5.

4.3 Comparison between simple tensile test results and fatigue test results

The adverse effect of outdoor exposure on the high-modulus epoxy type is not so significant on the mechanical properties when examined by simple tensile test. By fatigue testing, however, the number of fatigue cycles to failure is significantly reduced by exposure. In view of the fact that it is what we should concern the resistance to cyclic movement that practically occures, durability cannot be evaluated by simple tension tests alone.

Regarding low-modulus epoxy-type specimens, outdoor exposure does not significantly affect their mechanical properties, but the failure mode changes from a mixture of cohesive failure in the epoxy resin and interfacial failure to interfacial failure. In this respect, evaluation by cyclic loading is deemed significant.

5 ACCELERATED DETERIORATION TESTS

5.1 Investigation into deterioration test items

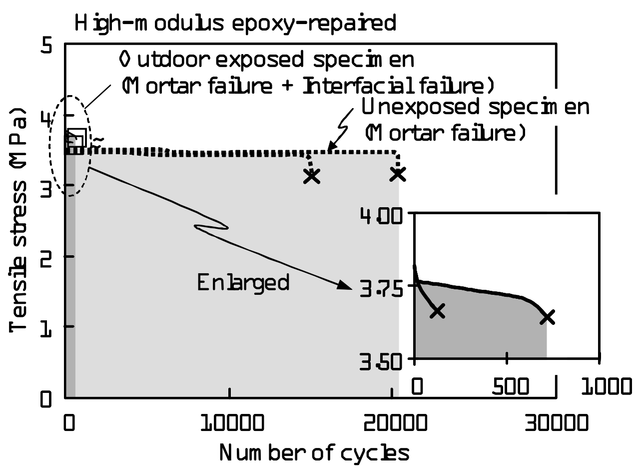

Figure 6 Stress change of outdoor exposed and unexposed specimens in fatigue test (High-modulus epoxy-repaired)

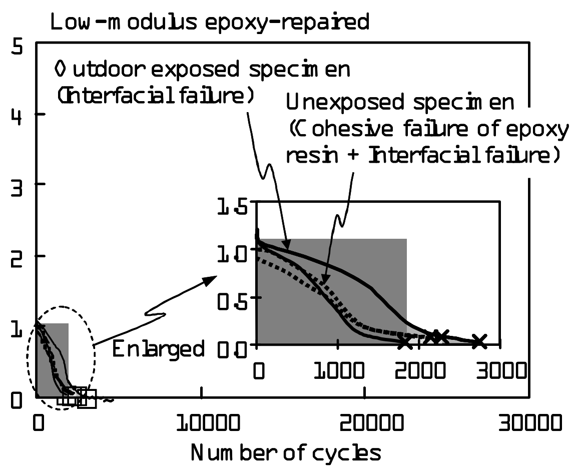

Figure 7 Stress change of outdoor exposed and unexposed specimens in fatigue test (Low-modulus epoxy-repaired)

Generally speaking, meteorological factors primarily affecting polymeric materials include air temperature, heat by the sun, solar ultraviolet irradiation, and moisture from mainly rainwater. The effect of heat, which underlies all types of deterioration, particularly in the high temperature range, is most detrimental. For this reason, the daily maximum temperature was focused on among other meteorological data, and the number of days, as well as the duration of bright sunshine and number of rainy days were arranged in histograms with respect to the daily maximum temperature ranges as shown in Figure 8. The air temperature exceeded 20°C on more than half the total number of days in the exposure period and even exceeded 35°C on 3 days. The duration of bright sunshine was long in the days with a temperature of 20°C or more, suggesting a strong impact of ultraviolet radiation at a high temperature. It was also inferred that the specimens were exposed to rainwater many times when the air temperature was high. It was therefore considered necessary to focus attention on the deterioration by heat in the high temperature range, combined deterioration by heat and ultraviolet radiation, and combined deterioration by heat and water.

Among these deterioration items, however, the effect of ultraviolet radiation was considered marginal on mechanical properties such as fatigue, since the deterioration of epoxy repair was limited to only a depth of 0.05 mm from the surface as described above, with the rest remaining intact. For this reason, only deterioration by heat and combined deterioration by heat and water were taken as the investigation items while omitting deterioration by ultraviolet radiation.

5.2 Heat deterioration testing

5.2.1 Investigation into heat deterioration test conditions

Though it was difficult to accurately define the deterioration conditions, test conditions of 50°C and 500 h were adopted for the following reasons: The temperature of epoxy repair in concrete cracks readily increases when it is exposed to sunlight at a high temperature. According to measurement in Yokohama, the temperature of subsurface epoxy repair was approximately 15°C higher than the air temperature in the daytime. A test temperature of 50°C was therefore adopted as a temperature 15°C higher than the highest maximum daily temperature. As for the deterioration time, the annual duration

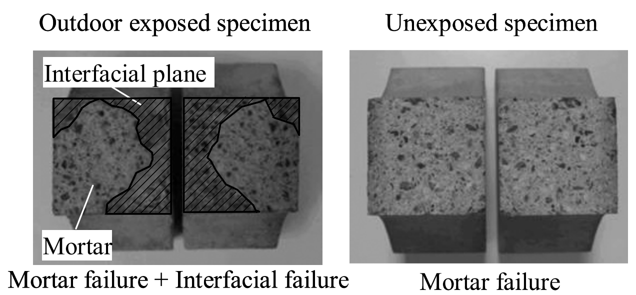

Photo. 4 Fracture mode (High-modulus epoxy-repaired)

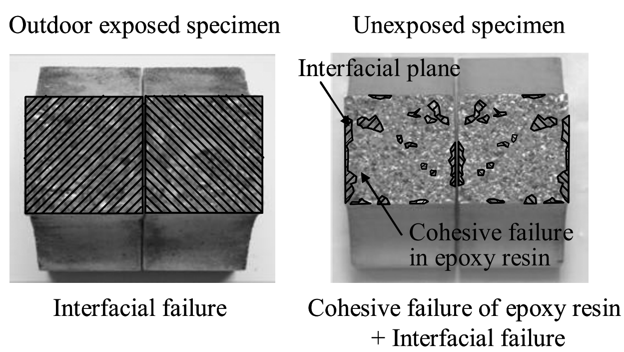

Photo. 5 Fracture mode (Low-modulus epoxy-repaired)

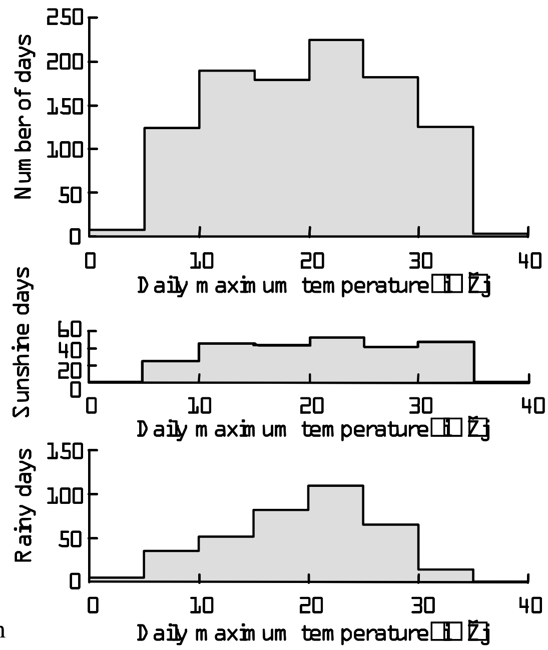

Figure 8 Weather condition of exposure period in Yokohama for 3 years

of high temperature was assumed to be 2 hours in the daytime for 3 months in summer, and the duration for 3 years is 540 hours. This was rounded off to 500 hours to be the test period.

5.2.2 Test procedure

Specimens repaired with high- and low-modulus epoxies were left in a controlled chamber at 50 ± 1°C for 500 hours. They were then subjected to cyclic movement of 0-0.002 mm and 0-0.03 mm, respectively, in an environment of 20°C after heat degradation treatment.

5.2.3 Results and discussion

The test results are shown in Figs. 9 and 10. The fatigue resistance of the heat degraded high-modulus epoxy type was nearly half that of unexposed specimens, whereas the resistance of the heat degraded low-modulus epoxy type rather tended to be higher than that of unexposed specimens. When compared with the outdoor exposure test results shown in Figs. 6 and 7, deterioration by heat alone is deemed insufficient, presumably because the effect of water involved in outdoor exposure is ignored.

5.3 Heat and water deterioration testing

5.3.1 Investigation into test conditions

To make test conditions closer to actual deterioration, methods of deterioration involving the effect of water were investigated. To facilitate such testing, immersion in warm water at 50°C was adopted. A deterioration time of 500 h was similarly adopted along with additional testing at 250 and 1,000 h for reference. Since the deterioration of high-modulus epoxy type was particularly significant under outdoor exposure conditions, shorter test periods of 50 and 100 h were also adopted for this type.

5.3.2 Test procedure

Specimens repaired with high- and low-modulus epoxies were fully submerged in warm water controlled at 50 ± 1°C in a water bath for the specified periods. These were then subjected to the same fatigue movement regimen of 0-0.002mm and 0-0.03mm, respectively, in an environment of 20°C.

5.3.3 Results

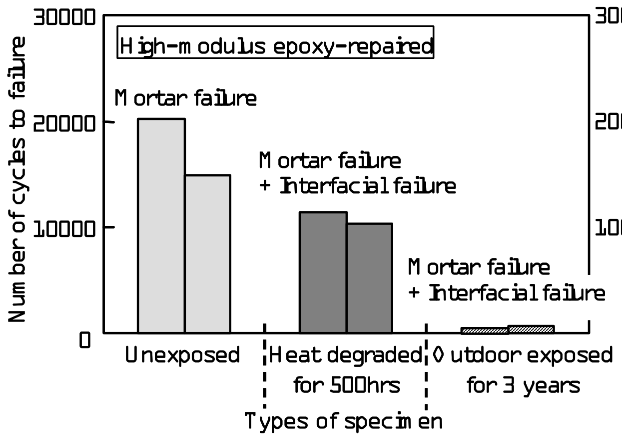

Figure 9 Fatigue test results after being heat degraded (High-modulus epoxy-repaired, movement: 0–0.002mm)

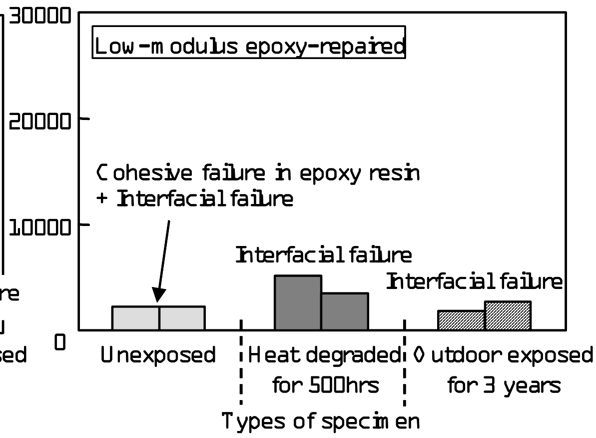

Figure 10 Fatigue test results after being heat degraded (Low-modulus epoxy-repaired, movement: 0–0.03mm)

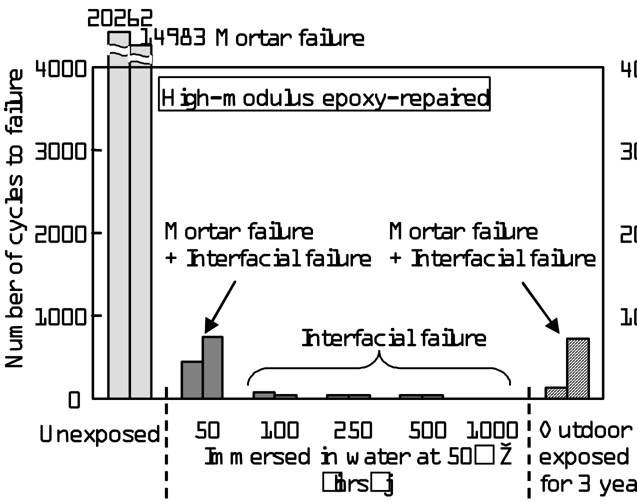

Figure 11 Fatigue test results after immersion in water at 50□Ž□movement: 0□0.002mm□j

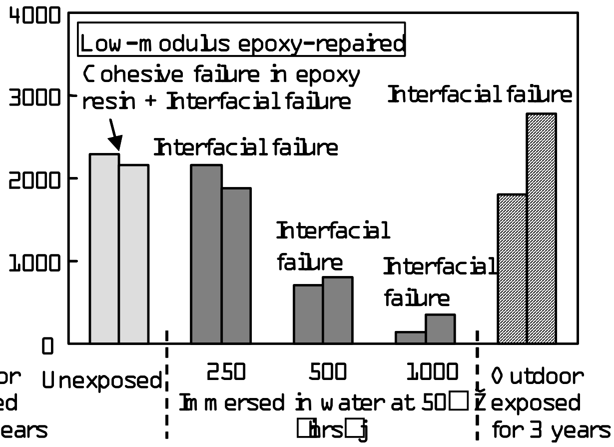

Figure 12 Fatigue test results after immersion in water at 50□Ž□movement: 0□0.03mm□j

he test results are shown in Figs. 11 and 12. As deterioration time increased, the number of cycles to failure decreased. The failure mode of the high-modulus epoxy type changed from mortar failure to mixed failure in mortar and at the interface, and then to failure solely at the interface. The failure mode of the low-modulus epoxy type changed from mixed failure of cohesive failure in the resin and interfacial failure as found in unexposed specimens to solely interfacial failure. The increase in the percentage of interfacial failure after accelerated heat and water deterioration can be attributed to the losses in the cohesive force between the epoxy and mortar under the influence of water. These results are similar to the results of outdoor exposure tests.

5.4 Accelerated deterioration conditions simulating outdoor exposure testing

Judging from the results, at least the presence of heat and water is essential as the factors of accelerating tests. However, the accelerated test results agreed with the 3-year exposure test results only in the cases of high-modulus epoxy type immersed for 50 to 100 h and low-modulus epoxy type immersed for 250 h. Since deterioration by accelerated test conditions thus varied depending on the material, the tests did not lead to a single set of conditions to simulate the outdoor exposure. The accelerated deterioration conditions of immersion in water at 50°C for 500 h were excessively harsh for simulating outdoor exposure for 3 years. However, since performance for longer ranges should be elucidated in any event, judgment will be made after obtaining the results of ongoing exposure testing.

6 CONCLUSIONS

The effects of meteorological deterioration on the fatigue resistance of the epoxy repairs of concrete cracking were investigated. The following results were obtained:

- The fatigue resistance of epoxy repairs is affected by meteorological conditions during outdoor exposure. The number of fatigue cycles to failure of exposed repair specimens decreases from that of unexposed repair specimens. The failure mode is also altered by outdoor exposure, with the percentage of interfacial failure increasing after exposure.
- Meteorological factors were analyzed and their effects were investigated based, the method of accelerated deterioration for their evaluation was then explored. As a result, it was ascertained that at least heat and water are essential as factors for accelerated deterioration.
- Though the conditions corresponding to outdoor exposure for 3 years varied depending on the material, immersion in water at 50°C for 50 to 250 h nearly simulated the results of 3-year exposure outdoors. Information on the deterioration conditions for investigating longer range performance of epoxy repairs remains insufficient. The development of ongoing exposure tests is

therefore awaited.

7 REFERENCES

I.A.Basunbul, A.A.Gubati, Al-Sulaimani and M.H.Bauch: Repaired Reinforced Concrete Beams; ACI Material Journal. Vol.87, No.4, pp.348-354, July-August 1990

H.W.Chung and L.M.Lui: Epoxy-Repaired Concrete Joints; ACI Journal, pp.264-267, June 1977

Lin Tu and Deon Kruger: Engineering Properties of Epoxy Resins Used as Concrete Adhesives; ACI Materials Journal, Vol.93, No.1, pp.26-35, January-February 1996

Moetaz EI-Hawary, Husain AI-Khaiat and Sami-Fereig: Effect of sea Water on Epoxy-repaired Concrete; Cement and Concrete Composites Vol.120, pp.41-52, 1998

M.Y.Al-Mandil, H.S.Khalil, M.H.Baluch and A.K.Azad: Performance of epoxy-repaired concrete under thermal cycling: Cement and concrete composites, Vol.12, pp.47-52, 1990

K.E.Hassan, P.C.Robery, L.Al-Aiawi: Effect of hot-dry curing environment on the intrinsic properties of repair materials; Cement and concrete composites Vol.122, pp.453-458, 2000

Kenji Motohashi: Durability of epoxy-repaired concrete joints by low pressure injection technique; Bousui Journal, pp.94-97, June 2003 (in Japanese)

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Modules of Environmental Assessment related to Durability and Service Life

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ABSTRACT

Where environmental assessment of buildings takes the full life cycle of the building into account, aspects of durability and service life of the building and its components deserve considerable attention. As service life is defined as the period of time during which the product fulfills or exceeds established performance requirements, it is an evident element of expressing the functional unit of a product. While the functional unit is in general the anchor of product alternatives' comparability, it is equally evident that performance requirements may be expressed on building level rather than for each building product. By that, products are to be considered as elements of the overall functionality. Functionality requirements must be expressed in such terms, that various design options can be verified as to the extent to which they meet these requirements. Where these design options in fact are products, it must be ensured that the functional relationship between the product option at hand and other products within the functional context reflects the performance requirements as expressed for the building or other functional entity. As functional units usually are expressed in terms of performance and relate to fitness for purpose, they also relate to the product in its application. The link to service life and durability is however not only concerning the duration of product application. Based on service life planning, scenarios for maintenance and demand for replacements can be identified and included in environmental life cycle assessment or other assessment methodologies.
Currently developed international standards, as well as recently published academic papers in the field of environmental assessment apply to increasing extent modular approaches. The advantage of such modularity is seen in the possibility to generate environmental product declarations, while offering users of declared information the possibility to replace scenarios included in the declaration and to thereby bespoke the content of a declaration to the preconditions under which the product is to be applied. Modular structures have been identified as key topics in the SETAC working group on LCA in building and construction [Kotaji *et al*], and can now be seen in recent developments of ISO 14044, ISO 14025 (Environmental Life Cycle Assessment and Environmental Declaration) and in ISO 15686 (Service Life Planning) as well as in the work on "Sustainability in Building construction" in ISO/TC59/SC17 and in CEN BT/174.

The main characteristic of the modular approaches is that the pre-usage life cycle stages and the use phase are initially not aggregated into one common environmental profile. With this as a basis, the reference service life can be modified into a more case- and application-specific estimated service life, and all processes taking place during the use stage and the service duration of the product itself can be adapted to that specific situation. While offering these advantages, it must be made aware of some difficulties. While moving potentials for adaptation to the generator of case specific environmental performance information, the responsibility to create reasonable scenarios and to identify relevant sources for performance requirements is also transferred.

KEYWORDS
Service Life, Life Performance, Performance Requirements, Declaration, Assessment

1 INTRODUCTION – SERVICE LIFE PLANNING AND LIFE PERFORMANCE

One of the guiding ideas behind establishing a concept for service life planning (SLP) was to enable planners to design buildings that meet or exceed established performance requirements throughout the service life of that building. With that, designers and manufacturers are facing new tasks to be handled as part of their design profession. The service life of a building is defined with reference to performance requirements. Meanwhile, the service lives of a building and its components can not be regarded as being independent of each other, as buildings require a certain performance from materials and components in order to be able to meet performance requirements expressed at the building level. At the same time however, materials, components and systems that are part of a building usually themselves require provisions by the building, in especially concerning serviceability. Such interrelationships and the topic of aggregation and disaggregation of performance requirements are discussed in [Trinius & Sjöström].

In order to become operational, SLP relies on the availability of information concerning service life and life performance of products, components and systems. The only reasonable source of such information is the manufacturer of construction products, in the widest meaning of the term. When addressing durability, service life and life performance, producers may preferably apply different forms of testing, or they may rely on experience gained from product application. Information provided by a manufacturer can then only reflect the situation that is simulated in testing, or reflect the conditions under which experience has been gained. Further, manufacturers will prefer to be preparing a service life declaration or a life performance declaration that is established for their product as such, but as they cannot be held reliable for the way their products are applied in a design made by another organization, these declarations will disregard many of the possible conditions of application of the product. The provided information - which is dealt with as generic service life information - in fact however does not have a generic character, but reflects specific default conditions. Provided information is, strictly regarded, only valid for these situations. As the built-in context and the in-use conditions in a building may differ significantly from the conditions for which a manufacturer has determined service life information, the designer has to adapt given information where reasonable and necessary. For such adaptation ISO 15686 elaborates the so-called factor method, by which an established reference service life (RSL) [ISO 15686-2] can be modified into an estimated service life (ESL) [ISO 15686-1].

This paper does not lay out a new methodology for environmental assessment of buildings, but discusses how elements of service life planning can assist environmental assessment methods in their application of service life scenarios.

2 PERFORMANCE REQUIREMENTS

As the definition of service life makes reference to specific performance requirements, a specification and communication of a quantified service life duration can equally only be established when these requirements are known. A manufacturer may identify reasonable or typical performance references for which a service life can be given, he does however usually not have knowledge about project specific performance requirements, and can hence not give service life information in relation to such specific requirements. Meanwhile, and additionally to the building specific context of product application, performance requirements established specifically for a certain building may lead to service life estimations that differ significantly from reference values provided by manufacturers. While construction products usually are produced in accordance to product standards, rather than to specific user related performance requirements, this situation is to be accepted; but when service life and life performance information is communicated in terms of reference service life (RSL), the conditions under which given information is valid must be clearly stated. In line with the building design process and the choice of products, it is then the task of the designer to ensure that the “right” products are chosen, meaning that the selected design options are in line with the established building specific requirements.

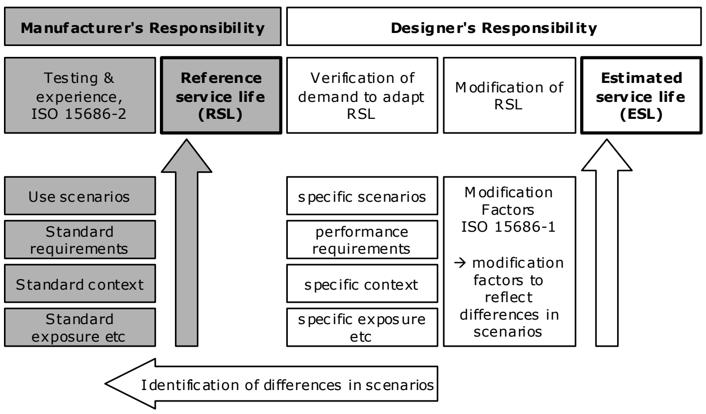
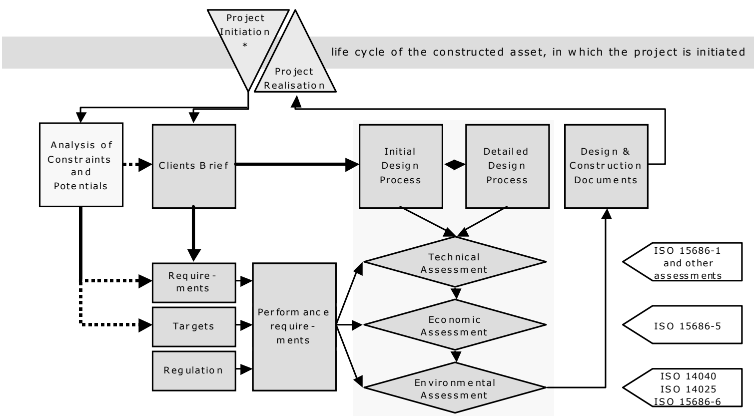


Figure 1: Responsibility of producers and designers in the establishment of reference service lives and estimated service lives

To enable this, the designer not only has to identify managerial performance requirements that reflect the demands of his clients, and that are relevant for the design decision at hand, but he additionally must enable himself to assess provided performance information in relation to these requirements. The demand to access and assess the validity of underlying scenarios and to adapt them where necessary may well go beyond the possibilities of design teams, both concerning effort and knowledge. The construction sector will need to prepare the presentation of RSL values that are valid for regionally typical applications, and to provide designers with guidance for the choice of typical modification factors.

A schematic illustration of an assessment procedure including life cycle aspects can be found in [ISO 15686-6], where the use of environmental information is introduced into a parallel assessment procedure with technical and economic life cycle assessments [Fig 2]. Further, the role of building industry and designers as responders to established performance requirements is discussed in [Trinius & Sjöström] and presented in [Fig 3]



* Project may be initiated at any point in the life cycle of the building

Figure 2: Procedure for integrated assessment of technical, economic and environmental assessment in relation to project specific performance requirements [ISO 15686-6].

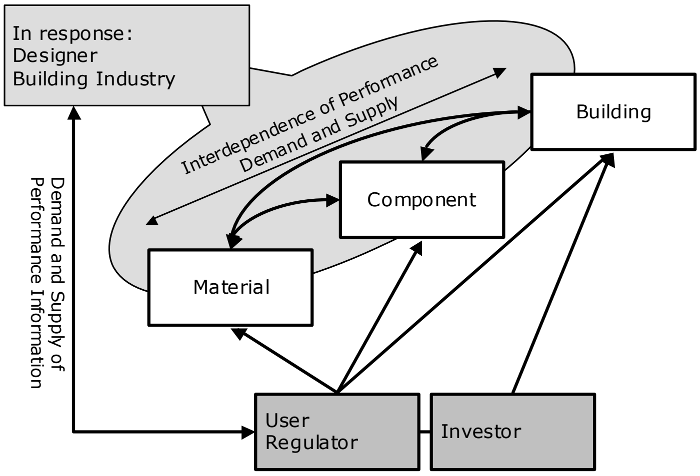


Figure 3: Performance requirements and communication of performance information [Trinius & Sjöström]. Performance demand and supply of materials, components and buildings is interdependent. Users, regulators and investors may express performance requirements. Designers and the building industry must respond to requirements and handle the interdependencies within their decision making.

While ISO 15686-6 establishes a procedure for the inclusion of environmental assessment into the concept of service life planning, the environmental assessment itself contains a significant module that is related to the use phase and with that to the service life of an assessed product or building. Thereby, ISO 15686-6 not only describes how to integrate environmental life cycle assessment into service life planning, but also illustrates how information from the service life planning process can be applied in the conduction of an Environmental Life Cycle Assessment (LCA). In order not to establish non-transparent assessment routines, ISO 15686-6, as well as the SETAC Working Group on LCA in Building and Construction [Kotaji *et. al.*] manifests that information on life cycle stages occurring after the completion of the building should not, at least not initially, be aggregated with information concerning earlier life cycle stages. Expressly, this means that information concerning the environmental impacts related to the end of life and the use phase should be presented separately from information on production related impacts. The advantage is seen in the possibility to adapt scenario-based information so that it reflects the current building context. Where aggregated data sets are applied, such adaptation is hampered, as first it would be necessary to identify and re-quantify the scenarios included in provided information. Also the current development of international standards in the field of environmental declaration [ISO14044] [ISO21930] and assessment of environmental performance of buildings [ISO21931] all follow this route. This is also one of the core adaptations in the development from ISO/TR 14025 to the full standard ISO 14025.

3 INTEGRATION OF STANDARDIZED MODULES

With the establishment of the European Construction Products Directive [CEC] and with performance based building regulation, as e.g. in the New Zealand building codes [Standards NZ], the construction sector faces to an increasing extent the demand to communicate information related to service life and performance aspects of construction products. As discussed above, manufacturers cannot provide the market with case specific information, and the designers are to handle the task to adapt information from the manufacturers so that it reflects the context of application in a specific building.

As at the same time the development and the application of environmental assessment tools is increasing significantly, the specific information on life performance of products also ought to be integrated into assessments of environmental performance of construction products and buildings. With the ongoing activities in international standardization, it is now possible to integrate the models, and to follow the provision of ISO standards throughout the entire procedure – from the generation and adaptation of service life information to the integration of service life modules into environmental declaration of products or environmental assessment of buildings.

A consequence of application of modular structures for assessments can be illustrated by a life cycle cost model that is currently under development for the German federal agency of construction and spatial planning [Kreißig *et.al.*]. This model is structured into two sequential steps, where the later step replaces standard assumption modules with project specific scenarios. Depending on the level of detailing in the design process, the designer can refine the models underlying the cost estimations. The underlying assumption is that the more refined the scenarios are, the more stable and case relevant the cost prognosis will be.

In general, assessment tools can benefit from modular approaches by clearly indicating, where the designer can gain more case specific assessment results by the adaptation of scenarios to the specific situation in the planned building. This also indicates, that modular approaches can enable the designer to tailor the system model underlying an assessment of environmental or economic building performance to the design he is currently making. With that, one general weakness of many environmental assessment tools may be improved, namely that the scenarios present in an assessment model have a significant influence on generated results, while they are not expressly indicated or justified, and can neither easily be changed by the user. Meanwhile it must be acknowledged that the availability of adaptation modules may make the application of assessment tools more difficult, as well as it may open for the consideration of "unrealistic" scenarios.

3.1 The missing module: derivation and identification of performance requirements

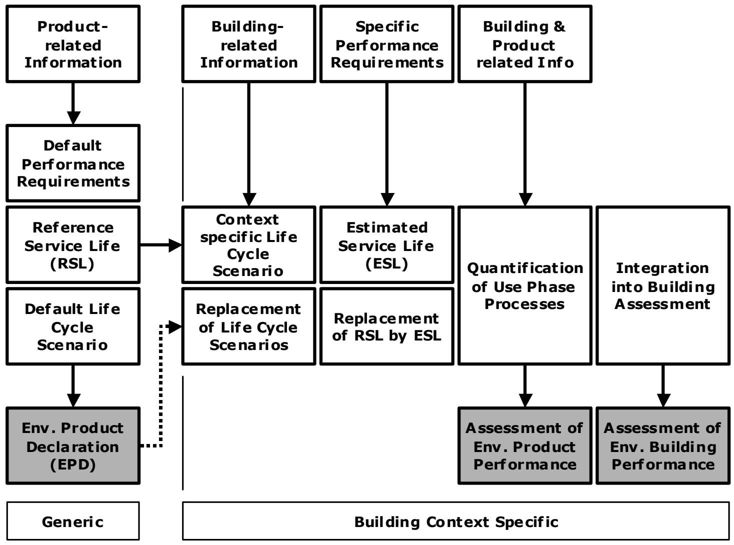
With the service life making reference to specific performance requirements, and with performance requirements being the reference point towards which design alternatives are assessed, the identification and communication of performance requirements becomes a key module within environmental assessment, service life planning and performance based building. Under the provisional title "Functionality Requirements and Serviceability", ISO has started to harmonize approaches in the field of performance aspects of buildings [ISO AWI 21933-1]. Further, ISO/TC59/SC14 is developing guidance on the inclusion of service life assessment and service life declarations in product standards [ISO AWI 15686-9]. Together, these work items are supposed to significantly contribute to a closing of the current gap. Following a modular structure, it is to be anticipated, that information based on these two emerging standards will be made available to means and tools that apply other standards in the field of sustainable construction and service life planning.

4 CONCLUSIONS

Sustainability assertions of construction products and entire buildings must consider aspects from the entire life cycle of the assessed object. Therefore, service life and performance throughout the service life are key aspects of concern. As assessments of a building design are supposed to reflect the current design rather than a fictitious combination of default-situation assessments of components, the manufacturers alone cannot provide the information needed for these assessments. However, information from the manufacturers constitutes the point of departure for the generation of case specific assessments.

A modular structure of the steps required to evolve from generic product information to project specific assessments illustrates two routes, one leading from reference service life through the route of service life planning into environmental assessment, the other route starting with environmental product declarations. As the later includes default scenarios, these need to be reviewed and where necessary replaced, in order to perform a relevant assessment. An adaptation of default scenarios to specific scenarios is possible only where provided information is not aggregated.

A modular structure and presentation of the content of modules is a precondition for efficient use of generic information, such as e.g. environmental product declarations. The route from generic to specific information is indicated in figure 4. For each of the established modules on that route, the procedures and information sources need to be clarified in order to enable stakeholders and actors to communicate necessary information and to adapt declared information and scenarios. Such adaptation strives to better relate the information to the specific situation in design and decision making



processes.

Figure 4: Modules of environmental performance assessment. The positioning of the modules also indicates the steps to be taken when moving from generic declarations to specific assessments.

With the approaches in current activities in international standardization, these procedures are under development. Tools for the assessment of environmental performance of buildings can now be developed in order to allow the designer to identify case specific requirements that then enable the environmental assessment to reflect these.

5 ACKNOWLEDGEMENTS

The discussions required to develop international standards usually take place among the experts present at work group meetings. As this paper reflects the current state in standardization and illustrates the benefit of putting several standardization items into direct context, also this paper to a large extent reflects discussions that took place mainly in ISO TC59 working groups.

6 REFERENCES

CEC, ‘Council directive of 21 December 1988 on the approximation of laws, regulations and administrative provisions of the Member States relating to construction products’, Council of the European Communities, 1988, Brussels. 89/106/EEC

ISO 14044, Draft International Standard developed by ISO/TC207 under preparation to restructure and replace ISO 14040 - 42

TT6-171, Modules of environmental performance assessment related to durability and service life, Wolfram Trinius

ISO/TR 14025, 'Environmental Labels and Declarations – Type III environmental declarations', ISO/TR 14025, International Standardization Organization, Geneva 2000

ISO/TC59/SC14, ISO subcommittee on Design Life preparing ISO 15686 standard series on Service Life Planning

ISO/TC59/SC17 ISO subcommittee on Sustainable Construction: Four standard documents are under preparation:
(1) General Principles and Terminology;
(2) Sustainability Indicators;
(3) Environmental Declaration of Building Products;
(4) Assessment of Environmental Performance.

ISO 15686-1, 'Buildings and Constructed Assets – Service Life Planning – General Principles', ISO 15686-1, International Standardization Organization, Geneva 2000

ISO 15686-2, 'Buildings and Constructed Assets – Service Life Planning – Service Life Prediction Procedures', ISO 15686-2, International Standardization Organization, Geneva 2001

ISO 15686-6, 'Buildings and Constructed Assets – Service Life Planning – Procedure for Considering Environmental Impacts', ISO 15686-6, International Standardization Organization, Geneva 2004

ISO AWI 15686-9, 'Buildings and Constructed Assets – Service Life Planning – Guide on the Inclusion of Requirements of Service Life Assessment and Service Life Declarations in Product Standards', Standard developed by ISO/TC59/SC14, International Standardization Organization, Geneva 2004

ISO AWI 21930,'Building Construction – Sustainability in Building Construction – Environmental Declaration of Building Products', Draft International Standard developed by ISO/TC59/SC17, International Standardization Organization, Geneva 2004

ISO AWI 21931, 'Building Construction – Sustainability in Building Construction – Assessment of Environmental Building Performance', Draft International Standard developed by ISO/TC59/SC17, International Standardization Organization, Geneva 2004

ISO AWI 21933-1, 'Buildings and Constructed Assets – Service Life Planning – Functionality Requirements and Serviceability', ISO AWI 21933-1, International Standardization Organization, Geneva 2004

Kreißig, J., Trinius, W. & Braune, A. 'Grundlagen für die Ermittlung von Lebenszyklus-Kosten von Gebäuden', 2. Zwischenbericht an das Bundesamt für Bauwesen und Raumordnung, Leinfelden-Echterdingen 2004

Kotaji, S., Schuurmans, A., Edwards, S., (Eds.) *Life-Cycle Assessment in Building and Construction*, Society of Environmental Toxicology and Chemistry 2003

Standards NZ, 'The New Zealand Building Code Handbook and Approved Documents', Building Industry Authority 1992, Standards New Zealand, Wellington 1992

Trinius, W., Sjöström, C., 'Service Life Planning and Performance Requirements', *Building Research & Information*, (March – April 2005) 33 (2), 173-181

Quality assessment of residential buildings for service-life

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ABSTRACT

After a period of new flats deficiency, the construction of apartment buildings in Slovenia has increased, recently, mainly thanks to the residential policy and better financing opportunities. Tenants and buyers of apartments ask for technical criteria and an assurance that they will get an adequate quality level to the money paid for the flats.

In this paper the methodology for assessment of long term quality level of residential buildings is described, based on the importance of durability of building and its components, life-cycle costs and sustainability of the whole project. Based on recent positive experiences with voluntary national quality labelling scheme (ZKG) for building products, components and services (over 80 ZKG labels issued since 1997) a similar approach was developed for quality assessment of residential buildings.

Criteria for evaluation of residential building quality are based on the principles of sustainability and defined by description of the influencing element and the corresponding set of indicators. For the determination of the particular element quality level, one or more indicators of either quantitative or qualitative form are used. The indicators allow up to five levels ranking of technical quality. The impact areas and their elements influence environmental, economic and/or social aspect of sustainability.

The following impact areas have been considered: building architecture, urbanism, building structure, building materials, HVAC systems, electric installation and intelligent systems, building physics, functionality and maintenance. Indicators are aggregated according to the area of impact. At the early stage a consensus-based method in the expert group was used as a technique for aggregated indicators weighting, later open discourses among the developers and participating stakeholders in residential buildings as well as polls were implemented for determination of the aggregated indicators weights in the national context of building sustainability. In conclusion, the perspectives for the up-take of the assessment method in the apartment buildings market are discussed.

The paper is based on the research project "Development of criteria for good building practice in residential buildings quality", co-financed by Slovenian Ministry of education, science and sports, Ministry of environment, spatial planning and energy (CRP 2001-2006) and 12 construction companies. The main beneficiaries are municipal housing funds in Slovenia and future apartment owners.

KEYWORDS

Residential buildings, quality labelling, indicators, weighting, sustainable building

1 INTRODUCTION

Perception of sustainable building has significantly changed in the last years. In the beginning, the technical criteria, like rational use of energy and limited resources, as well as reduction of the environmental impacts were focused. Recently, the non-technical topics, i.e. economic and social sustainability, gained the importance. According to the Agenda 21 on sustainable construction [CIB 1998] the strategies for sustainable construction should be compatible with the climate, the culture, building traditions, the level of industrial development and the nature of the building stock. Therefore the building sustainability can be evaluated only in the local conditions and consequently specific national criteria are needed.

A demand for the definition of technical quality of residential buildings has been expressed on the Slovenian real-estate market. This is expected to be the basis for quality ranking of residential buildings and for further the categorisation of apartments for buyers and tenants with different income. Many different informal ranking of apartment buildings are used when selling or renting the flats without any serious criteria. The term technical quality is used in this respect due to stressing the technical character of the considered indicators in opposite to the artistic and aesthetic criteria. In spite of this expression it is a common consensus of stakeholders that in this respect the building sustainability is considered.

2 QUALITY ASSESSMENT IN BUILDINGS

2.1 Quality labelling of construction products and services

The EU Directive CPD (89/106/EEC) prescribed the framework for indication of those characteristics of building products that influence the six essential requirements for the building. The national technical regulations impose more detailed requirements for technical quality of these products and buildings implementing them. These building products as well as many others, which are excluded from legal quality assessment procedures, influence the building sustainability through their life cycle. Although investors, designers, engineers, end-users and other stakeholders aim at using the building products with positive economic, environmental and/or social impact, they have currently no reliable criteria for distinguishing among different buildings products available on the market.

In order to support sustainable decision in selection of the building products the “Quality label in building and civil engineering” (i.e. ZKG label) was developed in Slovenia in 1997. ZKG is a voluntary quality-labelling scheme of national character, supported by Slovenian ministry of environment and spatial planning, Ministry of economy and Ministry of science. Quality labels are awarded to building products that receive sufficient number of points in the evaluation process. Criteria are developed by independent experts in the domain. The labelling scheme is well recognised on the Slovenian market and used, when the technical quality and sustainability of the building product has to be demonstrated.

The quality labelling of energy efficient windows is considered one of the ZKG’s most successful projects [Sijanec-Zavrl & Tomsic 2000]. In 1999-2002 labelled windows were directly eligible for state subsidies for energy efficiency. The label is awarded to energy efficient windows, produced by companies that concern about the environmentally friendly production and quality of their service. The evaluation scheme covers different criteria pondered according to their relevance to Slovenian situation. The evaluated criteria are: measurable technical criteria (U values, air-permeability, water tightness, mechanic characteristics), not measurable technical criteria (convenience of technical solution, functionality of the product) environmental criteria, efficiency and quality of production processes, satisfaction of buyers, fulfilment of the company’s business plan, global impact on the society and the environment. Since 1997 nearly 80 building products and services were awarded the

ZKG quality label. The quality assessment criteria were prepared by the group of experts on the domain. The criteria are updated with new findings on a yearly basis.

2.2 Sustainability assessment of residential buildings

Based on recent positive experiences with ZKG quality labelling a similar approach was used for quality assessment of residential buildings. The task of developing the criteria for sustainable, high quality residential building is far more complex than developing evaluation criteria for one building product. A successful plan of the building is a result of a fruitful communication among all stakeholders. Traditionally, in Slovenian practice, an architect, engineers, investors (developers or public bodies) are involved at the design stage, the end-user (buyer or tenant) have very low impact on the parameters of the building. Not much innovation is present at the design of residential building, mainly because of the costs reduction at the design and construction phase. When the apartments are put on the market they are often groundlessly described as high standard flats, promising high quality building, healthy environment, ecological materials and exceptional comfort. Therefore the criteria for quality assessment of residential buildings are needed in order to offer a basis for quality ranking of buildings. Thus the tenants and buyers of apartment could expect the adequate value for the money paid for the flats.

It was a common conclusion of the residential buildings stakeholders that the ZKG quality label for residential buildings could influence the real-estate market priorities, improve the overall technical quality of the buildings and thus contribute to the sustainable development of built environment. The potential users of the assessment methodology are end-users, i.e. buyers and tenants, investors (private and public), municipalities, architects, engineers, real estate agencies, building managers, the building industry and last but not least the ministry responsible for the strategic development plans in the residential domain.

Different evaluation methods for apartments and buildings used in the past, mainly based on simple measurable criteria and aimed at ranking of rental flats or determination of the real estate value. None of the methods used so far in Slovenia have a broader scope, taking into account the behaviour and impacts of the building through all phases of the life cycle. Successful German [Kuhndt & Liedtke 1999] and Danish [Steen Olsen 2000] methodologies for building sustainability evaluation are reported, potentially applicable also in the context described above.

3 METHODOLOGY

The criteria for evaluation of residential building quality are based on the principles of sustainability. The areas of building technical quality were defined and the list of concrete measurable or descriptive elements was developed. The impact areas and their elements influence environmental, economic and/or social aspect of sustainability.

3.1 Assumptions

Targeted product for the evaluation. The assessment methodology is intended for recently built apartment buildings, that passed the commissioning.

Aggregation level of criteria. The criteria should cover comprehensive and easily available information. The duplication of planners' work, regular quality control and commissioning during the evaluation process should be avoided in order to allow limited in time and affordable evaluation of the building technical quality. Quality assessment of the single product can be based mostly on very specific, measurable technical criteria. On the opposite, transferring the same principle on the building assessment shall create a need for analysis of a large number of technical criteria. Thus in our case more broad and complex criteria are desired, with measurable and/or well descriptive character, which can be apprised through a set of indicators.

Evaluation process. The methodology shall allow self-evaluation based on the defined criteria. The building investor project team itself should be able to describe the key technical elements that contribute to the overall sustainability of the building. Based on this information together with building plans, a site visit and a discussion with the investor and users, the assessment should be completed.

Expert group. The criteria should reflect the opinion and the needs in the local Slovenian framework. Therefore the expert group consists of well-recognised researchers from the specific technical areas. In order to integrate the view of the stakeholders in the criteria, the experts from the building industry, planning and construction companies, mayor investors, consumers associations and municipal housing funds participate in the definition of evaluation criteria and in the sensitivity analysis of the assessment methodology on the pilot buildings.

3.2 Areas and elements influencing sustainability

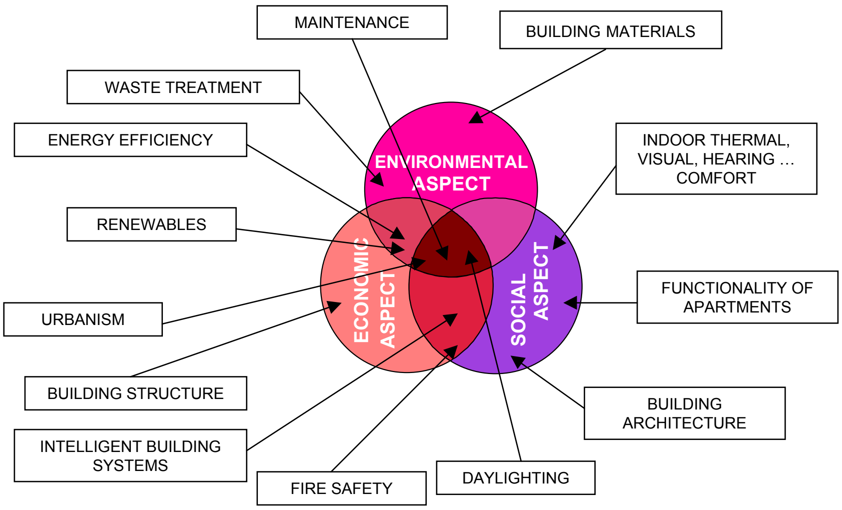


Figure 1. Some groups of elements influencing the sustainability of a residential building

The following impact areas have been considered: building architecture, urbanism, building structure, building materials, HVAC systems, electric installation and intelligent systems, building physics, functionality and maintenance. A list of evaluation elements has been prepared for each technical area. In the ‘Fig. 1’ some groups of the evaluation elements (like building architecture, functionality of apartments, energy efficiency) are shown together with their impact to the particular sustainability areas. Each evaluation element is followed by the description of criterion and the measurable or descriptive indicators for its appraisal [Table 1].

3.3 Indicators

The criteria are defined by description of the influencing element and the corresponding set of indicators. For the determination of the particular element quality level, one or more indicators of either quantitative or qualitative form are used. The indicators allow up to five levels ranking of technical quality. The compliance with the minimum technical requirements defined in the national regulation is assumed as *threshold* for further evaluation. Up to five additional quality levels are defined, from *poor* performance (low, but still acceptable quality) via *average* (business as usual), *expected* (additional effort mobilised) and *desired* performance (involves new technologies and additional costs) to the *target* performance (i.e. corresponding to the sustainable apartment buildings integrating latest technologies and solutions) [Sijanec Zavrl & Gumilar 2003]. The quality levels are converted into points, where 0 points correspond to *threshold* level, 1 to 5 points are given to other levels from *poor* to *target* level.

AREA	ELEMENT	CRITERIA	INDICATOR(S)					
Building physics, heat	Thermal insulation of building envelope	(a) U value	U value of wall, roof, windows, overall specific heat losses					
			THRESHOLD	POOR	AVERAGE	EXPECTED	DESIRED	TARGET
			Regulation	0.4, 0.2, 1.7, 0.5	0.4, 0.2, 1.5, 0.5	0.3, 0.18, 1.3, 0.45	0.25, 0.17, 1.1, 0.4	0.17, 0.15, 0,9, 0.3
Architecture	Parking places	(b) Availability, accessibility, safety	Number of parking places and safety level					
			THRESHOLD	POOR	AVERAGE	EXPECTED	DESIRED	TARGET
			Regulation	< 1 per apartment	Min. 1 per ap. + some for visitors, min 1 on the ground level	Min. 2 per ap. , security control		Min. 2 per ap. + 1 per ap. for visits, closed, private area
HVAC	Hot water preparation	(c) Participation of renewables	Percentage of RES used for hot water preparation					
			THRESHOLD	POOR	AVERAGE	EXPECTED	DESIRED	TARGET
			0%	0%	0%	50%	75%	100%
Building materials	Embedded energy	(d) Acceptability of building material in respect of grey energy	Level of acceptability					
			THRESHOLD	LOW	AVERAGE	EXPECTED	DESIRED	TARGET
			No requirements	Steel	Concrete	Brick masonry	Adobe bricks	Wood

Table 1. Example of criteria described with (a) several measurable indicators, (b) several descriptive indicators, (c) with a single measurable indicator and (d) with a single descriptive indicator.

3.4 Weighting

Indicators are aggregated according to the area of impact. A consensus-based method was selected as a technique for aggregated indicators weighting. An open discourse among the developers and participating stakeholders was used for the determination of aggregated indicators weights, supported by polls prepared for particular areas. The first part of the weight was determined within the expert group. The second part of the weight is based on the stakeholders' opinion about the relevance of particular criteria to the building quality. As the allocation of weights may be controversial, a cross section of stakeholders or persons with differing viewpoints should be included.

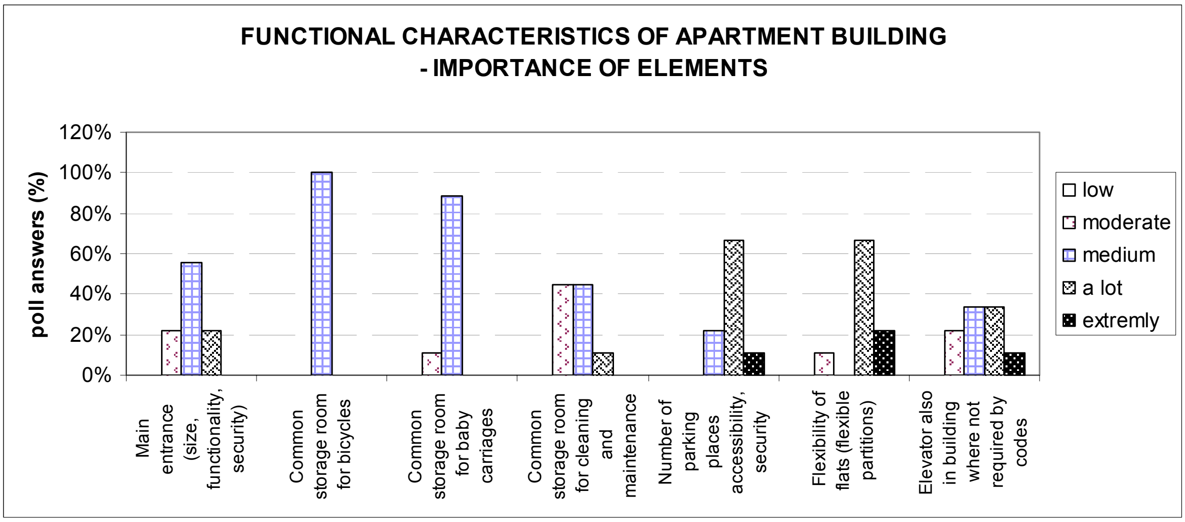


Figure 2. Importance of particular evaluation elements of "Functional characteristics of apartment building" field within the area "Architecture" - relevant for weighting of elements and evaluation of quality of residential building.

Importance of element		Importance of particular elements of "Functional characteristics of apartment building" field within the area "Architecture" and determination of weights Relative number of responses obtained in test poll among stakeholders				
Descriptive	Numeric	Common storage room for baby carriages, bicycles	Common storage room for cleaning and maintenance	Number of parking places accessibility, security	Flexibility of flats (flexible partitions)	Elevator also in building where not required by codes
	w_i	a_{i1}	a_{i2}	a_{i3}	a_{i4}	a_{i5}
Low	1	0%	0%	0%	0%	0%
Moderate	2	11%	44%	0%	11%	22%
Medium	3	89%	44%	22%	0%	33%
A lot	4	0%	11%	67%	67%	33%
Extremely	5	0%	0%	11%	22%	11%
Weight $\Sigma\, w_i\, a_{ij}$	[-]	2,89	2,67	3,89	4,00	3,33

Table 2. Determination of weights for some evaluation elements based on the answers form test poll among stakeholders (example for the area "Architecture", the field "Functional characteristics of apartment building").

The process of determination of the second part of weights is shown based on the poll in test sample of Slovenian stakeholders. The questionnaire for the area "Architecture" investigated the stakeholders' opinion about the importance of particular evaluation elements and relevance of these elements to building sustainability. The evaluation elements describing building architecture influence social part of building sustainability. In the 'Figure 2' the importance of some evaluation elements, relevant for building architecture, is presented. The polls focussing other areas of evaluation elements with impact to environmental and economic viewpoint of building sustainability, as shown in 'Fig. 1', are still in process. The weights for particular evaluation elements in the area "Architecture" are determined on the basis of the poll results as shown in 'Table 2'.

3.5 Expressing results of assessment

Once the experts evaluating particular technical area provide the descriptive values of indicators, these are converted into numeric values and pondered according to their importance for experts and other stakeholders. The final results are presented for each impact area separately, in terms of obtained points versus maximum number of points per area. Impact areas currently considered are: building architecture, building structure, building materials, HVAC systems, electric installation and intelligent systems, building physics and energy efficiency, functionality in maintenance. A multi-criteria presentation of the assessment results is highly important in order to clearly present the sustainability information per area. Further aggregation of points is considered as a negative since the quality of information may be lost.

4 RESULTS AND DISCUSSION

Currently, the first version of the criteria for the evaluation of building sustainability was prepared and tested on sample buildings. Consequently some findings were summarized.

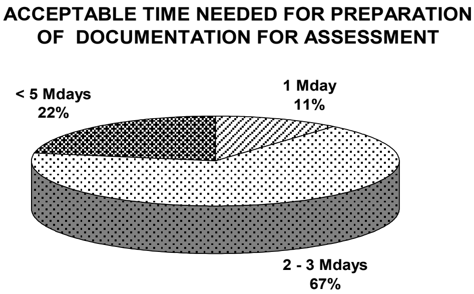
In spite of the fact that the indicators are relevant and well defined, there is a great risk that there will be no reliable information available to support the ranking. The situation occurs mainly because of the fact that many indicators are out of the scope of the building regulation and they are therefore rarely analysed in studies. In such a case the ranking is dependent on subjective judgement of the evaluator.

The proposed indicators may be compatible with both, existing and new coming regulation based on EU harmonised procedures. For instance: Eurocodes introduced a new approach in the structural design, taking into account the life-cycle and the economic view. But current national regulation and calculation procedures are still based on older principles and methods. Both methodologies may be applied, but the forms of proofs provided by these methods are different and therefore not directly applicable in evaluation of indicators foreseen.

For some criterions the regulation was recently updated (i.e. new regulation for thermal insulation, energy efficiency, thermal comfort, ventilation, indoor–air quality). Once the regulation is put into force, a *threshold* performance will be high, most likely correspondent to at least the *expected* performance. In such circumstances there is a very limited number of different technologies actually used. Not much space is left for ranking within the particular criterion.

Further investigation will be needed for the final definition of the criteria for evaluation of technical quality and/or sustainability. Further effort is put in sensitivity analysis of weighting of aggregated indicators and the reliability and objectivity of the evaluation.

The opinion of real-estate companies and investors (developers and housing funds) about the prospects for wide implementation of the method was also investigated. Acceptable additional cost of assessment of building seem quite low, i.e. 25% of respondents support 20 EUR per apartment, 37% (38%) consider 40 EUR (80 EUR) per apartment as acceptable costs. The opinion about the acceptable labour input for preparation of data (building owner) and for assessment itself (evaluator) showed similar, somehow discouraging, status, see ‘Fig.3’.



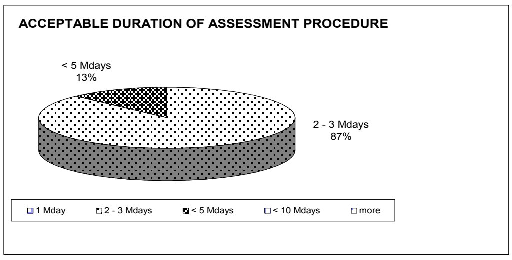


Figure 3. Stakeholders' opinion about the acceptable labour input for preparation of data (building owner) and for assessment process (evaluator), for middle size block with < 20 flats.

5 CONCLUSION

The currently developed methodology for assessment of building technical quality and sustainability proved that the well-established voluntary ZKG labelling scheme for products could be upgraded for evaluation of the building as a whole. Further investigation will be needed for the final definition of the locally acceptable assessment criteria for evaluation of technical quality and/or sustainability of residential buildings. In future more focus will be put on the weighting of aggregated indicators and on the reliability and objectivity of the evaluation.

6 ACKNOWLEDGEMENT

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7 REFERENCES

Sijanec Zavrl, M. & Gumilar, V. 2003, 'Znak kakovosti v graditeljstvu – tehnična kakovost stanovanjskih stavb' (Quality label in building and civil engineering – technical quality of residential buildings – in Slovene), Proc. of *Colloquium Država, državljani, stanovanja (State, citizens, apartments – in Slovene)*, Portorož, Slovenia, Nov. 13-14, 2003, pp. 224-231.

Sijanec Zavrl, M. & Tomsic, M. 2000, 'Supporting sustainable decision in selection of window technology', Conf. Proc. of International Conference Sustainable Building 2000, Maastricht, The Netherlands, 22-25 October 2000, Eds. C. Boonstra, R. Rovers, S. Pauwels. Aneas, Technical Publishers, Best, (NL), pp. 639-641.

CIB, 1998. *CIB and Sustainable Construction*, (http://www.cibworld.nl/pages/begin/ Pro2.html)

Steen Olsen, I. 2000, 'Evaluation of sustainable building in Denmark', Proc. of International Conference Sustainable Building 2000, Maastricht, The Netherlands, 22-25 October 2000, Eds. C. Boonstra, R. Rovers, S. Pauwels. Aneas, Technical Publ., Best, (NL), pp. 19-21639-641.

Kuhndt, M., Liedtke, C. 1999, *Die COMPASS – Methodik*, Wuppertal Papers, Nr. 97, Dec. 1999, ISSN 0949-5266, (http://www.wupperinst.org/Publikationen /WP/WP97.pdf).

Performing reliable LCAs for existing buildings A computer-based approach for inventory analysis

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ABSTRACT

The purpose of this paper is to present a new methodology and a software tool to carry out a Life Cycle Analysis (LCA) for existing buildings. The application of the approach and the tool are demonstrated by the example of a Life Cycle Analysis of an existing office building made of steel. In this paper are presented some results of the inventory step of this case study, that was established using the software tool.

The gathering of information about the material's composition of existing buildings as well as their quantitative analysis for a building represent not only a considerable amount of data but also a major problem when Life Cycle Analysis (especially inventory analysis according to ISO 14041) for buildings is realised. While performing LCA for new buildings is common today, the elaboration of a precise and rigorous inventory of construction's composition data for existing buildings is still quite difficult. Indeed, a building is the result of a combination of numerous elements with very varied origins, natures, procedures of placements and lifespan. Furthermore, the very heterogeneous as well as very often heteroclite character of a construction could make the quantification of the building's components difficult and complex, especially in cases where no information can be derived from drawings or LCA data already gathered. Thus, a computerised tool which allows to support the whole procedure of identification and gathering of construction's composition data for existing buildings is a valid tool for the assessment of building performance. In the approach illustrated, libraries of products which assist a rigorous organisation of the inputs and a downstream processing of the construction's composition data for existing buildings are used. An example of an office building made of steel points out how the tool can assist a reliable, rapid and precise audit.

KEYWORDS
Life Cycle Analysis, ISO 14041, database software tool, buildings

1 INTRODUCTION

One major objective of a Life Cycle Analysis (LCA) for buildings is to compare the environmental impacts of different buildings with similar functionalities. This requires the quantification of incoming and outgoing material and energy flows over the whole lifetime of a building and a further assessment of the environmental impacts associated with these flows. "Over the whole lifetime" means taking into account all phases beginning with the extraction of raw materials and their transformation into finished products, and ending with the demolition of the building and the subsequent waste treatment and disposal.

The first step of a LCA is the collection of data and the compilation of an inventory which comprises a listing of all materials entering the building, also known as inventory analysis that is described in this work.

While performing a LCA for new buildings is common today, the compilation of a precise and comprehensive inventory for a subsequent LCA for *existing* buildings is still quite difficult. Indeed, a building is the result of an agglomeration of numerous elements with very different origins, characteristics, procedures of placement and lifespan. Furthermore, the very heterogeneous as well as very often heteroclite character of a construction can make the quantification of the building's components very difficult and complex, especially in cases where no information can be derived from construction drawings or from LCA data already gathered.

In the following, a computer tool, which allows to support the whole procedure of data collection for existing buildings, is presented. The tool is based on a system of several libraries which assist a rigorous organisation of the inputs as well as a downstream processing of the collected data.

2 AUDIT OF MATERIAL FLOWS ENTERING A BUILDING

The audit of material flows entering a building consists – within the context of a LCA – of an organised and structured collection of numerous and diverse data on all materials necessary to erect the studied building. This means that every constituent of the building has to be taken into account in order to identify the significant incoming elements. This allows to realise a correct selection of the kept elements for the subsequent LCA according to the cut-off rules defined in the standard NF EN ISO 14 041. However, considering the complexity and the amount of elements of a building, it would not be rational to apply the same level of detail in gathering information for every element of a construction. Indeed, a large number of elements have a negligible influence on the LCA final results because they constitute only a minor part of a building and are ecologically harmless. Depending on the importance of the element in the building, the element should/could then be entered into the database either with its precise measures – with feasible accuracy – or using ratios or estimations. The low relevance of an element has to be checked not only by its quantity or harmfulness to the environment but also by the combination of these factors.

The data necessary to perform the audit of a building for a subsequent LCA are based on elementary construction elements. The data collection for each element should include its dimensions (length, surface, volume), quantity, unit mass, composition (materials and their relative proportions), as well as the delivery mode to the construction site (quantity of packaging), losses during implementation etc.

The required data concerning the element's characteristics for the realisation of an audit of material flows entering a building are given in documents like tender enquiry, technical and price studies of the considered building, technical specifications and descriptions of products, reports, plans etc. However, the older the building, the more difficult it is to obtain these documents. It is then necessary to collect the missing data in situ or to generate them with generic data based on other studies, literature and expert judgement.

3 REQUIREMENTS FOR A SOFTWARE TOOL

Taking the considerations in section 2 into account, the use of a software tool seems important, in order to allow limiting the risk of errors that could potentially occur during the compilation of the inventory for the different construction elements. This further ensures results with a better reliablility and higher quality. To reach this quality target, the tool should minimise the followings errors:
- duplicate data collection that could e.g. be the result of an architectural complexity,
- errors due to the fact that characteristics of construction products are collected several times when these products are present several times in the building,
- errors during the aggregation of data for the same material, the same product, the same family of products, etc. and errors resulting from a too large amount of data.

In order to carry out a subsequent LCA of the building with the inventory, that was established using the software tool, the data have to be arranged in a way that they can be used by a LCA software.

The software tool should also contain functions allowing the application of the cut-off rules as defined in the standard NF EN ISO 14 041, in order to assess the relevance of the diverse data, and by considering only the representative part of the building so as to facilitate the subsequent LCA.

Furthermore, the structure and organisation of the data should allow to assess the data collection in such a way that it is possible to attribute to each construction material the data concerning each phase of its own life cycle. Indeed, as illustrated in Fig. 1, the quantity of a given material varies between the different phases. Mainly three quantities are distinguished:
- *Q* as the quantity of a construction product needed to build a construction part, that has to be taken into account for the manufacturing, transports from manufacture to buiding site and construction phases,
- *Q'* as the quantity of a construction product integrated into the building, that has to be taken into account for the phases of use and end of life (elimination or recycling) and
- *q* as the quantity of broken material, losses during the product integration works, etc. of the construction product, which has to be eleminated after the phase of product integration into the building.

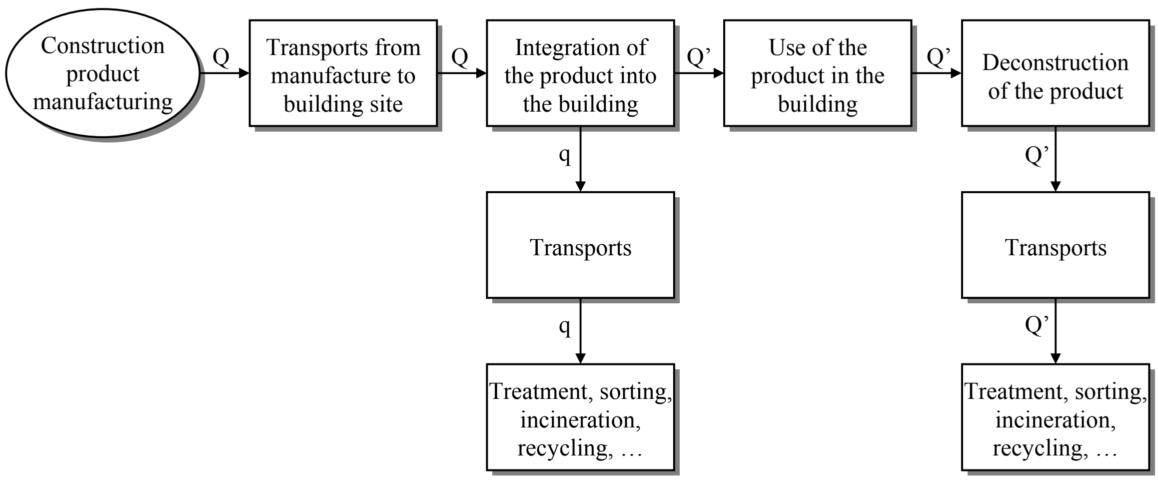


Figure 1. Flow chart of material flows for a construction product

It should be noted that the deconstruction phase of the product does not always correspond to the deconstruction phase of the building.

4 DEVELOPMENT OF THE SOFTWARE TOOL

The considerations in section 2 and section 3 have shown that for the compilation of an inventory of the products of a building a software tool is desirable to reduce the risk of errors and to support the selection of materials to be taken into account in a subsequent LCA. To meet these needs, the software *ACV-BatiBase* was developed. This tool provides a methodology to carry out a complete audit of the materials that enter construction sites and to aggregate and to further process these data.

The structure of the tool was designed in a way to meet all conditions mentioned above. As shown in Fig. 2, the data input is organised and supported by a system of libraries which are partially nested and linked with each other. This structure allows to limit the risks of errors and to easily modify or complete data in the libraries at any time. These modifications immediately affect the data of the ongoing audit.

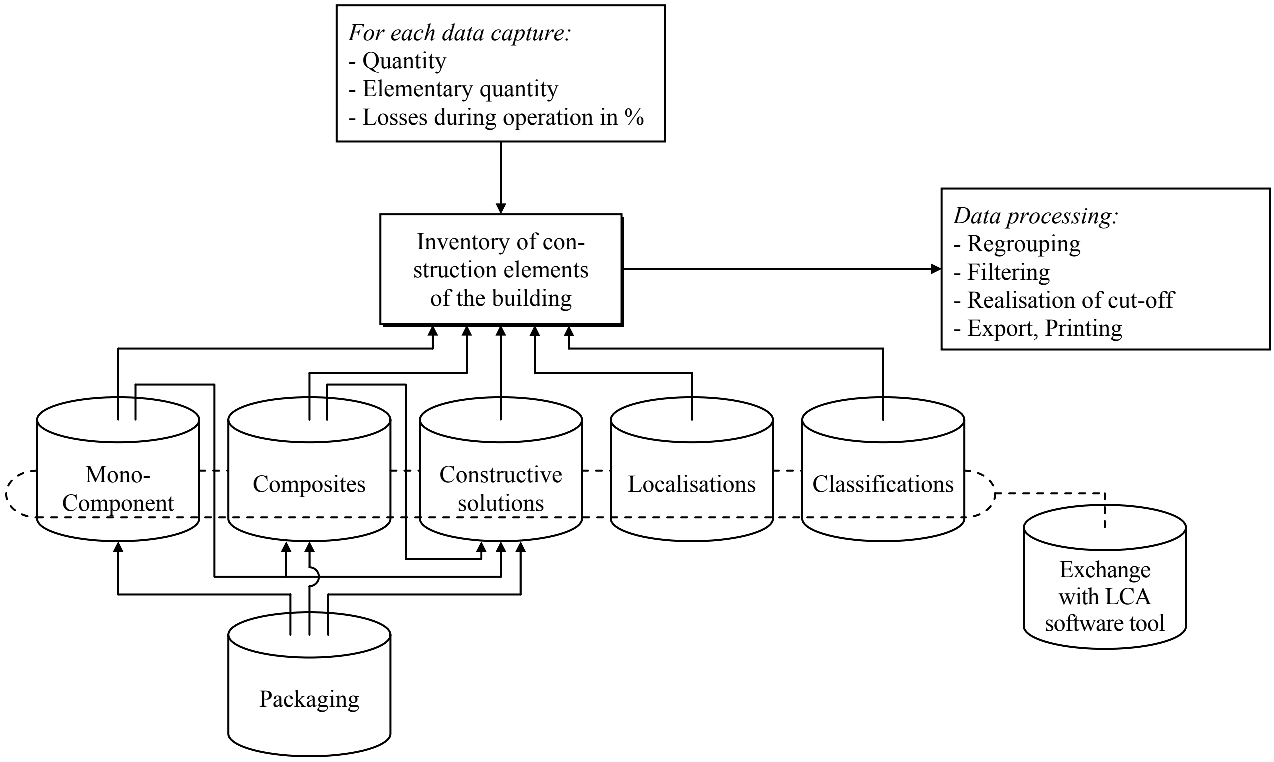


Figure 2. Structure of the software tool for auditing materials of buildings for further LCA

The software tool offers seven libraries that allow to associate with each data capture (see Eq. 1):

- monocomponent construction products i of masses m_i, to which one or several packagings j of masses M_j can be associated;
- composite construction products i with which monocomponents j with the quantities $N_{j/i}$ and masses m_j as well as packagings j of masses M_j are associated;
- constructive solutions i with which monocomponents j with quantities $N_{j/i}$ and masses m_j, composites j with the quantities $N_{j/i}$ that are themselves composed of monocomponents k of quantities $N_{k/j}$ and masses m_k, as well as packagings j of masses M_j are associated;
- a classification of the construction element stemming from the general nomenclature of product families named BatiBase, and adapted to the organisational needs of the data for a subsequent LCA. This classification allows to assign each acquisition to a structural group.
- a localisation of a component in the building in order to reduce the probability of error that would result from a complex architectural part;
- one correspondence allowing a data exchange with a LCA software.

The data capture is composed of a triplet of data (N_i, U_i and P_i) associated with a monocomponent, a composite or a constructive solution, as well as a localisation and a classification. N_i represents the number of times where the monocomponent, the composite or the constructive solution *i* is entered with the same unitary quantity U_i, the same localisation and classification during the acquisition in progress. P_i represents the percentage of broken material and losses during the product integration works of the monocomponent, composite or the constructive solution *i*.

The total mass of the building is then:

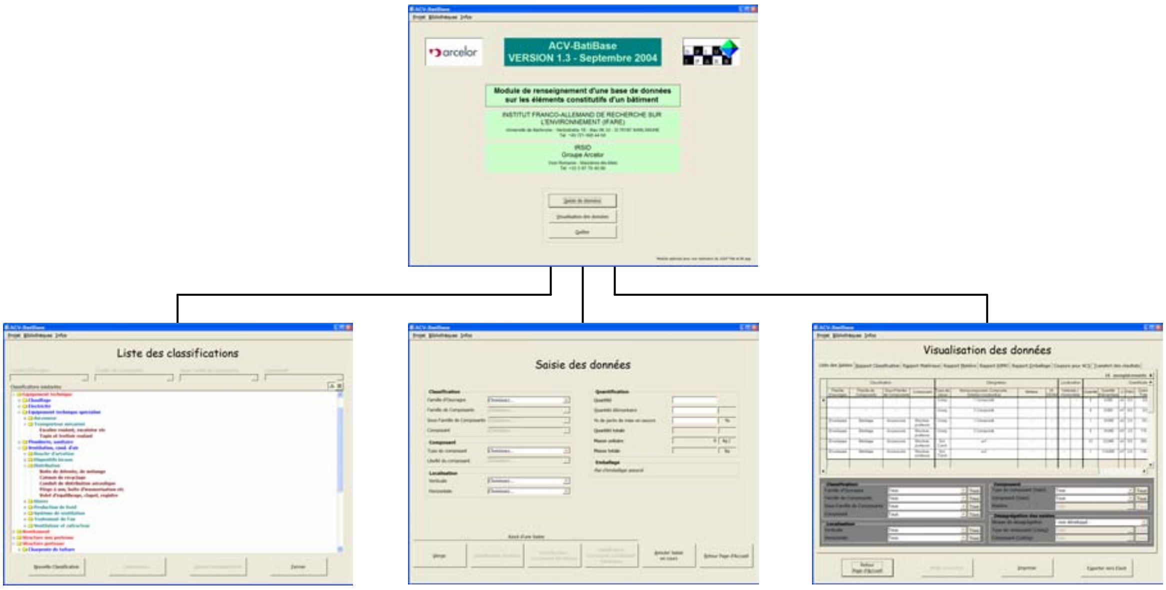
$$M = \sum_C \sum_L \left[\begin{array}{l} \sum_{i=1}^{MONO} N_i \cdot U_i \cdot (1+P_i) \cdot \left(m_i + \sum_{j=1}^{EMB_i} M_j \right) + \\ \sum_{i=1}^{COMP} N_i \cdot U_i \cdot (1+P_i) \cdot \left(\sum_{j=1}^{MONO_i} N_{j/i} \cdot m_j + \sum_{j=1}^{EMB_i} M_j \right) + \\ \sum_{i=1}^{SOLC} N_i \cdot U_i \cdot (1+P_i) \cdot \left(\sum_{j=1}^{MONO_i} N_{j/i} \cdot m_j + \sum_{j=1}^{COMP_i} \left(N_{j/i} \cdot \sum_{k=1}^{MONO_j} N_{k/j} \cdot m_k \right) + \sum_{j=1}^{EMB_i} M_j \right) \end{array} \right] \quad (1)$$

M : total mass of the building
C : set of all classifications
L : set of all localisations
$MONO$: number of monocomponents in L
$COMP$: number of composites in L
$SOLC$: number of constructive solutions in L
$MONO_i$: number of monocomponents in the composite or the constructive solution i
$COMP_i$: number of composite in the constructive solution i
EMB_i : number of packaging for a monocomponent, a composite or a constructive solution i
N_i : number of monocomponents or composites or constructive solutions i
U_i : unitary quantity of the monocomponent, composite or constructive solution i
P_i : broken and losses during the integration works of the monocomponent, composite or constructive solution i
m_i : mass of the monocomponent, composite or constructive solution i
M_j : mass of the packaging j
$N_{j/i}$: quantity of monocomponent j in the composite i or of monocomponent j in the constructive solution i or of composite j in the constructive solution i

The visualisation and further process of the data are organised in 8 sub-modules. One sub-module allows to form chosen groups, aggregations and disaggregations of the results. 5 sub-modules show the data with different fixed levels of aggregation and groups. 2 last sub-modules support the identification of the relevant incoming elements, with the purpose to justify the selection of the elements to be considered for the subsequent LCA according to the cut-off rule defined in the standard ISO 14 041, and to allow to organise data exchange with an LCA software.

The cut-offs are realised depending on the following parameters:
- the toxicity of elements;
- the elements not considered for the cut-off: it might be better for the assessment of a building not to consider all the very strongly dominating elements (such as for example the structure made of reinforced concrete or steel wich are anyway taken into account) during the realisation of a cut-off in order to have a more thoughtful cut-off in which elements and materials are considered whose mass quantities are small but important for LCA;
- the level of application of the cut-off: the cut-off can be realised on different data aggregation levels. A cut-off can be realised by comparing the aggregated quantities at the level of families of work, families of components, subfamilies of components, components or materials;
- the chosen percentage of cut-off.

The software tool offers three main modules for which the main characteristics are given in Fig. 3.



Libraries

Composition of the module
- List of classifications
- List of localisations
- List of monocomponents
- List of composites
- List of constructive solutions
- List of packaging
- Correspondence for a data exchange with an LCA software

Possible actions
- Addition of data
- Modification of existing data / further information for existing data
- Deletion of data

Data capture

Composition of the module
- Classification section
- Component section
- Localisation section
- Quantification section
- Packaging section

Possible actions
- Data captures
- Cancellation of the ongoing data capture

Visualisation and further processes

Composition of the module
- List of the input data
- Report of classifications
- Report of components
- Report of materials
- Report of broken parts and losses
- Report of packaging
- Cut-off for further LCA
- Exchange to an LCA software

Possible actions
- Aggregation, diasaggregation, grouping of the captured data
- Modification of captured data
- Cancellation of captured data
- Printings
- Export to Excel
- Fulfilment of cut-off for LCA

Figure 3. Structure and functions of the software tool ACV-BatiBase

5 EXAMPLE: AUDITING OF A STEEL-FRAMED OFFICE BUILDING

In the following, some results of the inventory step of a case study, that was established using the software tool, are presented.
The tool ACV-BatiBase has been implemented on a steel-framed building in Aubervilliers in France, (cf. Fig. 4). The structure under consideration is an office building designed and dimensioned in 1993 and 1994, then erected in 1995. With a total floor area of 9108 m², this construction includes a front of 116 m length and restrained but sensible enclosed spaces (maximum R+5 for the central part, R+4 for the side parts). Consisting of a variety of separable offices according to a regular framework of 1.35

meters, the number of vertical and horizontal possible movements of the workforce and the two major parts of the building separated by a road hold significant potential for different uses of the structure.

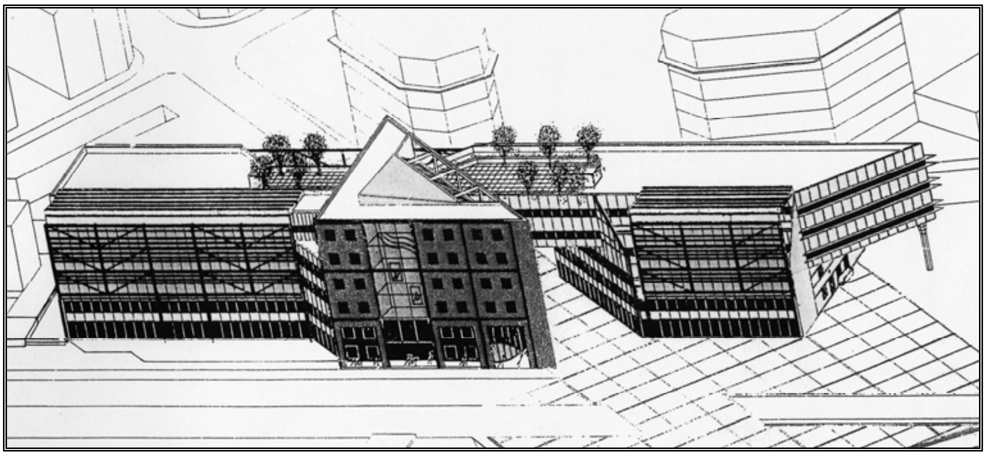


Figure 4. Representation of the building studied

Although quite a recent structure it proved somewhat difficult to obtain the necessary documents regarding the measurements of the building. The building audit thus had to be realised through data investigation campaigns and a variety of approaches including on-site measures in Aubervilliers, technical drawing measures, estimations from descriptive appraisals, surveys with building professionals (distributors, artisans, drawing offices, industrialists, etc.), etc.

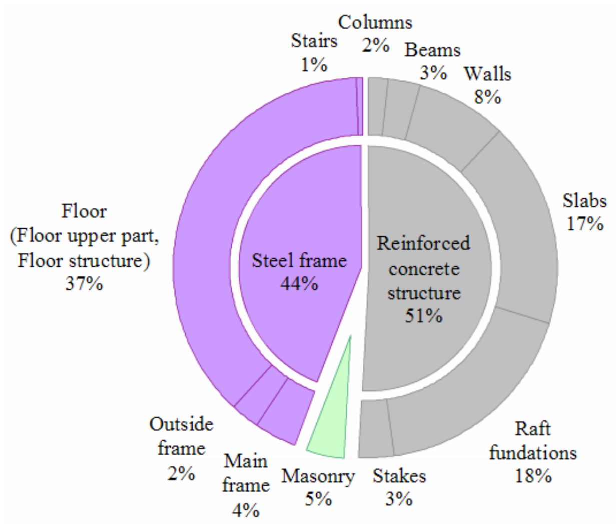


Figure 5. Composition of the structural part of the building (mass percentages)

About 12,000 data-sets have been collected during the inventory of the constitutive elements of the building studied. The data captures include about 530 monocomponents and about 100 different composite materials and constructive solutions. These data were represented in a matrix containing 319 classifications and 98 geographical addresses. Figure 5 illustrates the composition of the structural part of the building.

Given the architectural and constitutive complexity of the building, the use of *ACV-BatiBase* played an important role in the quality of the results. It permitted a reliable, rapid and precise audit through the structure of the captured data.

6 CONCLUSION

The realisation of a precise and comprehensive inventory of the components of a building for a subsequent Life Cycle Analysis can prove to be very hard, especially with regard to already existing buildings because of missing data. In such cases, remaking the whole or parts of the bill of quantities of the buildings might be necessary. The use of a computer tool that allows both supporting the whole procedure of data collection – thanks to the use of libraries of products as well as a rigorous organisation of the data acquisition – and the choice of materials to be considered for a subsequent Life Cycle Analysis through a downstream processing from the collected data, is of great importance. In order to meet these needs, the software « ACV-BatiBase » has been developed by the French-German Institute for Environmental Research (DFIU/IFARE). This tool provides a methodology to carry out a complete audit of the materials that enter construction sites and to aggregate and further process these data. This tool has been put into practice on a case study building and it could highlight the interest of the use of such a tool for an inventory of the components of a building. Indeed, the organisation of the collected data and the possibilities of processing data integrated in the tool made it possible to achieve a fast, precise and reliable audit and data processing.

7 ACKNOWLEDGMENTS

The French-German Institute for Environmental Research (DFIU/IFARE) thanks Arcelor Research for the financial support of the project called « *Développement d'un outil d'audit matière de bâtiments orienté Analyse de Cycle de Vie - Mise en pratique sur un bâtiment à ossature métallique partiellement constitué de produits composites contenant de l'acier* ».

8 REFERENCES

Raess, C., Schultmann, F., Rentz, O., 'Deconstruction of buildings with special consideration of structural steelworks', Proc. 24e Journées Sidérurgiques Internationales, Association Technique de la Sidérurgie Française, Paris, France, 10 11 December 2003, pp. 182-183.

Raess, C., Schultmann, F., Rentz, O., 'Dismantling of Structures with an Emphasis on Steel as a Building Material', Proc. Steel in sustainable construction, IISI, Luxembourg, 15-17 May 2002.

Schultmann, F., Rentz, O. 2001, 'Environment-oriented project scheduling for the dismantling of buildings', *OR Spektrum*, **23**, 51-78.

Schultmann, F., Rentz, O., 'Development of a Software Tool for the Optimal Dismantling and Recycling of Buildings', Proc. Symposium Computers in the Practice of Building and Civil Engineering, Lahti, Finland, 3-5 September 1997.

Advances in Methods for Service Life Prediction of Building Materials and Components – Final Report – Activities of the CIB W80

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ABSTRACT

The development of service life product standards and standardization of service life design and planning of buildings and constructed facilities are key elements for achieving "Sustainable Construction". The technical committee CIB W80 has focused over the past decade, on the development of knowledge in support of such standards and design methods and it has been the prime purveyor of fundamental information on the service life prediction of building materials and components. A final report is provided of work completed in the most recent work programme covering the period between 2002-2005. Advances in three areas of estimating the service life of building products components or systems are outlined. These include the factorial method, an engineering design approach and reliability-based methods. As well, a summary is given of work carried out in related areas, including environmental factors. Additional information is provided on failure mode effects analysis and its use in the building industry. Insights are provided into the collaborative efforts and related activities within ISO TC 59 SC14 – "Design life" and the Performance based building (PeBBu) thematic network initiative focused on "Construction materials" of the fifth EC framework on Competitive and Sustainable Growth

KEYWORDS

Building materials, building components, service life, sustainable construction

INTRODUCTION

Considerable work has been carried out in the area of service life prediction as requisite tools for helping assess long-term environmental effects, for maintenance management of infrastructure systems, such as roads, bridges, waterways, water distribution and waste-water removal systems, or indeed for maintenance of building envelope systems, envelope components and related materials. Increasingly, building material and component manufacturers are seeking systematic methods to assess the likely risk to premature deterioration of existing products given specific climatic effects, or the most vulnerable exposure conditions of new products in specified systems.

The CIB working commission (CIB W080), focused on reviewing methods of service life prediction of building materials and components, was created in September 1996. Prior to this, the joint CIB W080 / RILEM Committees (RILEM 71-PSL, 100-TSL, 140-TSL and 175-SLM) have been responsible for a preparing a series of useful working documents [e.g. Sneck 1982; Masters and Brandt 1989; Sjöström and Brandt 1991; Sjöström 1996; Lacasse and Vanier 1999; Vanier et al. 1999; Burn 2002; Daniotti 2003] as well as co-ordinating the efforts required to bring about nine international conferences related to durability and service life issues and the tenth taking place in Lyon, France. As has been stated previously [Lacasse and Sjöström 2003; Lacasse and Sjöström 2004], a substantial depth of knowledge to this field has been offered through this series of conferences and related symposia. However much of this information is not readily accessible to manufacturers of building materials and components, nor others who would likely benefit such as designers, specifiers, constructors, as well as asset and property managers. To enhance the knowledge derived from these collective efforts, the overriding task of the CIB W080 has been to, collate, review summarise and integrate information related to durability and service life from which guides and related types of information had been produced. The outgrowth of these efforts has culminated in contributions that enable practitioners to select the appropriate tools to predict service life.

This on-going exercise has in part been achieved through the committees' on-going support in developing the ISO standard 15686 prepared by the ISO TC59 SC14 on Design Life. The development of this standard has brought about broad recognition of the need to assess durability of components in all construction standards and this is being address in the EU and other national standards bodies. Additionally, collaborative efforts with the Performance based building initiative (PeBBu), currently in progress, are likewise increasing the outreach of work being carried out within the CIB W80 working commission (WC). This brief report provides an overview of activities completed in the period between 2002 and 2005, including a description of collaborative efforts and outreach activities.

ACTIVITIES OF THE CIB W80

Activities of CIB W80 extend back to 1978 and it has since seen different designations as it had been associated with the RILEM TC 71-PSL (Prediction of Service Life), 100- and 140-TSL and more recently the 175-SLM (Service Life Methodologies). The current work term will be completed as of the annual meeting in 2005 and the membership last met following the CIB World Building Congress – "Building for the Future" held in June 2004 in Toronto, Canada.

Collaborative efforts and outreach

The CIB W080 committee has always maintained links with related CIB committees such as the W060 on performance-based standards, W094 on Design for durability and W086 on Building pathology. However, since 1993 its outreach has been extended in large part due to its close collaboration with both the ISO technical activities, in particular the ISO TC 59 SC 14 (Design Life), the ISO TC 59 SC 17 (Sustainability in Building Construction), and the Performance based building (PeBBu) initiative undertaken within the context of the fifth EC framework for collaborative thematic research on

Competitive and Sustainable Growth. Brief overviews describing the nature of each collaborative effort are provided below.

Collaboration with ISO

The collaboration with ISO has been described in some detail in previous contributions [Lacasse and Sjöström 2003; Lacasse and Sjöström 2004], and the efforts regarding support on this standard within the WC are expected to continue for the foreseeable future. To date, the first three and the sixth part of the standard (ISO 15686 Buildings and Constructed Assets – Service Life Planning) have been published in successive years starting in 2000. A brief overview of the different parts is given below.

<u>Summary of standard and current progress</u>

Part 1 - General principles [ISO 2000] - describes the principles and procedures that apply to design, when planning the service life of buildings and constructed assets. It is important that the design stage includes systematic consideration of local conditions to ensure, with a high degree of probability, that the service life will be no less than the design life. The standard is applicable to both new constructions and the refurbishment of existing structures.

Part 2 - Service life prediction procedures [ISO 2001a] - of the standard is mainly based on the Service Life Methodology developed by Masters and Brandt [1989]. It describes a procedure that facilitates service life predictions of building components. The general framework, principles, and requirements for conducting and reporting such studies are given.

Part 3 - Performance audits and reviews [ISO 2002] - is concerned with ensuring the effective implementation of service life planning. It describes the approach and procedures to be applied to pre-briefing, briefing, design, and construction and, where required, the life care management and disposal of buildings and constructed assets to provide a reasonable assurance that measures necessary to achieve a satisfactory performance over time will be implemented.

Part 6 - Procedures for considering environmental impacts [ISO 2004] - describes how to assess, at the design stage, the potential environmental impacts of alternative designs of a constructed asset. It provides information on the interface between environmental life cycle assessment and service life planning (SLP).

A more detailed overview of the ISO 15686 series of standards and information on other parts of the standard are described in Sjöström et al. [2002]. Work undertaken within the CIB W080 directly supports development of the standard.

Activities related to the PeBBu

The EU thematic network PeBBu (Performance Based Building) was initiated in 2001 as part of the broader fifth EC framework for collaborative thematic research on Competitive and Sustainable Growth [Sjöström and Lair 2003]. The specific domain within the PeBBu on Construction Materials and Components (Domain 1) addresses issues related to the implementation and adoption of the performance-based standard on service life planning developed as ISO 15686. The motivation for this specific initiative emerged from the European Construction Products Directive (CPD) [Council of the European Communities 1988]. This directive specifies the "Essential (Performance) Requirements" that should be met of constructed works during their intended working life. This necessarily resulted in the need for establishing performance requirements on all building products from which the "life performance" (i.e. service life) of materials and products likewise has now to be assessed and declared.

A compendium of information regarding the project specifically related to the activities of CIB W080 focus on Domain 1 (Life Performance of Construction Materials and Components) can be obtianed from the PeBBu Web site (http://www.pebbu.nl/maincomponents/scientificdomains/domain1). These include for example:

- Specific Workplan: Objectives, scope activities, deliverables and milestones) of all tasks in the PeBBu programme.

- Domain members – list of participants, organisation, country, function
- Meetings and documents – Minutes to past meetings and other documents such as;
    - First Domain 1 report - provides information related to the first PeBBu Domain 1 Workshop (September 20, 2002, Gävle, Sweden) in which is presented the Work Programme, State-of-the-art on performance based building that summarises reports from a number of countries, and information on the need to develop service life data on building materials and components. Additionally, the applicability and development needs of the Factor Method and the Reference Service Life concept are presented, as are the domain research and research priorities. A comprehensive research overview is, however, not provided.
- Domain resources – source of documentation in which can be found, for example:
    - Domain 1: Synthesis Report Construction Materials and Components.
    - Performance Based Building – Some Implications on Construction Materials and Components [Sjöström and Lair 2003].
    - Operational Methods for Implementing Durability in Service Life Planning Frameworks [Lair et al. 2001].

The CIB W080 WC continues support of PeBBu network Domain 1 activities in that the WC's work programme has focused to further develop the Factor Method and the Reference Service Life concept as well as reporting on recent developments on the use of failure modes effects analysis (FMEA) for assessment of service life data for building materials and components [Lacasse and Sjöström 2003; Lacasse and Sjöström 2004]. Additional information related to the FMEA is offered in the subsequent section.

Recent work carried out in CIB W80

Publication of the results from the CIB W080 work programme that concluded in early spring 2002 have been summarised in two sets of documents:

(i.) Guide and Bibliography to Service Life and Durability Research [Jernberg et al. 2004]

(ii.) Performance Based Methods for Service Life Prediction [Hovde and Moser 2004]

Information on the service life and long-term performance of materials, components and system is a vital link in attaining sustainable and economically viable construction. Hence, it is hoped that in regard to the "Guide and Bibliography", this initial contribution will spur others working in this domain, in particular in the construction material manufacturing industry, to provide additional information on the durability of components, insights into their comportment in an assembly or system and related information on their performance and long-term performance.

Performance based methods for service life prediction focuses on two approaches to estimating service life: (i) the factor method and, (ii) the engineering method.

A brief overview of each document is offered below.

Guide and Bibliography to Service Life and Durability Research for Buildings and Components
This publication [Jernberg et al. 2004] covers work undertaken within the CIB Working Commission W080/RILEM Technical Committee 140-TSL on the prediction of service life of building materials and components during the period between 1991 and 1996, and as well, additional information subsequently provided in the period between 1997 and 2002. It was intended that this publication offer researchers and knowledgeable practitioners a useful guide to service life prediction. Essentially a primer that provides fundamental information related to methods of service life prediction, on environmental characterisation, and relevant to the performance and durability of construction materials.

The introduction offers background to the work, a general overview of the document and the terminology used in the text. Following which, the document is divided into four parts, the first of which provides an overview of service life and durability issues; it is an introduction to the topic. The second part represents a significant contribution on environmental characterisation, previously published by the Norwegian Institute for Air Research in 1996. The third part encompasses various contributions specifically related to materials. Originally, it was thought that this part would provide basic information on material properties and the performance and durability of a broad range of construction materials including, cement-based materials and concrete, different metals including, steel, iron, aluminium, and polymer based materials and so on. Although this part does provide some extremely useful information on copper, natural stone, brick masonry, clay and wood construction materials, the broad list of materials originally intended has yet to be completed. Given the broad scope of the task, the work of providing an all-encompassing compendium of relevant information on all construction materials must be considered on going. The final part of the document provides an annotated bibliography that includes abstracts or summaries of works related to service life and durability, case studies as well as experimental work on materials, components and systems, based on the original document prepared by Grondin at the National Research Council Canada [Grondin 1993].

Performance Based Methods for Service Life Prediction
The report [Hovde and Moser 2004] is divided in two parts (A and B) the first of which focuses on the Factor method whereas the second describes the Engineering approach to service life estimation.

Part A — This report contains a state-of-the-art regarding development, evaluation and use of factor methods for service life prediction, in particular the factor method presented in ISO 15686 Part 1 [ISO 2000]. The initial two chapters provide a short introduction and background to the topic, respectively. In Chapter 2, some important activities that have taken place during the last decade are described and highlight the increased focus on sustainable development and the need for service life prediction tools. These activities are evident both internationally and on a national level. Chapter 3 contains some examples in which the need for service life prediction is explained. Four different tables expressing design lives for different categories of buildings are provided. In Chapter 4, general requirements for service life prediction methods are delineated. It is evident that the need for estimating service life may be dependant the purpose for carrying out an estimate and thus would require different levels of sophistication to achieve meaningful values. As well, it is shown that service life prediction is encumbered with considerable uncertainties in estimating factors affecting the service life of materials and components, and as such, it is not an exact science. Chapter 5 contains a description and explanation of different factor methods whereas Chapter 6 contains an evaluation of factor methods. Illustrative examples of application of the ISO factor method are given in Chapter 7 – the examples show how factor methods can be incorporated in design for durability and development of sustainable buildings. Specific application of the factor method for service life prediction of facades and windows is provided. Finally, Chapter 8 contains important aspects regarding further development of factor methods.

Part B — Provides a literature review and appraisal of the state of the art of the factor method. A basic “engineering” approach is described that can be applied to the factorial method for standard cases as well as to other service life prediction methods that employ mathematical relations for service life. As opposed to using simple numerical factors, as is done in the original factor method, this approach incorporates the use of probability density functions for factors as well as for estimating the service life of individual components to arrive at an overall estimate of a building system's service life. The density distributions are established using reliable and understandable engineering techniques applied in a systematic and straightforward manner. Three examples are shown to illustrate the proposed procedure for different basic equations and different quality of input data.

Review of recent Work program (2003-2005)

Following discussion within the group on a notional program developed in 2002 [Lacasse and Sjöström 2003] it was decided that emphasis would be placed on: (i) the use of FMEA (failure modes effects analysis), and; (ii) further development of information related to obtaining Reference Service Life data for building components (products). No new information has been provided in regard to Reference service life that was not previously reported [Lacasse and Sjöström 2004]. Hence the following will concern providing information on recent work related to FMEA.

Failure mode effects analysis
The method of failure modes and effects analysis (FMEA) was developed and used in the aircraft industry as early as the 1960's [Lair 2003] as a means to help ensure adequate levels of systems reliability and maintainability during the production phase given the many different components that potentially could cause failure in modern aircraft. For building construction, it was suggested as a useful tool for the curtain wall industry to help mitigate risk of premature failure arising from problems that might be encountered not only over the course of fabrication, but also during installation [Layzell 1997, Layzell and Ledbetter 1998].

The applicability of FMEA to help assess the durability of building materials and components was first suggested by Lair and Le Teno [1999] and thereafter further refined by Lair [2000]. FMEA is used to understand the functionality and hierarchy of building elements including the interrelation among the different systems (building envelope, structure) sub-systems (cladding, windows, doors, roofing membrane) and other components of the building.

The use of a multifaceted operational approach has been suggested as a means of implementing the ISO standard on service life planning, ISO 15686, in practice [Lair et al. 2001, Sjöström et al. 2002]. The method is proposed for estimating the reference service life of a building component (RSLC) on the basis of service life data using the basic three items in the approach: (i) FMEA; (ii) data gathering and (iii) data analysis. Further refinement of the approach was provided by Lair and Chevalier [2002] in which the durability assessment methodology is described. The method is extended from design and implementation to an in-use assessment tool for maintenance management. Additional work by Talon et al. [2003] is reported on developing a simplified tool for decision making in both the design and in-use stages of building.

SUMMARY

The current work program was intended to develop additional information to promote the use of the ISO standard on design life, further refinement of methods of data gathering and analysis, in particular the use of the FMEA. The examples provided in the literature on the use of FMEA suggest that the method is readily adaptable to many building systems. Indeed, it has been demonstrated for certain roofing systems, insulated glass units, and for window and IG unit systems. Given that the method has been applied to many other building systems a comprehensive guide is being prepared on the use of FMEA for the building industry that would provide useful to other organizations seeking to undertake service life studies in a systemic fashion.

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REFERENCES

Burn, S. Ed., (2002), Proceedings of the 9th International Conference on the Durability of Building Materials and Components (9DBMC), Brisbane, Australia

Council of the European Communities (1988) Council directive of 21 December 1998 on the approximation of laws, regulations and administrative provisions of the Member States relating to construction products, 89/106/EEC

Daniotti, B., Ed. (2003), Proceedings of the International Workshop on Management of Durability in the Building Process, Milan, Italy (June)

Grondin, G.Y. (1993), Durability of Civil Engineering Structures: Annotated Bibliography, Internal Report, Institute for Research in Construction, National Research Council Canada, 646, pp. 78, July, 1993 (IRC-IR-646)

Hovde, P.J. (2003) Factor methods for service life prediction – A state of the art – Report of CIB W80/RILEM TC 175-SLM, Norwegian University of Science and Technology (NTNU), Dept. of Building and Construction Engineering, Trondheim, Norway.

Hovde, P. J. and Moser K. (2004), Performance Based Methods for Service Life Prediction, CIB Publication, 294, Rotterdam: International Council for Building Research, Studies and Documentation, pp. 107, (ISBN 90-6363-040-9)

ISO, International Organization for Standardization (2000) *ISO 15686-1: Buildings and Constructed Assets – Service Life Planning – Part 1: General Principles*

ISO, International Organization for Standardization (2001a) *ISO 15686-2: Buildings and Constructed Assets – Service Life Planning – Part 2: Service Life Prediction Procedures*

ISO, International Organization for Standardization (2001b) *ISO/WD 15686-8: Buildings and Constructed Assets – Service Life Planning – Part 8: Reference Service Life*

ISO, International Organization for Standardization (2002) *ISO 15686-3: Buildings and Constructed Assets – Service Life Planning – Part 3: Performance audits and reviews*

ISO, International Organization for Standardization (2004) *ISO 15686-6: Buildings and Constructed Assets – Service Life Planning – Part 6: Procedures for considering environmental impacts*

Jernberg, P.; Lacasse, M.A.; Haagenrud, S.E.; Sjöström, C. (2004), Guide and Bibliography to Service Life and Durability Research for Buildings and Components, CIB Publication, 295, Rotterdam: International Council for Building Research, Studies and Documentation, pp. 370, (ISBN: 90-6363-041-7)

Lacasse, M.A. and C. Sjöström (2003), Methods for service life prediction of building materials and components - recent activities of the CIB W80/RILEM 175-SLM, Proceedings of the International Workshop on Management of Durability in the Building Process (Milan, Italy, 6/25/2003), pp. 1-11, June 01, 2003 (NRCC-45691)

Lacasse, M.A. and C. Sjöström (2004), Recent advances in methods for service life prediction at building materials and components - an overview, Proceedings of the CIB World Building Congress 2004 - Conference on Research and Innovation (Toronto, Ontario, 5/2/2004), pp. 1-9, May 01, 2004 (NRCC-47030)

Layzell, J. P. (1997), Failure mode and effects analysis in the cladding industry, International Conference on Building Envelope Systems and Technology, ICBEST' 97, Bath, UK (April 1997), pp. 85-90

Layzell, J. P. and S. Ledbetter (1998), Failure mode and effects analysis applied to cladding systems - reducing the risk of failure, Building Research and Information, 26(6): 351-357

Lair, J. and J-F. Le Teno (1999), Durability assessment of building systems, in: Durability of Building Materials and Components 8, M. A. Lacasse and D. J. Vanier, Eds., Proceedings of the 8th International Conference on the Durability of Building Materials and Components, Vancouver, Canada (June), pp.1299-1308.

Lair, J. (2000), Évaluation de la durablité des systèmes constructifs du bâtiment, Thèse doctorale, Université Blaise Pascal – Clermont II, Clermont-Ferrand, France, 213 p.

Lair, J., J-L. Chevalier and J. Rilling (2001), Operational methods for implementing durability in service life planning frameworks, CIB World Building Congress, Wellington, New Zealand (April), 10 p.

Lair, J. and J-L. Chevalier (2002), Failure mode effect and criticality analysis for risk analysis and maintenance planning, 9th International Conference on the Durability of Building Materials and Components (March), Brisbane, Australia, 10 p.

Lair, J., (2003), Failure Mode Effects and Criticality Analysis (FMEA) – a tool for risk analysis and maintenance planning, Report submitted to the CIB W80/RILEM 175-SLM Service Life Methodologies, February, CSTB France, 15 p.

Masters, L.W. and Brandt, E. (1989) Systematic methodology for service life prediction of building materials and components, Materials & Structures/Matériaux et Const., 22, 385-392

Sjöström, C. and E. Brandt (1991), Collection of In-service Performance data: State-of-the-Art Approach by CIB W80 / RILEM 100-TSL, Materials and Structures, Vol. 24, No. 139.

Sjöström, C. Ed., (1996) Proceedings of the 7th International Conference on the Durability of Building Materials and Components, 7DBMC (Stockholm, Sweden, 5/19/1996)

Sjöström, C., Jernberg, P., Caluwaerts, P., Kelly, S., Haagenrud, S. and Chevalier, J-L., (2002) Implementation of the European Construction Products Directive via the ISO 15686 standards, Durability of Building Materials and Components 9, Brisbane Australia, 10 p.

Sjöström, C. and J. Lair (2003), Performance Based Building – Some implications on construction Materials and Components, Proceedings of the ILCDES 2003, Second International Symposium on Integrated Lifetime Engineering of Buildings and Civil Infrastructures, December 1-3, Kuopio, Finland, pp. 1001-1008.

Sneck, T., Ed., (1982), Activities on the Durability of Building Products: Background Study, Technical Res. Ctr. of Finland (VTT), Research Notes, Vols. 96 & 97, Espoo, Finland.

Talon, A., Lair, J. and Boissier D. (2003), Failure mode effect and criticality analysis to improve the quality of building products and processes, International Workshop on Management of Durability in the Building Process, Milano, Italy (June), 12 p.

Talon, A., Boissier, D. and Chevalier, J-L. (2003), Failure Mode, Effects and Criticality Analysis: risk analysis methodology for capitalization and use of experience and knowledge on building products degradations, Proc. 2nd International Symposium on Building Pathology, Durability and Rehabilitation, Lisbon, Portugal, 6-8 November 2003.

Talon, A., Boissier, D., Chevalier, J-L. and Hans, J. (2004), A methodological and graphical decision tool for evaluating building component failure, Proc. CIB World Building Congress 2004, Toronto, Canada, 2 7 may 2004.

Vanier, D.J.; Lacasse, M.A.; Payer, D.R. (1999), SL/AM-IT: A CD-ROM-based, interactive bibliography on service life and durability, 8th International Conference on Durability of Building Materials and Components (Vancouver, B.C. 5/31/99), Ottawa: Institute for Research in Construction.

Self-sufficiency in the provision of Indonesian construction materials

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ABSTRACT

In line with the plan of the Indonesian government to build and develop the infrastructure as well as building a million houses for middle low incomers, the provision of local construction material is strictly prerequisite. The purpose of building up the infrastructure (i.e. roads, dams, power plants, drainages and ports) in addition to mass houses is providing the job opportunity and increasing the public services demand.

The use of sustainable construction materials in Indonesian construction business is still considerably low, whereas the use of local material is rare. Business purposes with economical benefit considerations remain to be the major issue in the local construction firm.

Java as the most developed region in Indonesia customarily supplying woods for construction purposes from the tropical rainforests of Borneo and Sumatra, even though the use of these material are more durable but in the other hand its energy consuming and lack environment awareness. Furthermore its construction sand materials had been largely taken from the beach area, which brings about a number of serious environmental degradation. Likewise the construction material for Borneo, Sumatra and Eastern Indonesia most of the advanced production for material supplies such as cement, pre-fabricate concrete, steel and finishing material are mostly coming from Java region. This article aims to generate the knowledge vis-a-vis the possibility of self sufficiency and sustaining its construction material durability within local construction firm throughout the country.

KEYWORDS

Sustainablility, construction-industry, material, material supply , Indonesia

1 INTRODUCTION

1.1 Government Plans

In line with the new government plan to develop new infrastructure (road, dams, plants) and house for lower and middle class citizen in Indonesia, the construction company becoming one of the major factors involved to make it real. In Indonesia the construction sector always becoming first choice from central and local government for reducing the unemployment and reflecting the economics development. The demands to provide low and middle class houses undeniable, the accumulation for providing the lack of housing is nearly 5,9 million units in year 2003. Furthermore not to mention added by a necessity growth 800.000 thousand new house units as effect of the population growth [Kompas 2004]. Java as the most dense population in Indonesia have to provide a decent house for most of the population around the country, more than 55 percent of all Indonesian population live in this fourth biggest island in Indonesia [BPS 2002]

The new elected president promised in his campaign to provide cheap housing units for middle and low class citizen. This promise is very important for the shake of equalization and to minimize difference socialized gap. But in the other hand the Housing and Region Infrastructure as the government representative should prepare everything to make it real, the preparation not only the region housing planning, procurement and tendering but also the preparation of self sufficient for all of the region to provide its construction material. Java Island as the most dense population area should prepare the lack of raw material for the construction demands especially the lack of structural woods. Furthermore Borneo, Sumatera dan Eastern Indonesia should prepare the lack of production from process material such as asphalt, cement, pre-fabricate concrete, steel bar and steel plates. Those needs of the raw and process material for construction purposes definitely have to be fulfilled by the local government and its parties involved around the country. Providing local material and developing or giving the opportunity for local industry to proceed raw material becoming ready material will forcing a competitive price and less environment impacts.

1.2 Unbalance Material Supply in Indonesia

Some years lately the raw material inventories of natural resources especially wood in Java Island had been degraded, this is caused by the depletory of natural forest. The effect of political reformation spread out from district to the remote area and every villages in Java Island. The illegal logging which is either individually or organized progressively worsen the forest condition in this island, furthermore this is obvious that since Soeharto leadership orientation to developed a self sufficient of food resource and industry. Those policy and acts resulting wide spread deforestations in Java Island, since then the forest area only left 19,6 percent and this is by means that not all of this forest area consist of forest plants but some part represent a degraded forest plants.

In fact that most of the main construction material in this country had been supplied from any other places, some coming from the neighborhood area or known as local material but plenty of them had been taken from across the region or province and even across the island. For instant, most of the raw wood material for supplying houses roof structure in Java Island had been taken from Borneo. Asphalt as the main material ingredients for constructing roads and bridges in Sumatra have to be taken from Java (in fact that the raw material are coming from eastern part of this country) and even more importing from Singapore. In this case Suamtera does not have any asphalt production they only have a storage place.

Even some basic stone substance for roads foundation in Sumatera Island are abundance, but the fact still occurs that this material have to be taken away from district Merak (West Java). The basic aggregates as the main ingredients from concrete also had been taken from Java, The unavailability asphalt material mentioned because of industry processing petrify mining (mining milestone) yielding quality aggregate with delicate quality are seldom there, while in Java a lot of industry and investor capable to building up and penetrate the industry.

The Java Island densities reaching almost 60% from entire Indonesia resident result demand for the housing progressively insist on, as Kompas [2004] explained the housing demands almost reach 6,4 million houses which, the consequences most of this house demands have to build in Java. The necessity of raw material of construction will can be avoidance.

2 SUSTAINABLE COSNTRUCTION MATERIAL

According to CIRIA [1995] sustainability is an issue of great importance for society, and for the building sector. Sustainable for building and construction must be affordable, material durable, safe & healthy, energy-efficient and local material oriented. Sustainable buildings are designed, constructed, maintained, rehabilitated, and demolished with an emphasis throughout their life cycle on using natural resources efficiently, while also protecting global ecosystem [Forum explores sustainable building envelope materials 2000]. In terms of sustainability the use of local material to support the whole construction material needs also play an important role to force the more sustain in construction industry.

As part of the sustainable development process, the sustainable construction are required the durability of its building and the use of existing local resources in order to minimize the negative impact to the environment.

The building sectors are a great consumer of resources comparing with other sectors, the use of cement, milestone, wood, bricks and sand. As CIRIA [1995] stated that almost seventy five percent of the construction material comes from natural resources and as one of the riches country in terms of natural resources, mainly the Indonesian constructions industry including the material supplier and material assembly does not have any significant problem to supply its construction material especially the raw material.

2.1 Transportation Play Negative Impact for the Environment

As Plum [1977] explained that the distance traveled to the site is an additional consideration. Choosing materials that are local or regional makes good sense. Local materials are usually more economically viable and climatically appropriate, thereby contributing to natural energy conservation. Supporting a local or regional economy is equally important as a cultural contribution. Moreover the use of local material will decrease the time consuming and surely supported the less construction cost.

Type of transport	*CO_2* [g/ton km]	*SO_2* [g/ton km]	*NO_x* [g/ton km]
Diesel: road	120	0.1	1.9
Diesel: water	50	0.3	0.7
Diesel: rail	50	0.05	0.75

Table 1 Energy pollution from different forms of transport. Source, Fossdal [1995]

The construction material transportation account huge impact for the environment especially for its gas emission (table 1). Furthermore in Indonesia transportation surely play a negative impact as well in terms of air pollution, noise, energy consumption, global warming and acid gas emission, the across island transportation and road transportation with more than 50 km distance from quarry to the site will resulting a negative impact for the environment. Air pollution one of the negative impact, furthermore the noise nuisance during its transportation across city, passing trough the rural area and dense population in the city then the dust occurs during their transportation in the remote or village area. Moreover the water transportation, accept resulting 50 g/ton km (table 1) of CO_2 and amount of SO_2 and NO_x also tend to degraded the water sea quality in term of the poor knowledge for handling the oil machine used.

3 INDONESIAN CONSTRUCTION MATERIAL FACTS AND FIGURES

3.1 Indonesian Construction Industry

The Indonesia construction industry accounts for ten percent from GDP and employed in about four million people [BPS 2002]. The used of natural material resources in Indonesian construction industry consumed in more than ten trillion rupiah or more than six hundred and sixty six million pound sterling (1 £ = Rp 15.000), Iron concrete account the biggest in more than four hundred million pound sterling and the second one is woods in more than two hundred and sixty two million pound sterling respectively [BPS 2002]. And the indices trend up to year 2002 had shown an increasing number for all type of construction in Indonesia.

Indonesians total surface area is 1.9 million square km, with total populations in about 213.6 millions [Worldbank 2002]; furthermore Indonesian annual deforestation 1.2% reflected that the concern towards forest depletion and destruction was very low. A researcher from LIPI Umar [2004] giving an survey that Java forestry area only 7 percent left, which its have to 30 percent for the forest area in every region as a standard. This 7 percent is not enough to supply the water and fresh air demand as well as the wood industry in this island. Furthermore it is very bad the fact that 7 percent is not entirely full of forest plants but already being deforested by local citizen for farming purposes and illegal logging for business purposes.

Forests play an important role in Indonesia from economic, socio-cultural and ecological perspectives. Yet, in line with population and national economic growth, pressure on forest resources is constantly increasing. This is evident from the high deforestation levels (deforestation rate of approximately 3 million ha. per year for the past three years), moreover as GATRA [2002] observed the deforestation in Indonesia more than 57 million hectare, then half of all the destruction comes from permanent Indonesian forestry. Then 90 percent of the destruction resulted forest wood which are for construction and furniture purposes and 10 per cent for the destruction is from the extraction of mining industry.

3.2 Field Study

The un-self sufficient material supply within Indonesian construction industry seems not an important issue for the Indonesian government and other parties in the construction industry. The abundant of the natural resources to provide construction material needs tends to be a major reasons, the explanation below will measure the un-self sufficient material supply from the Indonesian construction industry-related.

Distance	*0-10 km local material* [%]	10-25 km [%]	>25 km *Across region/province* [%]	> 500 km *Across island* [%]	*Imported* [%]
Concrete	9.09	9.09	63.63	9.09	-

aggregates					
Cement		27.27	72.72	-	-
Bricks	72.72	18.18	9.09	-	-
Sources of Woods	-	-	-	100.00	-
Aluminums	10.00	70.00	20.00	-	-
Stone and rocks	9.09	54.54	27.27	9.09	-
Asphalts	9.09	18.18	63.63	9.09	-

Table 2. The main indonesian construction material; type and resources distance from the site

As can be observed in the table above, there are 7 main materials for construction projects that are mostly having a projects site in Java Island. The projects consist of road, water installation and building projects. Three of the materials such as concrete aggregates, cement and asphalt mostly being supplied from across region or across province. As shown in the table that the percentages obviously more than 50% of the site were coming from other region or other province, the distance is vary starting from 10 km up to 25 km away. Concrete aggregates as one of the biggest part of the construction material projects had been taken from specify quarry and being proceed in another place. Most of the concrete permanent batching plant planted in the capital city of the province in Java, and there were seldom movable batching plants in Java (only huge concrete companies provide a movable batching plants). The consequences that there are not plenty options to obtain concrete supply from the local concrete batching plants will be very difficult for construction site to provide its concrete supply in the others island except Java, even though Java having quite plenty batching plants, but the place is not scattered proportionally, only the province capital city provide the concrete batching plants. Another problem measure with the batching plants in the capital city, most of its sand raw material had been taken from beach area; this is because of the lack of good quality of raw sand material in the neighborhood (most of this sand beach problem occurs in Jakarta). As well as the aggregates, the cement production also have a limited place to produce, and mostly placed in Java Island and Sumatera, if the project conducted in Sulawesi or Papua the whole cement supply have to deliver from Java or Sumatera Island.

Based on the observation of the Asphalt, shown that the production/factory mostly in Java Island meanwhile its raw material had been supplied from Sulawesi or Batam (500-1000 km) from Java Island, Furthermore presently the development of the Indonesian infrastructure like road, bridges and dams will be constructed in Sumatera, Borneo and Eastern part of Indonesia (Sulawesi, Maluku and Papua). The complicated and unsustainable transportation will be measured for decades to come and this phenomenon needs a serious assessment and a real act from government and the local government to provide the demands of asphalt products.

Furthermore woods for supporting the construction industry in Indonesia still high, its account 25 – 50 percent of the whole construction progress. Mostly the use of timber for roof construction at the mass housing projects and scaffolding for medium multi-storey building, for the first purpose usually using a good quality of wood which has been supplied from another island (Borneo and Sumatera) meanwhile most of the mass housing projects are conducting in Java Island. The lack of good quality wood for supplying its woods material demands in Java, because of the governmental policy changes under Soeharto leadership and being worsen in the reform era (where the rural inhabitants feel free to exploits the national forest). Most of land use in Java had been changed from forest into farming, industrial area and housing.

Distance	*0-10 km local material* [%]	10-25 km [%]	>25 km *Across region/province* [%]	> 500 km *Across island* [%]	*Imported* [%]
Concrete	9.09	9.09	63.63	9.09	-

Steel	8.33	8.33	66.67	8.33	8.33

Table 3. The distance of pre-fabricate material from the factory or supplier to the site

Buildings based on prefabricated technology and require the minimum of concrete mixing, steel bar assembly, carpentry work, site clearance, and excavation. They produce less construction waste than any other available options, and they are also relatively easy to extend at a later date. The prefabricated components are easy to assemble on-site, thus reducing construction time, costs and more environmental friendly in terms of noise and dust.

Even though the percentage of steel and pre-fabricate concrete consumption in Indonesian construction industry barely small, but the trends for years to come will increase in line with the new government program. In fact that most of the pre-fabricate steel or concrete only in Java specifically in western part of Java (Banten province and Surrounding Jakarta) worsen the unsustainable use of construction material in Indonesia. Most of the infrastructure project will be concentrated in Eastern of Indonesia and Borneo Island, which less or none steel assembly and concrete fabricate.

4 CONCLUSION

Providing self-sufficient construction material in order to support sustainable development as a holistic view is not only the responsibilities of government and the construction related-industry. Local government and local people need to get involve and to be aware to the impact of not developing its local material and the impact occurs from its gas emission. A model to reflect the relationship between all parties in the Indonesian construction industry as explained as follows;

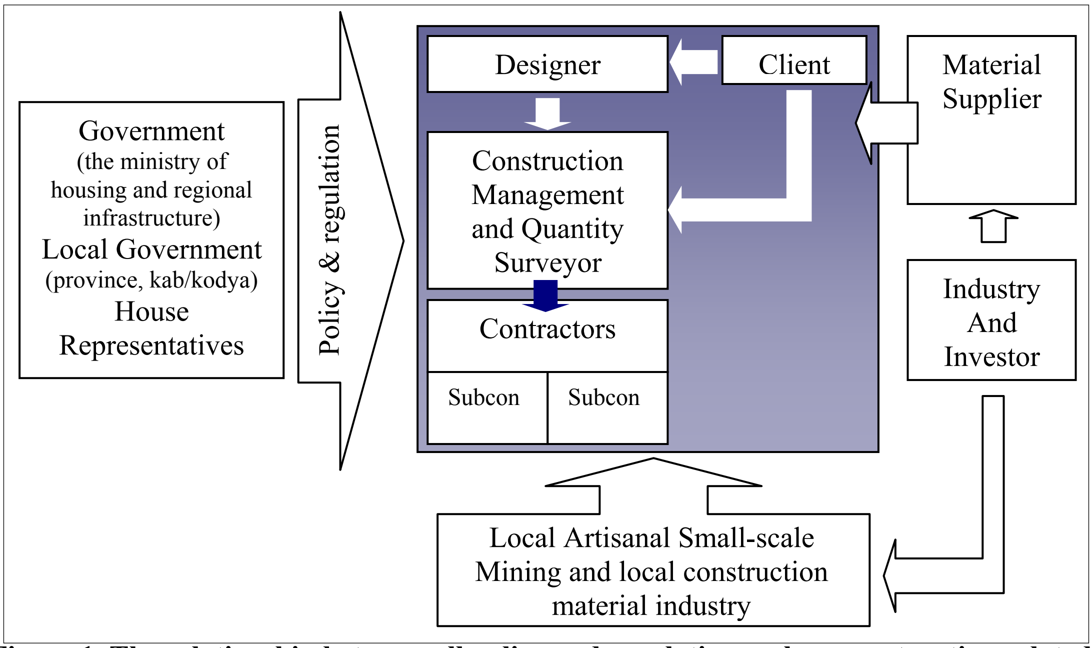

Figure 1. The relationship between all policy and regulation maker, construction related-industry, material supplier and Industrial/investor.

In order to generate the knowledge vis-a-vis the possibility of self sufficiency and sustaining its construction material durability within local construction firm throughout the country, can be huge possibility happened but this act should be supported by all the parties involved in this business. Accept the construction industry-related play for sustaining its self-sufficient construction material, the citizen also play an important part to make it happened.

The figure above will be breakdowned as follows;

Center Government,
Represent by the ministry of housing and regional infrastructure, which is directly involve (as a policy maker, supervisor or even a client) with every physical infrastructure development owned by the government;

- To promote and sosializing the importance of using local material in the construction related industry.
- To enforce and develop legislation for the government standards as an application to implement the use of local material around the country.
- To deliver a new policy and regulation which is more concentrate to the increasing of the use local materials and local production
  This policy and regulation has to deliver for designer, client, contractors and material supplier and producers.
- Fair law enforcement for violence in every aspect at the non-local utilizations (based of the policy and regulation).
- To support financial alternatives that give priority to the local industry and citizen to develop its capability for producing the local material assembly and processing.
- Providing advance knowledge and technology in order to develop local material processing industry (outside java) for creating more durable material (good quality). This act is to prevent the material supply competition from Java.

Local Government
Starting from municipal level (kabupaten/kotamadya) up to provincial level

- To deliver an uncomplicated bureaucracy to the investor for providing an opportunity in terms of developing a new business material production supply
- Develop the local construction material industry
- Providing a huge quantity of industrial forest for construction purposes
- To form a regional alliance (provincial) in order to prevent the across island construction material trading.
- To encourage local production (especialy outside Java Island) to develop more durable type of material in terms of quality (for preventing the material supply from Java).

House Representatives

- To support executive in terms of delivering policy and regulation which support the local industry development.

Industry/investor

- To open opportunity to penetrate its industry for providing construction material in the regional area, such as province and municipal (kotamadya/kabupaten).

Supplier

- To provide construction material with more local ingredients
- Commitment to supply only for local construction site
- Willingness not to supply the material across the island
- To form an alliance and agreement with other supplier, especially supplier outside its province area or outside the island.

Local Industry and ASM (Local Citizen)
Most of the local industries in construction material and the small-scale artisanal mining owed by the local citizens, the understanding of environmental awareness have to be implemented seriously.

Client
Having a clear understanding about the advantages to use local material as its main material ingredients. Continuously deducting all needs of using unnecessary material, especially which is not coming from local production or local natural resources.

Designer, Construction management and Quantity surveyor

- Delivering design with more local material ingredients
- To supervise and control the use of local material in every project site.
- Surveying local material and deliver as an input to the designer and contractors.
- In order to promote the use of local material, the industry should consider to take the the ISO 14000 into account.

Contractors and sub-contractors

As main the main part of the construction industry, contractors and sub-contractors tends to use local material rather than non-local material. Local material assessment has to be implemented in the first place in order to fulfill and maximize the use of good quality with competitive price.

- In order to promote the use of local material, the industry should consider to take the the ISO 14000 into account.
- To supprot local technologies in order to develop its local materials and quality to make it more durable.
- To encourage local production (esspecialy outside Java Island) to develop more durable type of material (for preventing the material supply from Java)

I believe that this all actions will provide more better self-sufficient construction material within Indonesian construction industry, and this acts will be valid if those parties have a good will to implement the suggested act. Furthermore the desires and needs of using a fancy and imported material should be reduce extensively as well.

5 REFERENCES

Fossdal S. 1995, *Energi og miljoregnskap for bygg*, NBI, Oslo.

Plum, N.M. 1977, *Ecology handbook*, Christian Ejlers Forlag, Kobenhavn

George O. 2000, 'Greening the construction supply chain in Singapore', *European Journal of Purchasing & Supply Management*, 6(2000), 195-206

Liliana Miranda & Liliana Marulanda, 'Sustainable construction in developing countries a peruvian perspective', *Agenda 21 for Sustainable Construction in Developing Countries, Latin America Position Paper*

Construction Industry Research and Information Association (CIRIA). 1995,'Environmental impact of materials', *SP 116, CIRIA , London*

Forum explores sustainable building envelope materials 2000, *June 2000 v103 i6 p16L*

Umar A.J. 2004, 'National forest preventing rest of the forest left', *Monday, October, 24th 2004, Indonesian National Science Association (LIPI)*, http://www.lipi.go.id/www/www.cgi?baca&1044592625

Kompas 2004,'Why we need a ministry of public housing?', *Saturday, October, 16th 2004, Daily national newspaper (bahasa Indonesia edition).*

Gatra 2002, 'Forest destruction', *Gatra, Agustus, 12th - 18th 2002 , national weekly magazine*

Hightechfinland 2002, 'Prefabricated solutions for better buildings', http://www.hightechfinland.com/2003/newmaterialsprocess/forestindustry/woodworld.html

BPS Indonesia 2002, 'Population of Indonesia by Province 1971, 1980, 1990, 1995 and 2000', *Indonesian central statistical bureau*, http://www.bps.go.id/sector/population/table1.shtml

BPS Indonesia 2002, 'Population 15 Years of Age and Over Who Worked by Main Industry 1997, 1998, 1999, 2000 and 2001', *Indonesian central statistical bureau*, http://www.bps.go.id/sector/employ/table2.shtml

BPS Indonesia 2002, 'Value of Selected Materials Used for Construction Projects, 1995-1998', *Indonesian central statistical bureau*, http://www.bps.go.id/sector/construct/table4.shtml

World Bank 2002,'Indonesian data profile', http://devdata.worldbank.org/external/CPProfile.asp?CCODE=IDN&PTYPE=CP

Durability of flat roofs: practical experience on service life and consequences on the maintenance strategy

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ABSTRACT

In the last twenty years, maintenance of buildings has grown in importance, especially in the European countries, due to the generalized aging of the real estate park and the necessity to preserve the built patrimony in terms of efficiency and productivity. As result, the cost maintenance is reaching and overcoming the investment cost related to new constructions: this implies the development of new professional competences and a new political vision of maintenance. It requires a global approach for the real estate maintenance.
In this context, the University of Applied Sciences of Southern Switzerland has been mandated by the Government of Canton Tessin (Switzerland) to develop a methodology for building management, which has as its objective the elaboration of a new maintenance plan for its school building pool.
The EPIQR program (Energy Performance Indoor Quality and Retrofit) and its related methodology served as a basis for the analysis. The method enables a fast but sufficiently accurate evaluation of the deterioration state of a building, and provides a cost estimation to bring it back to its original condition. Energy aspects can easily be taken into account with the elaboration of scenarios. Although the EPIQR program had been developed for buildings designed for habitation, its use was extended to school buildings.
Forty-two buildings were analysed in 2003, representing 38% of the 108 school buildings of Canton Tessin. The analysis of all the data showed that the flat roof was the element most subject to maintenance operations. The paper presents the results concerning the different roof typologies. Service life as well as life cost were established for the most significant roof typologies. The implications for the maintenance of the school building park are discussed. It highlights the necessity for scheduled maintenance based on structured information which has to be carefully collected from the past life of each building.
A roof retrofit is also an opportunity to improve roof insulation and integrate a photovoltaic power plant. Such an innovative solution has been devised and is presented as a roof retrofit possibility.

KEYWORDS

Durability, service life, life cost, deterioration, maintenance, flat roof

1 INTRODUCTION

The government of Canton Tessin holds and administers building assets which consists of more than 500 administrative and school buildings, for an estimated global value of about 1 billion euro. The real estate assets are located both in alpine and in sub-tropical climate zones in heavily urbanized and rural regions. In this area climate is characterized by sudden changes in temperature, heavy rainfall, long periods of drought and high summer temperatures. Therefore, under such climatic conditions, the tendency towards deterioration in buildings, especially in the envelope, increases. Moreover, in a situation where resources are limited, investment in maintenance tends to be ad hoc and only for urgent cases. In this context, the University of Applied Sciences of Southern Switzerland has been mandated by the Government of Canton Tessin to develop a methodology for building management, which has as its objective the elaboration of a new maintenance plan for its school building pool.

Assessment is based on visual inspection, condition appraisal and diagnosis of as many as 50 different elements, like for example: external infrastructures, roofs, floors, façades, structural elements, technical plants. The analysis of the deterioration and the building dimension coefficients form the basis for an estimation of the building renovation costs. This allows the manager of the facility to plan the financial requirements over a period of years, thus enabling him to discard the unpleasant process of breakdown intervention and to pass over to a programmed or preventive maintenance system.

Research consisted in visiting 42 buildings and diagnosing 50 construction elements per building. From the survey it emerged that the flat roof, apart from being one of the elements most under stress is also that showing the greatest differences in deterioration, some materials resisting a long time whereas others have a life-span of only a few years.

This article reports the results of the analysis regarding the life-span of flat roofs attained from the sample of school buildings examined and states the consequences for maintenance strategies and costs. Finally, a concrete example is presented where an opportunity was taken during a maintenance operation for an innovative refit where the restoration of a flat roof involved the integration of a thin layer of photovoltaic material in the waterproofing membrane.

2 METHOD

2.1 Research Method

The analysis of school facilities was carried out by means of EPIQR (Energy Performance and Indoor Environmental Quality Retrofit), which is a methodology developed within the framework of JOULE - a European research program, supported by the European Commission, DG XII for Science, Research and Development - that involved various organizations and experts from seven European countries (DK, F, D, GR, CH, NL and UK). This new method was developed to assist architects, engineers and other professionals who are involved in refurbishment or retrofitting (upgrading) work on apartment buildings [Flourentzos et al. 2000].

EPIQR is typically used for an overall assessment and diagnosis of the existing condition of residential buildings (deterioration state), the evaluation of various refurbishment and retrofit scenarios, and cost of induced works, in the preliminary stages of a project. The entire building is divided into 50 elements such as windows, façade, structure, roof, heating and cooling systems, electrical installations etc. Each element can be subdivided in different types. As a result of the building inspection the level of deterioration of each element is determined.
This is done by selecting a deterioration code “a, b, c, or d” defined in the method. The "a" code means that the element is in good condition, while the "d" code means that the element has deteriorated and needs replacement or extensive repair. b and c are intermediate states. These codes

have been defined accurately for all elements and can be observed during a building survey on the basis of the building model described in EPIQR.

Another current research project [Medimmo 2002-2004], supported by the Swiss innovation promotion agency (CTI) is developing an extension of the EPIQR method on administrative and school buildings. The results of the above research are obtained with a draft version of the new software tool referred to as EPIQR+.

2.2 The sample analysed

The buildings used for school activities in Canton Tessin total 108. The combined surface area is ca. 380,000 m^2 and the total volume is ca. 1,333,000 m^3. The average volume per building is around 16,000 m^3.

The real estate pool has not had a steady increase: almost 60% of the buildings were constructed in a relatively short space of time between 1970 and 1979. Our study has shown that this building surge has resulted in a significant increase in maintenance work over a short space of time.

The analysis was carried out on a representative sample of 42 school buildings (38% of the cantonal school pool). A reading of the results indicates that problems regarding deterioration or deficiencies were concentrated in the thermal insulation of the facades (65% of the sample), the roof (57%), the windows (44%) of the concrete façade [Teruzzi et al.2003 a,b] and the waterproofing of the flat roof (39%). The final element is of particular interest in that despite partial repairs and rebuilding it continues to present problems and can cause serious collateral damage.

3 RESULTS

3.1 Roofing typologies

Four typologies were present in the 72,800 m^2 of roofing studied (Fig. 1). Presented in order of appearance on the market they are: bituminized cloth consisting of 2 or 3 layers (23% of total surface area), synthetic membranes (55% of surface area), compact roofs (19% of surface area) and reverse roofs (3% of surface area).

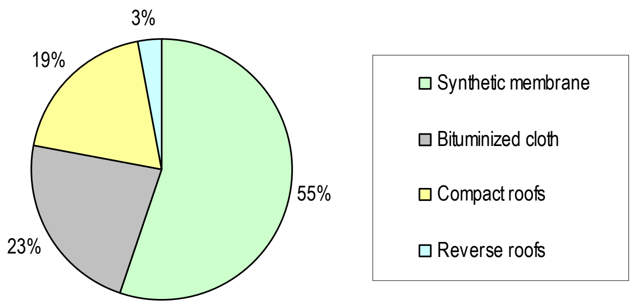

Figure 1. Distribution of the four typologies in the sample studied.

3.2 Damage typologies

The damages found relate to the type of construction. In compact and reverse roofs no damage was found. In roofs using bituminized cloth damage was found in 13.5% of its surface area (2,200 m^2). With the synthetic membrane technique 27% of the surface area was classed as either code c (significant but potentially reparable damage) or code d (significant damage requiring complete substitution of material). These definitions derive directly from the working method chosen and allow us the classify the elements studied.

The synthetic membrane technique became popular during the 1970s at a time when there was a sharp increase in school building construction in Tessin. The high percentage of damage found must be carefully considered in light of the possible repercussions on maintenance both technically and financially.

3.2.1 Condition of flat roofs with synthetic membranes

The 21.5% of the surface of the roofs examined (corresponding to 8,500 m^2) needed to be substituted as the damage was severe and beyond repair. Another 5.6% (2.200 m^2) was badly damaged and almost beyond repair. When a synthetic membrane breaks, water penetrates below the surface, soaks the insulation below (unless it is an extruding foam) and spreads over the supporting surface since the insulation is not glued but laid (Fig 2). The damage caused to the building is therefore always significant especially as the damage is usually noticed only after a certain delay.

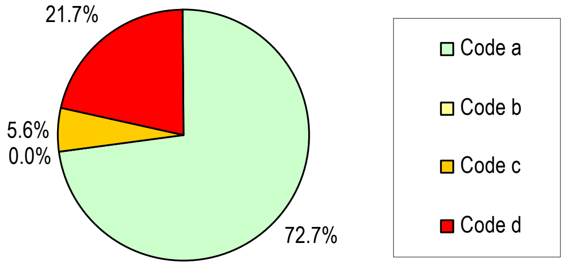

Figure 2. Distribution of the four deterioration codes on flat roofs with synthetic membranes.

The damage reported is essentially due to material shrinkage. Up to the 1980s, the waterproofing membrane was made of PVC. Because of the damage provoked by this phenomenon – which is illustrated in the pictures in Fig. 3 where it is possible to observe how the shrinkage of the membrane has caused tension around the fixing points – the manufacturers took steps to substitute the material and the cover is now made of flexible polyolefin membranes.

Figure 3. Examples of damage found on roofs with synthetic membranes.

Considering the flaws of this type of covering and its popularity – thanks to the large amount produced in the 70s as it was relatively cheap, easy to lay and easy on the eye – a study of the average life span of roofs with synthetic membranes was deemed necessary.

Assuming that the refurbishment of a roof occurs when the end of the life span has been reached, and with the date of construction and the date of the work on its repair to hand, it is possible to calculate the life spans of various roofs. Taking into account only roofs with synthetic membranes, for which there was sufficient data, it was possible to calculate the average life span as being

$$T = 23.8 \pm 5.5 \text{ years}$$

The graph in Fig 4 shows an accumulated statistic, that is, the ordinate of the single dots indicates the number of buildings with a life span lower or the same as that reported in abscissa. The continuous curve represents, in integrated form, the Gauss-distribution corresponding to the s and μ values obtained from the experimental data and then normalized. It can be observed that the curve follows the dots obtained from the study in a satisfactory way. Applying this kind of distribution, out of 100 buildings with first generation synthetic membranes, 16 would not exceed a life span of 18 years.

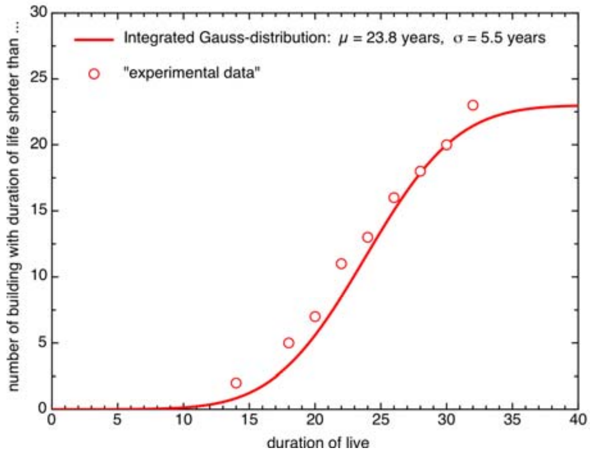

Figure 4. Life span vs. number of buildings.

The bibliography at our disposal, based on data from owners of large real estate assets (Confederation, UBS), generally indicates 30 years as being the upper limit for the average life span of a flat roof [Meyer et al.1995], thus confirming the validity of our study.

3.3 Consequences for maintenance strategy

The consequences of the damage resulting from this type of covering for the maintenance of the buildings have been and will be important. In this specific case, one aspect not to be underestimated is the concentration of maintenance work within a very short space of time. As can be observed in Fig 5, where the number of schools (solid line) and the number of c and d deterioration codes (bar) are reported as a function of the year of construction of a building, many cases of serious deterioration are to be found in buildings constructed between 1970 and 1979. The problem found on the roofs, which must be remedied quickly, has also certainly contributed towards this result causing what can be termed as a "maintenance peak".

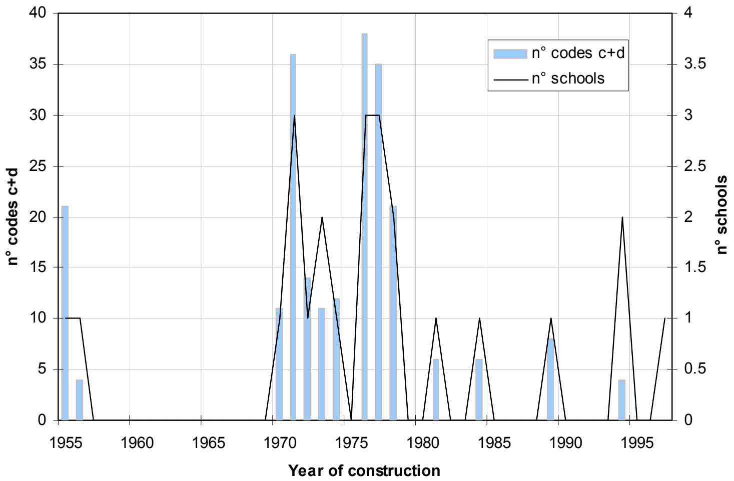

Figure 5. Number of buildings and c and d codes as a function of the year of construction.

On the basis of collected data it has been possible to evaluate the specific cost of the roof refurbishment (125 SFr/m^2). The total surface of roofs (four typologies) that needs to be renovated is about 13'000 m^2 only for the analyzed sample (42 buildings) that means a cost of about 1.6 million of Swiss Franc (~ 1 M€).

Assuming that the sample is sufficiently representative of the entire population of data, the maintenance costs for the roofs is extrapolated to all school buildings [Table 1].

	N. roofs	*Surfaces* [m^2]	*Surfaces* [%]	*Damaged roofs* [m^2]	*Estimated costs* Mio [SFr]
Roofs of the analyzed sample	42	72’800	38%	12’937	1,6
Roofs of all school buildings	108	191’523	100%	34'474 [5]	4,3

Table 1 – Estimated cost of maintenance related to the roofs of all school buildings.

This cost prevision is a very useful instrument for decision maker. In fact an adequate strategy can be developed in function of the needs of buildings and the financial availability.

This situation has some interesting repercussions on the technical front: on the one hand only those materials of proven endurance should be selected, whilst on the other hand the concentrated work load could allow speedy adaptation to current technical standards (opportunity maintenance). Moreover, from the energy point of view, the opportunity should be taken to carry out work aimed at reducing thermal loss through the roof so as to reduce– within the limits of economic sustainability – the energy requirements of these buildings as much one can. Where possible, renewable energy production technology could be integrated during refurbishment.

3.4 Opportunity maintenance: an example

When there is important maintenance work to be done, it is possible, in the case of flat roofs, to exploit these large surfaces for energy production. In fact, the thermal and mechanical features as well as the reliability of the new flexible polyolefin synthetic membranes are similar to those of the flexible amorphous-silicon photovoltaic solar modules [Chianese et al. 2003].

In a pilot project [D. Chianese et al., 2004] a flat roof was covered with a single ply roofing system based on Sarnafil T flexible polyolefin (FPO) membranes laminated together with flexible 15.4kW a-Si (amorphous silicon) triple-junction OEM 22-L-T UNI-SOLAR solar modules. The installation was integrated into the 960mq flat roof of the Centro Professionale di Trevano (CPT), situated near the University of Applied Sciences of Southern Switzerland. The power plant, which is shown in the picture of Fig.6, went into operation in December 2003.

Figure 6 Flexible polyolefines (FPO) membranes flat roof laminated together with a 15.4 kWp a-Si triple-junction PV power plant.

The modules are laminated on the membrane which is directly placed on the roof. The membrane is joined to the roof structure by means of hot air welding and then mechanically fastened. The thermal insulation of the roof is 15 to 18 cm thick, and the single elements have an inclination of 3° thus allowing the rain water to flow off. The thermal insulation does not allow ventilation of the modules as usually required by crystalline silicon PV modules. This leads to a heating of the modules and consequently to changes in the electrical operating PV parameters.

In the past, an improvement in the energy yield could be observed due to the thermal insulation of one string of an a-Si (single-junction) installation situated on the roof of the LEEE-TISO laboratory.

With this pilot installation it will be possible to analyze the behaviour and the energy yield of the 15.4kWp triple-junction thin-film amorphous silicon PV system, and verify in which way the better thermal behaviour of a-Si technologies can compensate for losses due to the quasi-horizontal roof integration. First results show an energy production higher (+15 %) than the one expected for the first eleven months of exposure.

4 CONCLUSIONS

By means of EPIQR method (Energy Performance Indoor Quality and Retrofit) 42 school buildings, selected out from a total of 108 buildings, have been analyzed inspecting and judging more than 50 different elements for their deterioration.

In this paper two main results have been shown. The first one is related to the analysis of the selected buildings combined with their age. In the close future an important number of maintenance actions may occur in a short period of time. This is the consequence of various factors, like the age, the choice of materials, the past maintenance.

A second important result consists in the role of the flat roofs in the deterioration of the analyzed buildings. From the collected data it was possible to evaluated the cost for their future maintenance to about 4.3 mio SFr. The analysis has put into evidence a major problem for the flat roof with synthetic membrane, for which it was possible to evaluated a mean life time of 24 years.
In the elaboration of the maintenance strategy it should be considered the possibility to improve the quality and the energy performance during a maintenance intervention. A pilot project has been

described as an example. It is a flat roof covered with a synthetic membrane in which flexible solar modules have been laminated for photovoltaic energy production (15,4 kW).

5 ACKNOWLEDGMENTS

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6 REFERENCES

Chianese, D., Realini, A. et al. 2003, 'Analysis of Weathered c-Si PV Modules'**,** Proc. 3rd World PV Solar Energy Conversion Conference, Osaka (J), 11-18 May 2003, vol. C, pp. 2922-2026.

Chianese, D. et al. 2004, 'Flat roof integration of a-Si triple-junction flexible modules', Proc. of the 19th European PV Solar Energy Conference, Paris (F), 7-11 June 2004

Flourentzos, F, K. Droutsa, K.B. Wittchen. 2000. EPIQR Software, *Energy & Buildings*, **31**, 129-136.

Kaehr, P., Salvatori, D., Pahud, D., Bernasconi, A., *AES-Rapporto di sintesi sullo stato di conservazione,* Technical report, SUPSI-DACD, Laboratorio Energia Ecologia Economia, Lugano

Medimmo. 2002-2004, Méthode de diagnostic et d'aide à la décision pour la rénovation des bâtiments et la gestion de parcs immobiliers CTI n. 5972.1 KTS

Meyer, P., Büchler, M., Christen, K.1995, *Vieillisement des éléments de construction et coût d'entretien*, Office fédéral des question conjoncturelles*,* Bern,

Teruzzi, T., Cadoni, E., Frigeri G. 2003a, 'Evaluation of the residual service life of existing concrete structures: a valuable tool for maintenance planning', Proc. 2nd International Rilem Workshop on "Life Prediction and Aging Management of Concrete Structures", 5-6 May 2003, Paris, France, pp. 207-216.

Teruzzi, T., Cadoni, E. 2003b, 'Application of a life-time management method on existing concrete structures of 25 scholastic facilities by probabilistic estimation of the residual service life', Proc. 2nd International Symposium on "Integrated Lifetime Engineering of Buildings and civil Infrastructures", 1-3 december 2003, Kuopio, Finland. pp. 385-390.

SERVICE LIFE ESTIMATION FOR COMPONENTS IN HISTORIC BUILDINGS

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ABSTRACT

This paper concerns a project undertaken by BRE for English Heritage, the UK government agency responsible for protecting and supporting the historic built heritage of the UK. It includes a brief description of results of an earlier project in which BRE undertook a literature review to determine how much published information was available on whole life costing and life cycle analysis of components in heritage buildings.

The first part of the project involved identifying a list of components and materials used in historic buildings. In order to do this it was necessary to categorise historic buildings on the basis of both age and use, which provided a structure in which the list of components could be developed.

The second phase of the project was to develop a methodology for estimating service life which took account of the specific issues arising in respect of heritage buildings. This drew on best practice guidance on service life planning in the ISO 15686 series of standards. While many of the principles were directly applicable it was necessary to adapt some of the techniques to take account of the very long design life requirements and also of heritage issues such as availability of skilled labour and appropriate historic materials. The other key issue to be taken into account was existing condition - a consequence of the previous exposure, maintenance regime and original quality of design and materials.

Finally the methodology had to be applied to the specific components and materials selected, and the results tabulated and reported in a way that captured the various considerations involved in attributing service lives, and the underlying assumptions and caveats.

This project represents an initial attempt to assess service lives of components and materials in historic buildings, and many lessons have been learnt in the course of the project. There is a clear need for such techniques to be developed in support of sustaining the built heritage and supporting the essential craft skills and materials and sustainable use of heritage buildings.

KEYWORDS

Heritage, components, residual service life , conservation

1 INTRODUCTION

This paper describes work undertaken by BRE for English Heritage (EH), a government Agency in the UK with responsibility for protecting and supporting the built heritage, including Ancient Monuments and buildings. The paper mainly concerns a project to develop service life planning methodology to use in estimating the residual service life of components in historic buildings. The paper also briefly summarises the results of an earlier project looking at the availability of Whole Life Costing and Life Cycle Analysis data on heritage buildings.

2 PROJECT BACKGROUND – EARLIER LITERATURE REVIEW

An earlier literature review undertaken by BRE for EH [Anderson and Bourke 2004] examined existing sources of information pertaining to whole life costing and life cycle assessment for historic buildings. The review specifically focused on buildings rather than monuments. It aimed to identify current areas of research that would be of relevance to historic buildings and identify gaps in the existing body of work on the subject. This analysis formed the basis for proposals for future research. Themes that were explored related to:

- valuation of historic buildings including income from listed buildings
- cost-benefit analysis (and contingent valuation analysis) of historic buildings;
- financial issues including VAT/ fiscal regimes, grants and insurance;
- costs and benefits of works to historic buildings concerning technical issues such as fire, disability access, acoustic performance and environmental control;
- embodied and operational life cycle assessment (LCA);
- indirect benefits/ externalities.

One of the conclusions of the report was that there was little available quantitative data on assessing the whole life costs of heritage buildings, although valuation methodologies had been proposed and to some degree tested. A further conclusion was that methodologies had not been developed for estimating residual service life in the context of heritage buildings. The literature review included a limited number of available case studies where quantitative analysis of whole life costs and /or environmental performance was available. These indicated that quantitative analysis would be both possible and useful in respect of historic buildings and to improve sustainability.

An example is shown in Fig 1. The building is Norton Park, in Edinburgh, Scotland [Atkins et al 2003]. It is a Grade II listed (lesser level of protection) former school that fell into disuse in the early 1990's. The limited use to which it had been put (workshops and storage) was insufficient to justify the increasingly onerous maintenance and heating costs. The redevelopment of Norton Park provided low-cost office accommodation for charities and voluntary organizations in a manner that meets high environmental and accessibility standards and minimizes energy use.

- The refurbishment of Norton Park cost a total of £2,837,000 including fees. This is equivalent to £756/m2- much cheaper than an equivalent new building.
- The income from the project is sufficient to staff and maintain the building including a sinking fund for long-term repairs.
- The case study demonstrates that the energy efficiency of the refurbished building is comparable to current "new build" best practise.

Figure 1 – Norton Park – an example of quantiative analysis of performance of an historic building

The rest of this paper addresses the initial development of heritage service life planning methodology and associated reviews of residual service lives of components.

3 EVOLUTION OF PROJECT METHODOLOGY

3.1 Overview

There were three major tasks in the project in chronological order:

1. Identify long list of historic building components, and refine this to a short list for further review
2. Review existing best practice guidance on service life planning and estimation of service lives, and propose a methodology to adapt this to a heritage building context
3. Test the methodology against a limited number of historic building components

3.2 Methodology and Conclusions from listing of historic building components

The list was developed from an existing listing by BRE of heritage components in religious buildings, which was extracted from surveys undertaken for the Ecclesiastical Insurance Office. This was amended and reviewed to cover the broad range of building types protected by English Heritage, on the basis of information in their publication Heritage Counts 2004 [EH 2004]. This includes data on the types and ages of protected heritage buildings (over 500,000 in UK, or 25% of the total stock), as well as an extensive review of the state of the built heritage in England.

On first view, the lists of components indicate substantial repetition of components within different types and ages of buildings – see Table 1 for example. However, in practice the type of materials, the relative durability of different specifications and the current condition of the components is likely to be very different. Key variables include whether the building is occupied – which will tend to improve its condition, and whether it was a “high” or “low”status building when first constructed. At various different ages materials such as timber or stone will have been available in varying qualities and costs, with imported materials typically more expensive, and reserved for high status buildings, for example. This also affects the current condition, in that some local, exotic or rare materials are unavailable or protected now, and therefore not available for repairs.

We therefore concluded that components needed to be considered in the context of the building in which they were installed. 10 representative component descriptions were identified for testing the methodology, on the basis of their criticality and covering a range of materials and building types.

Domestic Buildings	Industrial Buildings	Agricultural buildings
concrete	concrete	concrete
cast concrete	cast concrete	cast concrete
rock faces	flint infill between bricks for décor	flint infill between bricks for décor
cob	lime or cement render	lime or cement render
shiplap	lime or cement mortar	lime or cement mortar
lime or cement render	terracotta	wall tiles
lime or cement mortar	wall tiles	lime stucco
flint infill between bricks for décor	lime stucco	limewash
slate wall tiles	hollow glass blocks	cob
lime stucco	corrugated iron	tarred shiplap
hanging shingles	wattle and daub	wattle and daub
clay hanging tiles	cob	

Table 1 – Example of apparently similar historic walling materials between different building types

3.3 Principles of service life planning methodology applied to historic assets

The methodology was based on BS-ISO 15686-1 Building and constructed assets – service life planning – Part 1 General Principles. A key tenet of Service Life Planning (SLP) is that good design and material selection can be used to help manage the long term performance and cost implications with respect to the fabric of the built environment.

There needs to be a considerable shift in this perspective when considering the heritage scenario. Here one is wishing to manage a pre-existing estate. This entails investigating the performance and cost implications of decisions are set firmly in both the utilitarian aspects of achievability, functionality and health and safety but also the social/ethical considerations that are inextricably woven into the concept of 'preserving our heritage'.

From a service life planning perspective the starting point for heritage materials is not the reference service life of ISO 15686 but a residual (or remaining) service life. The objective then becomes how to extend this residual life as far as possible – maybe indefinitely. This can be at a level of the whole asset or individual components. It may entail sacrificing some components to ensure the survival of the whole asset.

A new building starts with a blank sheet. With heritage buildings (or any existing building) that sheet is already full of non-negotiable facts, past events and irreplaceable components. Expressing this in terms of the 'time-line' for a single component, say guttering we may have the scenario :-

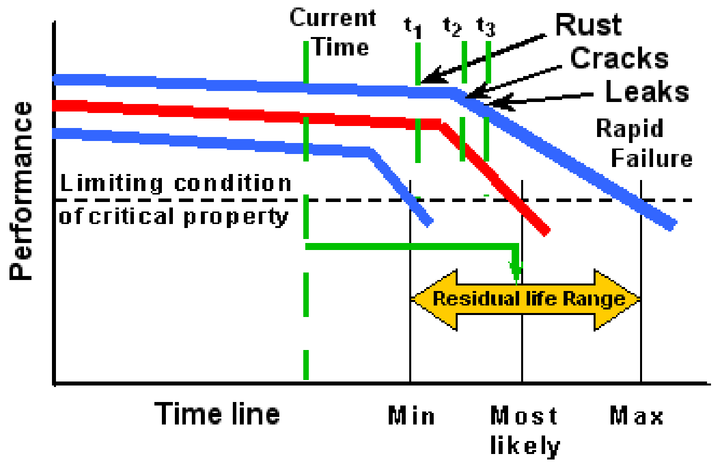

Figure 2: Example of a time line for cast iron gutters showing current time and three future events at t time t_1,t_2,t_3 and the range of residual service life times. The difference between the current performance and the limiting condition is proportional to the residual service life.

Traditionally in 'new' build there would be a choice of time-lines depending on the types of material available. With the heritage building that is already fixed, to say cast iron as in the above example. The position on the time line is also fixed by virtue of the prior history of the building and that component. The objective is to extend the life time of the component by suitable strategies whilst minimising the cost or loss for the desired result. This can be as simple as frequent cleaning or renewing a protective coating but may require more complicated strategies. Expressing this in terms of the factors described in ISO 15686-1 gives the analysis of "variables" in Table 2.

Agents and factors		**Relevant Conditions (examples)**	**New Build**	**Heritage**	**Comment**
Agent related to the inherent quality characteristics	A	Quality of components	Yes	No	Already chosen
	B	Design Level	Yes	No	As built
	C	Work execution level	Yes	No	As built
Environment	D	Indoor environment	Yes	Yes/No	Depends on type of asset
	E	Outdoor environment	Yes	No	Location as built
Operation Conditions	F	In use conditions	Yes	Yes/No	Depends on type of asset
	G	Maintenance level	Yes	Yes	

Table 2 – comparison of variable factors affecting service life in new build and heritage construction

With a modern building often replacement is seen as the immediate solution to restoring service life. This obviously is not possible in a heritage scenario unless the interest is to conserve the form of the building rather than the actual fabric of the building. Thus, conservation issues and ethics also close down options available to restore or extend the residual life of a component.

This critically depends on the concepts of a physical service life versus the functional service life. The former refers to the actual physical deterioration of the component whilst the latter to the job that the component fulfils. In common usage the latter is usually a practical or factual attribute of the component e.g., guttering transporting water. A gutter may fail to adequately transport water long before it physically disintegrates. Within the heritage context there is a requirement to extend the remit of functionality to include a greater range of characteristics. e.g., a component may be perceived to have failed in a functional sense if it is replaced by an identical modern equivalent – thus its functionality is embodied in its age.

4 DEVELOPMENT OF HERITAGE SLP METHOD

4.1 Overview

From the discussion above it is evident that SLP of heritage stock, even at the level of a single component, has many different facets. Thus, more than a single indicator is required to define the state of a component, the consequences and then the residual life time. The objective of this exercise is to determine out of the possible options available for a heritage component, and within the constriction of extending the residual service life, which gives best long term value or cost/benefit. In order to determine the best value a large number of factors need to be brought together within a decision matrix, as illustrated in Fig 3. These factors are analysed below.

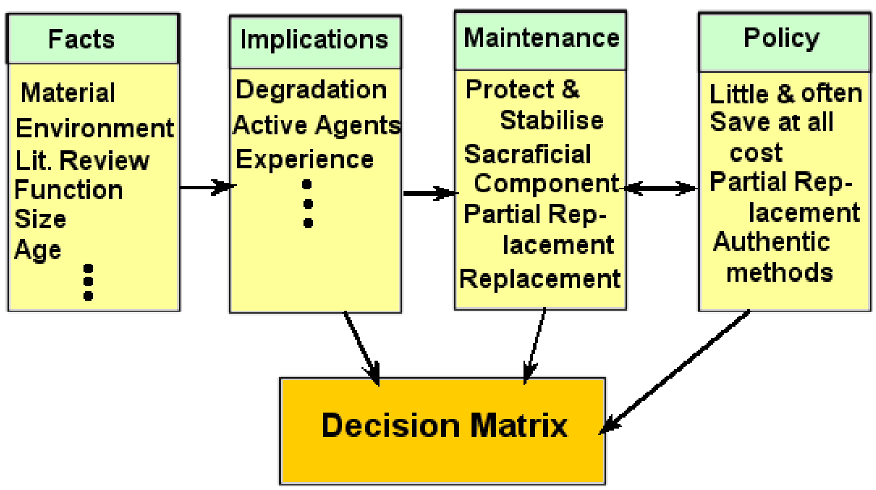

Figure 3 – Drivers for best value decision-making in heritage context

4.2 Inventory of Facts and Figures

The basic requirement for SLP is an inventory. This is essentially a list of facts about a component or a group of components. These should include :-

- Type of material (composition)
- Size
- Juxtaposition to other elements
- Location with respect to building
- Building
    - location (Rural, urban, adjacent buildings)
    - Current use

  - Historic use
  - Age
  - Grading
- Environmental information (inside, outside, temperature, humidity, pollutants)
- Function -
  - structural/non-structural,
  - decorative/non-decorative
  - sacrificial (render, paint coat, wallpaper, etc.)
- Age of component
- Provenance – extremely rare, rare, frequent, one of many
- Available technical literature on material, craftsmanship and construction method

4.3 Implications

From each of the facts and figures cited in the first stage (Section 4.2) of the assessment come a series of implications relating to the component under consideration. The critical issues are determining: -

- the **critical property threshold** of the component (Figure 2),
- the **shape of the time line**, via common experience, literature reviews, or standard product literature, (Figure 2)
- **Proximity** of the component performance to the critical threshold, and thus
- determining the **residual life** of the component
- Adjunct to these considerations are **identifying the active agents** which will cause the current condition to change,
- Identifying **cultural risk**, or loss of heritage functionality
- Identifying **physical risk**, or effect of loss of integrity

4.4 Maintenance Interventions

As seen above, of the agents and factors traditionally used in SLP only categories D to G remain which affect the environment and operation conditions. Whilst it is possible to alter the environment via enclosure or adjustment of the internal heating and ventilating regime it was assumed that these steps have been taken or are inordinately expensive. This only leaves adjustment of the operating conditions which primarily are maintenance regimes in terms of heritage objects. The broad options within this category are :-

- **Protection and stabilise**
- Create **sacrificial** sub-components
- **Partial replacement**
- **Total replacement**

Protection and stabilise is seen as slowing the rate of deterioration down to the minimum possible. Reduction in the rate of decay is nearly always inversely proportional to the cost of doing so – the slower the rate the greater the cost. Thus, slowing the rate of decay is seen as a trade-off between what is possible, cost and importance. The concept of sacrificial components can be extended to various scales of component. The most obvious is the use of paint layers or renders. However, the concept still holds if it is possible to think of major components as expendable E.g. renewing mullions and tracery work of cathedral windows in order to preserve the glass and the shape and form of the window feature. Likewise replacing items that are accessible with authentic replicas could again fall under the remit of sacrificial components E.g. replacement of thatch. This shades into partial replacements. Partial or total replacements are less favourable options, as they inevitably involve the loss of some of the historic material which constitutes the cultural value of the building.

4.5. Example of application of heritage SLP methodology to cast iron gutter

The following table records part of the analysis of a typical cast iron gutter in the context of a heritage building, and the proposed interventions with the associated residual service lives. Guidance referred to was both technical and related to service lives.[Greathouse & Wessel 1954 and RICS 2001]. .

<table>
<tr><th>Specification</th><th>Causes of deterioration</th><th>Factors influencing rate of corrosion</th></tr>
<tr><td>Cast ductile iron. Oil based gloss painted.</td><td rowspan="3">• Adequacy of support (whether fixings are rust resistant and spacing)
• Design capacity (note potential for overflows where valleys etc discharge)
• Exposure (both wind and pollution, and south facing elevations will tend to get hot, accelerating ongoing deterioration, causing fixing problems due to expansion)
• Maintenance regime
• Vandalism</td><td>Primary
• Nature of metal surface
• Electrode potential
Secondary
• Dissimilar metal reaction
• Hydrogen-ion activity</td></tr>
<tr><td>Current condition</td><td>Required interventions</td></tr>
<tr><td>• Sagging due to broken brackets or rotten fascia
• Water starting to get under paint layer and causing rusting
• Decaying leaves and mosses causing acidic pooling of water</td><td>• Rectify support conditions and rusting and paint thoroughly
• Clean out plant debris yearly
• Paint every 5 years making sure the backs of pipes and underneath the gutters are painted properly
• Fit wire leaf guards to keep the leaves out of the guttering</td></tr>
<tr><td colspan="3">Residual Service Life Assessment – overall and with proposed maintenance interventions:</td></tr>
<tr><td colspan="3">Typical new component service life: 50 years
Typical Lower Limit: 16 years
Typical Upper Limit: 100
Residual Service Life Assessment with proposed interventions: 40 years +</td></tr>
</table>

Table 3 – Analysis of residual service life of cast iron gutter, showing proposed interventions

5. Conclusions on policy to adopt in assessing interventions

The “little and often” approach is the preferred option wherever possible. Replacements of materials of cultural value should be reduced to a minimum. Where new materials are required authentic replicas should be used if possible.

5 REFERENCES

Anderson J and Bourke K 2004, *Literature Review of Whole Life Costing and Life Cycle Assessment in relation to Historic Buildings,* (unpublished report.)

Heritage Counts 2004, *The State of England's Historic Environment*, English Heritage, London 2004 (see www.heritagecounts.org.uk)

Atkins R, Cook M and Michaelis C, 2003, *Environmental Monitoring and Energy Conservation - a case study of Norton Park* . Proceedings of EMCH Conference, Edinburgh, November 2003.

Greathouse GA and Wessel CJ, *Deterioration of materials, causes and preventive techniques.* Reinhold Publishing Corp, USA, 1954

Life Expectancy of Building Components: Surveyor's experiences of buildings in use – A practical guide, RICS, London 2001

MECHANISM OF EFFLORESCENCE ON HISTORICAL BRICK MASONRY BUILDINGS REINFORCED WITH CONCRETE

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ABSTRACT

Seismic reinforcement of historical masonry buildings in Japan is commonly achieved through the application of concrete to inner walls and the installation of concrete columns. However, concrete reinforcement has been shown to cause efflorescence of the external masonry brickwork, leading to decay of heritage structures. In this study, an accelerated test method is employed to reproduce efflorescence on masonry walls reinforced with concrete. The application of intense wetting and drying cycles resulted in the appearance of efflorescence after a only few cycles. Investigation of the relationship between water content and the rate of evaporation from the saturated brick based on the weight change revealed that evaporation is high in brickwork with high water content, and much lower at low water content. From this analysis, it was shown that below a water content of 4–5 vol%, water moves in vapor form and efflorescence does not occur. However, above this threshold, water moves in the liquid state, causing soluble salts to migrate to the brick surface. Accelerated tests were also performed on a wall specimen consisting of bricks, mortar and cement lime mortar backing accompanied by continuous water content measurements using the Time Domain Reflectometry probe technique. Heavy efflorescence occurred on areas of brick that underwent dramatic changes in water content (6–16 vol%), whereas very little efflorescence was observed on brickwork experience lower and less variable water contents (1–2 vol%). It is concluded that efflorescence only occurs at water contents above 4–5 vol%, where water moves in liquid form and causes soluble salts to migrate from the concrete backing to the masonry surface. Thus, when concrete is used for masonry reinforcement, direct contact with concrete should be avoided in areas where high water content is expected.

KEYWORDS

Brick Masonry building, Reinforced Concrete, Efflorescence, Conservation, Water content

1. INTRODUCTION

Many brick masonry buildings were built during the Meiji (1868–1912) and Taisyo (1912–1926) eras in Japan, and approximately 500 brick masonry buildings recognized as being of heritage value are marked for conservation and renovation. Seismic reinforcement is necessary in many cases, as the original masonry buildings typically do not provide sufficient seismic safety. The various materials such as concrete, steel, synthetic resins, and high-strength fibers have been used for seismic reinforcement. Despite the knowledge that concrete reinforcement is incompatible with conservation idea, it is widely used because the reinforcement effect expected in such cases is quite good considering the similarity to masonry walls in terms of rigidity. However, reinforcement with concrete has been noted to cause heavy efflorescence on masonry brickwork due to the leaching of alkali salts from the cement, resulting in extensive decay of the original structure.

This study aims to clarify the influence of water content in brick on the efflorescence of brickwork in heritage masonry structures. An accelerated test method is employed to induce rapid efflorescence, and the effect of water content in the brick is examined in detail.

Efflorescence of masonry brickwork has been investigated extensively,[1-4] including the development of test methods[5,6] and experimental reproduction of efflorescence outdoors.[7] Young (1957), in particular, considered the efflorescence of masonry brickwork that had been reinforced with concrete blocks, and showed experimentally that soluble salts in the concrete cause efflorescence.[8] Thus, it is considered important to further clarify the mechanism of efflorescence on concrete-reinforced masonry brickwork with respect to the effect of water content. It is anticipated that such information will be useful for improving the design of concrete reinforcement for masonry buildings.

2. PROBLEMS WITH CONCRETE REINFORCEMENT

2.1 Classification of reinforcement with concrete

When historical masonry buildings are reinforced with concrete, new columns, beams, walls, and slabs are generally installed as cast-in-place structures and then anchored to the old masonry walls. This method of reinforcement has been employed for inner walls for many years, and is considered to provide sufficient seismic stability. However, this approach makes it difficult to conserve the interior space. The installation of beams and slabs also has the effect of coupling the old masonry walls, but conservation of the original roof or floor is difficult. Buttress reinforcement, on the other hand, while not affecting the interior space, significantly affects exterior design. The installation of concrete columns in the old masonry wall requires a groove to be cut in the old wall, where a steel rod is fitted and the cavity injected with cement mortar. The use of concrete in all such reinforcement techniques inevitably leads to efflorescence.

2.2 Example of concrete-reinforced masonry building

Many historical masonry buildings have been reinforced with concrete in Japan. An example of such a building, built in 1890 and reinforced with inner concrete walls in 1977, is shown in Figure 1. The building was assessed in 2003 to evaluate the extent of efflorescence on exterior walls. Figures 2 and 3 show examples of the efflorescence on the exterior walls of this historical building. Efflorescence was observed on most exterior walls, with a considerable accumulation below the eaves, around cornices, below window frames, and at joints between walls, that is, areas that are likely to remain wet after rainfall. From this survey it was inferred that the mechanism of efflorescence on masonry buildings reinforced with concrete appears to be related to the movement of moisture and salt in the masonry wall.

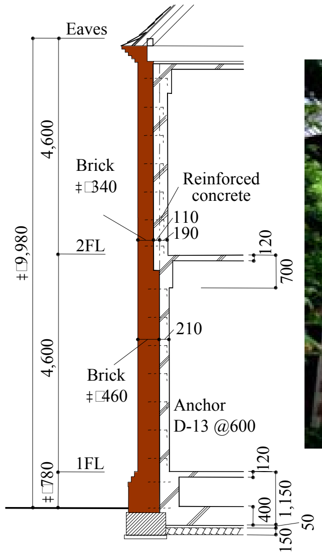

Figure 1 Detail of reinforcement

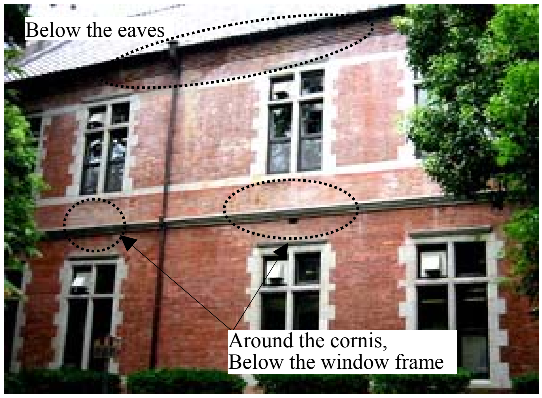

Figure 2 Efflorescence on a historical masonry building reinforced with concrete

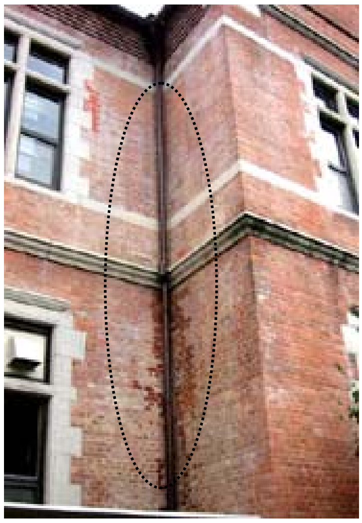

Figure 3 Efflorescence at wall joints

3. ACCELERATED EFFLORESCENCE TEST

3.1 Test wall

The model wall fabricated to attempt to reproduce the efflorescence appearing on masonry walls reinforced internally with concrete is shown in Figure 4. A lining of cement mortar was formed in direct contact with the brick. The backing cement mortar had a water-cement ratio of 0.6 and a cement:sand ratio of 1:2.5. The absorption of 24h cold water of used brick was 13.3 vol%. The brick was wetted for 30 min prior to applying the lining in order to ensure good adherence to the cement mortar.

3.2 Test method

Epoxy resin was applied to the surfaces as shown in Figure 5. In the test, the specimen was kept wet for 24 h by pouring water into a reservoir above the specimen, and subsequently was allowed to dry for 72 h, after which the surface of the brickwork was observed. This process was repeated ten times. The test was carried out in a thermostatic chamber maintained at 20 °C and 60% relative humidity.

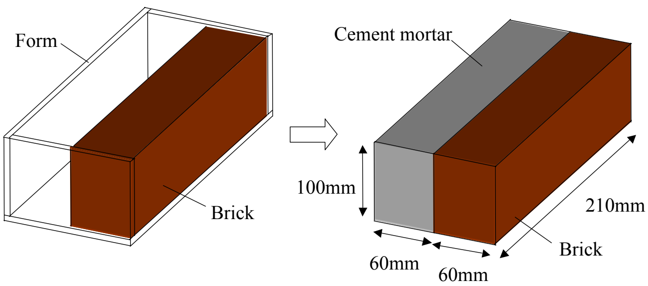

Figure 4 Test specimen

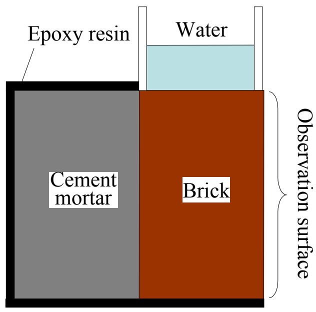

Figure 5 Test method

3.3 Results

The test results are shown in Figure 6, and the ratio of efflorescence area to the total exposed brick area is shown in Figure 7. The ratio was calculated by applying 140 of threshold value (0=black, 255=white) to gray scale images of test result pictures. And it calculated by the image processing program *imageJ* developed by National Institute of Health. Efflorescence appeared in a few cycles of repetition of wetting and drying. It was considered that efflorescence was successfully reproduced by this procedure.

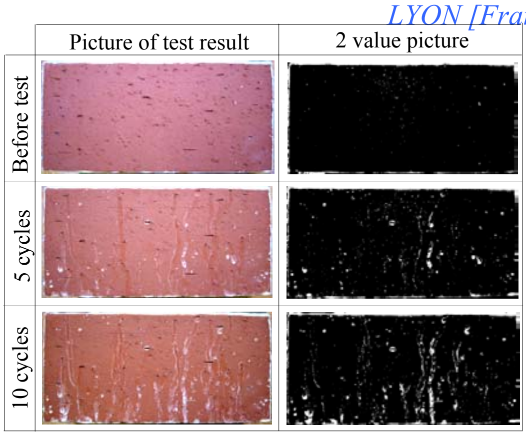

Figure 6 Test result: View of observation surface

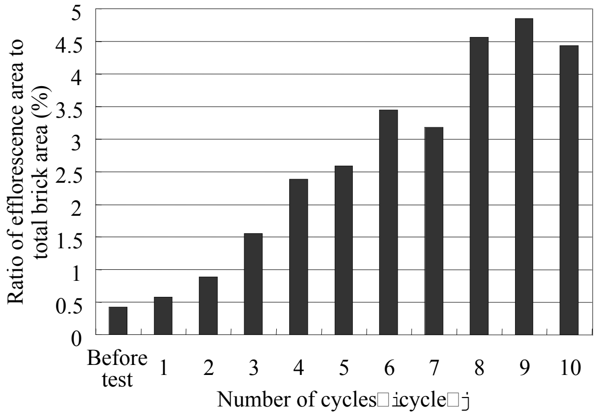

Figure 7 Ratio of efflorescence area to total brick area

4. MEASUREMENT OF WATER CONTENT IN BRICK

4.1 Time Domain Reflectometry

Time Domain Reflectometry (TDR) is a relatively new method that is used to measure the water content in soil or porous materials by applying an electromagnetic pulse via a probe embedded in the material. The dielectric constant can then be calculated from the propagation velocity of the electromagnetic wave in the material. This allows for very sensitive measurement of water content, as the dielectric constant of liquid water ($\varepsilon \cong 80$) differs considerably from that of soil ($\varepsilon \cong 3$–10) and air ($\varepsilon \cong 1$).

The TDR probe configuration and waveform are shown in Figure 8. The dielectric constant around the probe is calculated by

$$\varepsilon=(ct_s/2L_p)^2 \quad (1)$$

where c is the speed of light (m/s), L_p is the length of the probe (m), and t_s is the travel time for the pulse to traverse the length L_p (s).

The volumetric water content θ (m^3/m^3) is calculated from the following empirical equation proposed by Topp et al.[9)]

$$\theta=(-530+292\,\varepsilon-5.5\varepsilon^2+0.043\varepsilon^3)\times10^{-4} \quad (2)$$

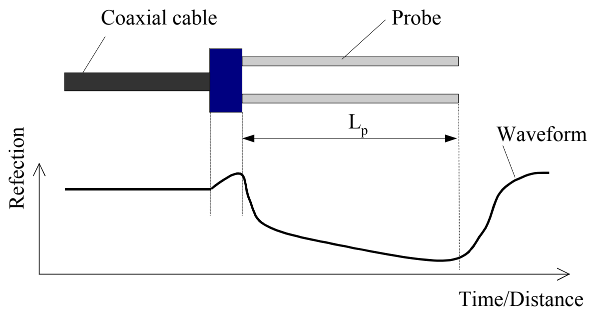

Figure 8 TDR probe and waveform

However it is necessary to calibrate the equation for individual material samples in order to measure the water content more accurately.

4.2 Calibration method

The TDR probe was constructed from stainless steel rods with a diameter of 3 mm. The probe length was 50 mm, and the rod separation was 10 mm. The calibration method is outlined in Figure 9. The TDR probe was embedded in a cylindrical brick sample (50 mm in diameter, 100 mm in height) for measurement of the dielectric constant. In the test, the saturated brick sample (volumetric water content: 0.128) was placed in a thermostatic chamber maintained at 20 °C and 60% relative humidity. The dielectric constant was measured continuously until the brick sample had dried. The weight of the sample was also measured during this process, and the water content was calculated from the weight change.

4.3 Relation between dielectric constant and water content

The relation between the dielectric constant and the volumetric water content is shown in Figure

10. The test results are considered to correspond relatively well with Topp's empirical equation (2) with a slight shift of values. The following third-order approximate equation was therefore derived for the present type of brick:

$$\theta=(-776+350\,\varepsilon-5.90\varepsilon^2+0.0401\,\varepsilon^3)\times10^{-4} \quad (3)$$

Equation (3) is thus suitable only for the brick type examined in this study.

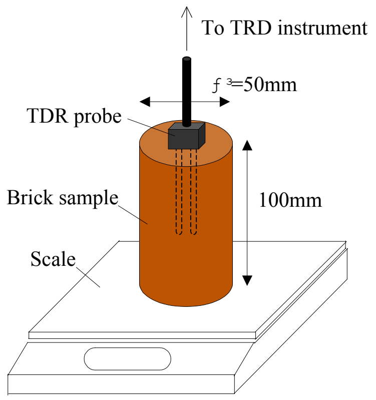

Figure 9 Calibration method

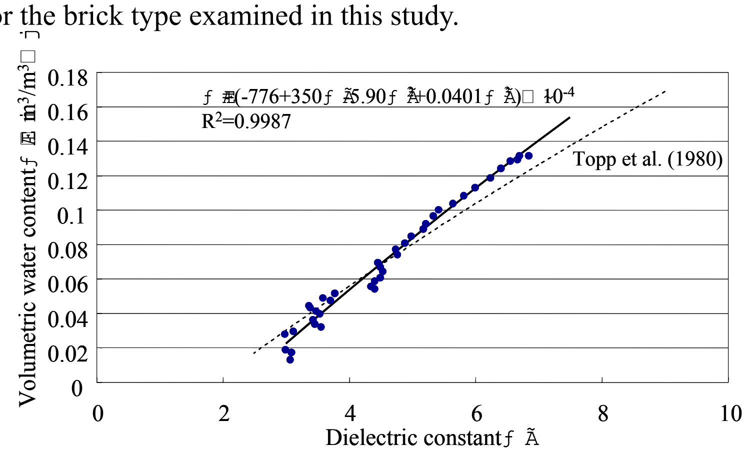

Figure 10 Relation between dielectric constant and water content content

5. MODE OF MOISTURE MOVEMENT IN BRICK

Water moves through brick in both liquid and vapor forms, however it is the movement of liquid water which is expected to move the soluble salts responsible for efflorescence. Therefore, the relationship between water content in the brick and the mode of moisture movement was investigated.

It is possible to understand moisture movement from the relationship between water content and the rate of evaporation from the brick. Rapid evaporation from the brick occurs at high water contents due to the movement of liquid water in the brick to the dried surface. However, the evaporation from brick at lower water contents is much less and relatively constant, driven by the movement of water in the vapor state. The relationship between evaporation rate and water content, calculated from the test results in the previous chapter, is shown in Figure 11.

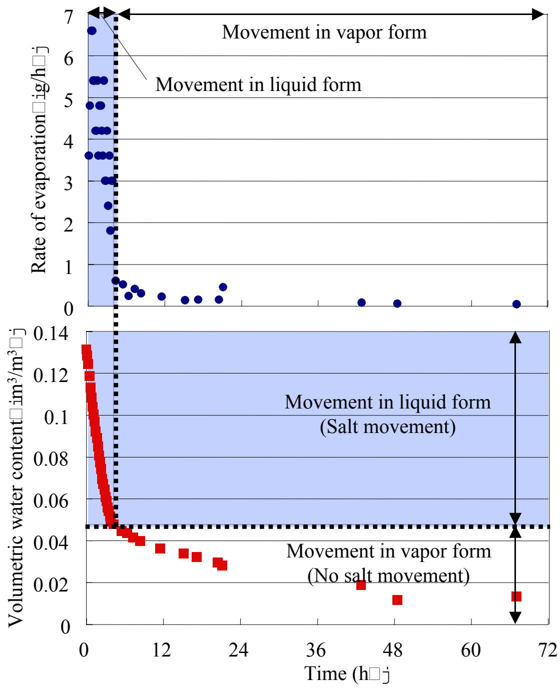

Figure 11 Evaporation rate and water content

The rate of evaporation was very high at high water content, but decreased with reducing water content to reach a low, relatively constant value below a water content of 4–5 vol%. This result shows that water does not move in the liquid state when the water content in the brick is below this lower limit. That is, the movement of soluble salts is expected at water contents higher than this lower limit.

6. ACCELERATED TEST OF MORTARED BRICKWORK

6.1 Model wall specimen

Although the accelerated test was performed above for a brick specimen, it is also necessary to consider the influence of joint mortar on efflorescence as an integral component of masonry brickwork. A wall specimen similar to actual historical masonry brickwork was prepared, as shown in Figure 12. The specimen consisted of two bricks with a cement lime mortar joint and a cement mortar backing.

The brickwork was laid using cement lime mortar and allowed to cure for 28 days at 20 °C and 60% relative humidity. A lining of cement mortar was then formed in direct contact with the brickwork. The brickwork was made wet for 30 minutes before the lining in order to adhere to cement mortar well. The joint mortar included a lime constituent, as commonly employed before the Second World War. The specimen properties are listed in Table 1.

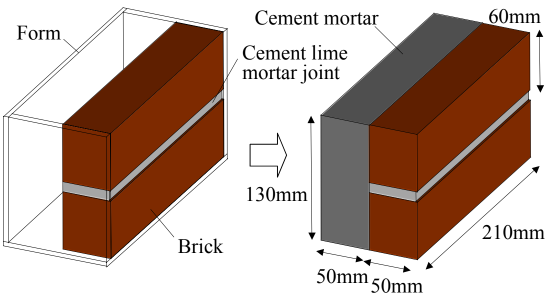

Figure 12 Test wall specimen

	Position of brick	Absorption of 24h cold water (m^3/m^3)	Joint mortar	Cement mortar backing
Cement mortar backing	Upper	0.16	Cement*1:Lime*2:Sand*3 □□1:2:5□iby volume, Sand:2.5mm below) Water/Lime□{Cement =0.6□iby weight□j	W/C=0.6, Cement:Sand =1:2.5 □iSand:5mm below□j
	Lower	0.16		
No backing	Upper	0.17		
	Lower	0.17		

*1 Ordinary Portland cement, density: 3.16 g/cm^3
*2 Slaked lime, density: 0.57 g/cm^3
*3 Density: 2.62 g/cm^3, Absorption: 1.27%

Table 1 Absorption of brick, joint mortar and mortar backing

6.2 Test method

The test method is outlined in Figure 13. In the test, as in the previous test, the specimen kept wet for 24 h by pouring water into a reservoir above the specimen and subsequently dried for 72 h. This process was repeated ten times in a thermostatic chamber maintained at 20 °C and 60% relative humidity. The water content of each brick was measured continuously during the test using TDR probes embedded in each brick.

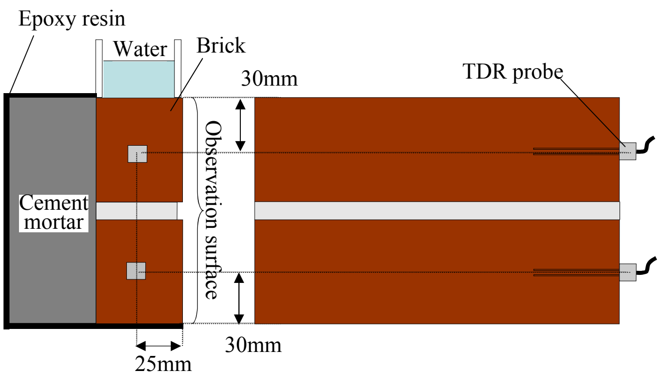

Figure 13 Test method and position of TDR probe

6.3 Observation results of efflorescence

The test results are shown in Figure 14, and the calculated ratio of efflorescence area to the total exposed brick area is shown in Figure 15. The efflorescence area increased with repeated wet/dry cycles. The joint mortar (cement lime) appeared to contribute little to the efflorescence compared to the cement mortar backing, indicating that cement lime mortar supplies less salts than the cement mortar backing. The distribution of efflorescence accumulation was not uniform, with heavy efflorescence appearing on the upper portion of the brickwork. This is attributed to a non-uniform distribution of water content in brickwork, varying from high content near the top (closest to the reservoir), to low water content near the base.

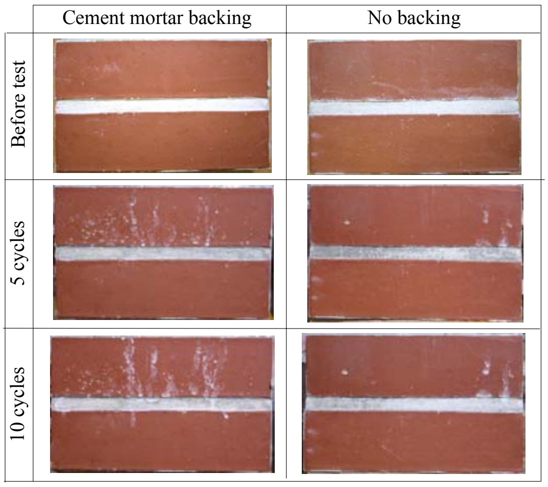

Figure 14 Observation surface after accelerated test

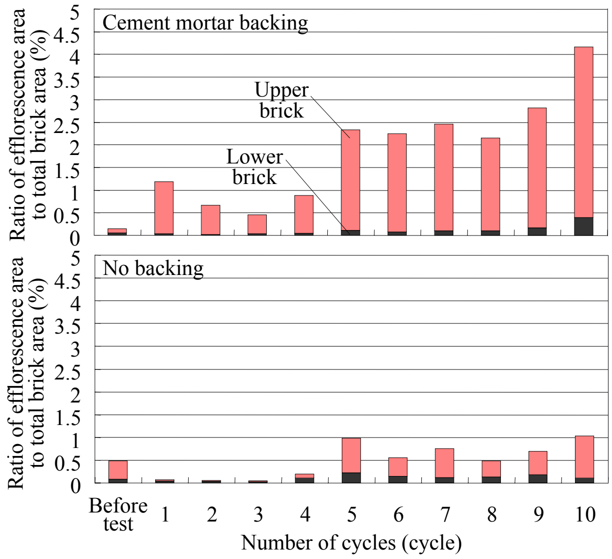

Figure 15 Ratio of efflorescence area to total brick area

6.4 Water content of brick

The change in the water content in each brick is shown in Figure 16. The results are similar for both specimens, with and without cement mortar backing. The water content of the upper brick changed from about 6 vol% to 16 vol% at saturation, whereas the water content of the lower brick changed varied from 1 to 2 vol%. It is considered that water did not migrate into the lower brick due to the lower permeability of the intervening joint mortar.

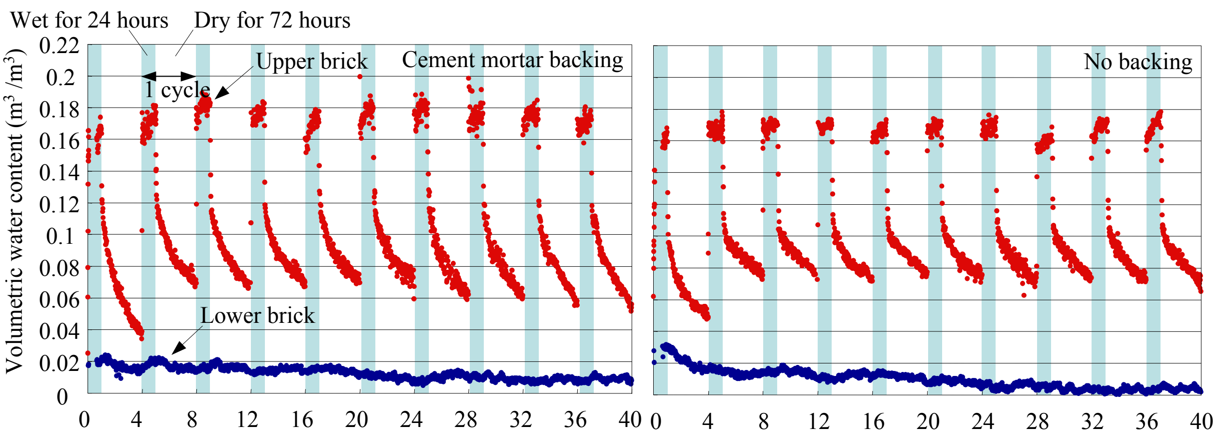

Figure 16 Change of water content in each brick

6.5 Relationship between water content of brick and efflorescence

Judging from the observation of the heaviest efflorescence on the upper brick surface of specimen with cement mortar backing, efflorescence on masonry surfaces is expected to occur in areas where the brickwork comes into direct contact with the concrete backing and to be driven by high water content and wet/dry cycles. The heavy efflorescence on the upper brick is considered to indicate a high water content of above 4–5 vol%, at which water moves in the liquid state, causing soluble salts to move to the brick surface. The lesser efflorescence on the lower brick can therefore be attributed to the lower water content, less than 4–5 vol%, at which water moves in vapor form and is unable to move soluble salts to the surface.

7. MITIGATION OF EFFLORESCENCE

Based on the above results, efflorescence on concrete-reinforced masonry brickwork can be successfully mitigated by taking measures to ensure that the water content of the wall remains low and does not fluctuate dramatically. Efflorescence can also be prevented by not allowing the wall to come into direct contact with the concrete reinforcement in areas expected to be subjected to high water contents. The prevention of water infiltration can be achieved by conserving buildings indoors, that is, sheltering the building from rain by installing some form of cover. However, it is often difficult to maintain a low and constant water content over the long term as buildings are usually conserved outdoors. Therefore, it is important to clarify how water is supplied to masonry wall, during rain, by moisture content rising and by condensation. It is also important to ensure that highly wetted areas do not come into direct contact with the concrete reinforcement during the design process. If this is not avoided, concrete and masonry wall should be separated by waterproof layer. The best practice, such as good water insulation and appropriate adhesion providing construction method should be adopted for historical masonry buildings. The mitigation method will be studied more concretely in the future.

8. CONCLUSIONS

Accelerated wetting and drying tests successfully reproduced the efflorescence observed on concrete-reinforced masonry brickwork, and it was revealed that efflorescence appears on masonry surfaces where the brickwork comes into direct contact with concrete backing. Efflorescence was also shown to be driven by high water content in the brickwork, and to emerge with repeated wetting and drying. Efflorescence did not occur below a water content of 4–5 vol%, below which water moves only in vapor form. Above this water content, the movement of liquid water causes soluble salts to migrate to the surface. Therefore, efflorescence on masonry brickwork can be mitigated by taking measures to ensure a low and constant water content in the brickwork, and by not allowing areas of brickwork expected to be subjected to high water contents to come into direct contact with the concrete reinforcement.

REFERENCES

1) Chin, I.R., "Spalling of Brick," Masonry: Opportunities for the 21st Century, ASTM STP 1432, D. Throop and R.E. Klingner, Eds., ASTM International, West Conshohocken, PA, 2003, pp.97-113
2) Chin, I.R., and Petry, L., "Design and Testing to Reduce Efflorescence Potential in New Brick Masonry Walls," Masonry: Design and Construction, Problems and Repair, ASTM STP 1180, J.M. Melander and L.R. Lauersdorf, Eds., American Society for Testing and Materials, Philadelphia, 1993, pp.3-21
3) Tomassetti, Albert A., "Problems & Cures in Masonry," Masonry: Components to Assemblages, ASTM STP 1063, American Society for Testing and Materials, Philadelphia, PA. 1990, pp.324-338
4) Grimm, C.T., "Durability of Brick Masonry: A Review of Literature," Masonry: Research, Application, and Problems, ASTM STP 871, J.C. Grogan and J.T. Conway, Eds., American Society for Testing and Materials, Philadelphia, 1985, pp.202-234
5) Isberner, A.W., "A Test Method for Measuring the Efflorescence Potential of Masonry Mortars," Masonry: Research, Application, and Problems, ASTM STP 871, J.C. Grogan and J.T. Conway, Eds., American Society for Testing and Materials, Philadelphia, 1985, pp.27-37
6) Harris, H.A., "A Method to Determine Efflorescence and Water Permeance of Masonry Mortars," Masonry: Research, Application, and Problems, ASTM STP 871, J.C. Grogan and J.T. Conway, Eds., American Society for Testing and Materials, Philadelphia, 1985, pp.88-100
7) Ritchie, T., "Study of Efflorescence on Experimental Brickwork Piers," Journal of the American Ceramic Society, Columbus, Ohio, Vol.38, No.10, Oct. 1957, pp.357-361
8) Young, J.E., "Backup Materials as a Source of Efflorescence," Journal of the American Ceramic Society, Columbus, Ohio, Vol.40, No.7, July 1957, pp.240-243
9) G.C. Topp, J.L. Davis, and A.P., Annan. "Electromagnetic determination of soil water content: Measurements in coaxial transmission lines", Water Resources Research, Vol.16, No.10, June 1980, pp.574-582

Building Pathology database and maintenance approach in a well defined contex: Calabrian Historical Centres

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ABSTRACT

The practise of "design for durability", connected with increasing durability, flexibility and reducing the need for expensive maintenance, clearly needs new design strategies oriented to the assestement of service life of materials and components, to the control of the main factors affecting service life and to the prevention of disturbing factors. At the same time it is crucial the necessity of collecting, recording and evaluating data related to failures of building, in a systematic way.
This study deals with the close relationship existing between the durability of building materials and components and the maintenance approach. The paper begins from results of a more general research activities developed at DASTEC, *Mediterranea* University of Reggio Calabria, whose aim is the formulation of a normative guide to operate in the whole Region, through the introduction of Protocols for the diagnosis and the control of the different phases of the maintenance plan.
The paper presents a methodological proposal for ex-post evaluations of the durability of building materials and components, through the definition of a "building pathology database" usefull in the planning and the management of maintenance program. At the same time there are introduced the preliminary results of an ongoing research developed by the authors and pointed at the possibility to apply the Factor Method to existing buildings in a well definited contex, with regard to the definition of values of Factor G, taking care of the principal management maintenance instruments regulated by the National Laws and Local Codes. The pre-modern building of the Calabrian smaller historical centres built in masonry, is the investigation object of the research. This patrimony is still representative of a constructive tradition whose features testify persistence of technical and material specificity. Today many of these buildings show evident marks of degrade both in form of pathologies, instability, alterations and technical-morphological incongruity. The "building pathology database" proposed, aims to structures in a systematic way data available from the results of the mentioned investigation methodology applied to 52 different Calabrian historical centres using a survey protocol. The record sheets have a simple and flexible configuration to guarantee the easy integration and correction of information. In particular, the work elaborates the results of recognition, description and understanding of morphological and materic characters of these historical buildings, as well as detection and evaluation of their pathologies, in order to verify and evaluate the possibility to introduct the mentioned Protocols in to the National Normative instruments for planning and management of maintenance program.

KEYWORDS

Survey Protocol, Pathology, Service Life, Building Pathology database.

1 INTRODUCTION

The principles of the sustainable development, pointed up in the last years, actually attempt to move the attention to connect the demand of a more rational use of available resources, with the problems of building durability and the promotion of maintenance in order to assure the extension of conventional life cycle of buildings as well as materials and components and a global economic efficiency.
Actually, one of the questions clearly expressed at international level is reducing the whole life cicle costs of buildings and the environmental impacts of the construction sector, which is indeed the major responsible for using of non renewable resource. Questions, furthermore, validated just now, thanks to some important experiences as well as the ISO Standards on service life planning (ISO 15686 Parte 1, 2000); the Directive 89/106/EEC; and, with regard to the Italian situation, the Law Merloni 109/96, which is focussed upon the strengthening in a qualitative direction of the central role of the building construction planning process, through the assumption of the concepts of materials and components' durability, material's compatibility and possibility to easily verify performances over the time, elements' replaceability, maintainability, maintenance and management-related costs.
According to some final results developed in different Countries thanks to national experiences, like CRESME and ANCE in Italy, the Directorate of Cultural Heritage in Norway (Nypan, 2002), and further the UK and USA ones reportered in ISO 15686 Annex A, a high percentage of buildings life cycle costs is brought about ineffective or late maintenance policies. Moreover, in Europe, some key documents, validate from Brussels, identify Built Cultural Heritage as an important element in sustainable development (EU REPORTS, 1996–1999). These documents emphasise the need to protect, maintain and enhance the Built Cultural Heritage, as well as the need to develop such methods and tools for monitoring the maintenance state of this patrimony.(Haangerund *et all*, 2001)
Failures prevention and the organization of efficient maintenance programs require the availability of methodologies and tools eable to predict the way and the time of performance quality decay of buildings and/or to evaluate the real state of conservation of existing buildings. The main difficulty in formulating such analytical method resides in the increasing complexity of buildings, materials and systems, as well as in the complexity of the degradation process, which usually depends on a number of aspects such as the caracteristics and the compatibility of materials and components, the macro & micro environmental conditions, the installations modalities, the in use conditions, the maintenance practices, ecc.. Actually, it appears essential learning from the past and making use of the knowledge that can be derived from building failures and their treatment, in order to allow an easy identification of defects, as well as questing for solutions to reduce failures incidences, and their effects, avoiding consequently unforeseen repair or maintenance costs.
On the basis of this background, the practise of "design for durability", connected with increasing durability, flexibility and reducing the need for expensive maintenance, clearly needs effective design strategies oriented to the assestement of service life of materials and components, to the control of the main factors affecting service life and to the prevention of disturbing factors. At the same time it is crucial the necessity of collecting, recording and evaluating data related to failures of building in a systematic way, with, where possible more systematic feedback from experiences and existing knowledge, to providing information to involved bodies like: regulations and code makers, designers, contractors, implementers of quality assurance systems, insurance companies, planners, etc.

2 AIMS

In the last years knowledges about Building Pathologies have increased remarkably, as well as the tools and the methodologies for collecting and assessing date. Despite that, much has still to be done in the direction to connect the knowledges about building pathologies to the prediction and the assessment of performances of building materials and components during their Service Life.
This study deals with the close relationship existing between the building Pathology, the durability of building materials and components and the maintenance approach.
The "building pathology database" proposed, is the result of a research activities developed at DASTEC, *Mediterranea* University of Reggio Calabria. (Nesi, 2002)
The mentioned Database elaborates the results of recognition, description and understanding of morphological and materic characters of historical buildings, as well as detection and evaluation of

their pathologies. The final objective of research activities is acting so that evalutative protocols of durability become part of the elements usually considered necessary for a complete tecnical and economic definition of the “project” in the building process: from the requirement of acceptability of materials and components, to specification of performance with the relative proof test methods, to the Maintenance Plan, both in new buildings design and like in this case, in existing building interventions. (Lauria, 2002)

Moreover, the paper presents the preliminary results of an ongoing research developed by the authors, which aim is to define a methodological proposal for ex-post evaluations of the durability of building materials and components. In detail, the methodological proposal is pointed at the possibility to apply the Factor Method to existing buildings in a well definited contex, with regard, in particular, to definitions of values of Factor G, taking care of the principal management maintenance instruments regulated by the National Laws and Local Codes.

3 THE RESEARCH FIELD

Restoration of historical centres, without abdicating aims of development, productivity and sustainability, is one of the most urgent problems in Calabria. Knowledge and assessment of building constructive characters has a main and complicated role in this field.

The pre-modern building of the Calabrian smaller historical centres built in masonry, is the investigation object of the researches. This patrimony is still representative of a constructive tradition whose features testify persisting of technical and material specificity. A strong homogeneity of constructive behaviour and a shortage of technical solutions actually correspond to a range of formal, technical and material results. Constructive oneness tied up to the availability of materials and local competencies to realize architectures connected to the territory.

Today many of these buildings show evident marks of degrade both in form of pathologies, instability, alterations and technical-morphological incongruity, for which interpretation it is necessary interface the knowledge of materic-constructive characteristics of the technical elements constituent the building with the study of data context. The work introduces, on the one hand, the delimitation of the geographical and historical criteria; on the other hand, the definition of a survey protocol, with the normalization of *what* and, above all, *how* to survey.

The investigated sample is made up of 104 buildings in bearing wall in the 52 different Calabrian historical centres chosen according to historical-geographical and statistical criteria.

4 INVESTIGATION METHODOLOGY

Today’s defect investigation is more complicated due to the increasing complexity of buildings, materials and systems and to an increasing rate of developments in our societies. Undoubtedly it is necessary a reliable identification and a clear and accurate description of the defect, together with a suitable method of investigation that leads the most probable cause.

Results of some research activities worked up at DASTEC, partial joined in this paper, make up, at now, an important and useful systematic collection of informations concerning the knowledge of morphological and material characters of historical buildings, individualization and evaluation of pathologies, local constructive traditions, local environmental conditions, availability of materials.

The investigation methodology has used a survey protocol which has favoured the acquisition of different information. On each one of the 52 historical centres selected, the following activities have been set up:

Bibliographical and archive investigation: to take information about the centre history, its development, architectural emergencies.

Retrieval of the cartographic and aerofotogrammetrical supports: to define the morphological and naturalistic characteristics, as well as the extension of the confinements.

Acquisition of the climatic-environmental data: to punctually define characteristics and relationships with the “environment system” of the examined historical centre.

Contact with the village Mayors: to involve the technical structures and the inhabitants too in the survey actions.

Systematic collection of photographic and iconografic material.

Recognition on the regional productive activities, to know actual availability of traditional and innovative construction materials.
In particolar the acquisition of technical information concerning:
- Geographical data: latitude, altitude, location, topomorfological features of the site.
- General historical report about the village and its territory.
- Subdivision of the historical centre in homogeneous zones.
- Individuation of urban morphology and building types in the urban compartments.
- Individuation of buildings-sample.
- Knowledge of buildings: filing of the technical-constructive features, of the constituent materials and of the dimensional characteristics of the structures.
- Seismic safety requirements: survey of the meaningful aspects to the seismic goals; hypothetical structures of foundation; structures of elevation (masonry, openings, connections); horizontal structures (basic, intermediate, roof); vertical connections; completion elements; materials (raw materials adopted, origin, permanences of materials and techniques, historical news, and so on).
- Check of indoor hygiene and safety, as well as of correspondence between technical elements and energetic requirements.
- Recording of the uncontrolled recurrent transformations, of the projected ones and of the possible interventions of maintenance.
- Individuation of recurrent pathologies.
- Evaluation of the state of maintenance of the technical elements.
- Performances and compatibility of lapideis material and mortars. Collecting of champions of materials of the masonry: external and inside plasters, mortars and rocks.
- Individuation of the degradation factors characterizing the exposure environment of the building.

The building pathology database proposed takes into account building profile, design and technological parameters, type of materials, alterations, degradation factors, quality of construction, macro & micro environment including topography, orientation, maintenance practices, and usage. Data, acquired have been systematically transferred in an informatic “informative system”. The sheets have a simple and flexible configuration to guarantee the easy integration and correction of informations concerning the knowledge of buildings as well as individualisation and evaluation of their pathologies. (Annex 1)

4.1 The Survey Phase. Identification of the alteration processes

The technological system of the buildings sample, has been decomposed according to the criteria of the classification UNI 8290-Parte 1 (1981), in *foundations*, *elevation structures* (masonry, openings), *horizontal structures* (base and intermediary slabs, balconies, roofs, mouldings), *vertical connection structures*, *completion elements* (indoor partitions vertical/horizzontal, fixtures, balusters, handrails, and so on).
According to this decomposition, it has been decided to make use of “survey card” to detectioning and evaluating recurrent pathologies, which contains the denomination of *pathology* analized, a *description of the phenomenon*, the *damage degree*, i.e. the indication of the level of advancement of degrade and the instrumentations to develop possible *diagnostic examinations*.
During the survey phases, the direct observation has shown that the pathologies of buildings frontiers life roofs, masonry, attacks to earth and so on, are the most evident images of the urban degrade. The environment actions on the exposed surfaces are the principal cause of the progressive alteration that strikes the historical patrimony investigated. In particular, the different contexts of the samples selected are characterized by the preponderance of physics, chemical, and biological actions.
The physical factors, which cause to materials mechanical stress, directly influencing façades performances, have caused in the building structures damages that have involved both hangings and coverings, transforming actually the image degrade in functional degrade.
The action of the air motions, the erosive action of the beating rain, the bumps, the thermal and the higrometric alterations, the migrations of water inside the materials and the its changes of phase, capillary reascent of damp, the absorption of meteoric waters, the infiltration damp, the condensation and the evaporation, the intense cold, have caused a series of significant alterations on the exposed surfaces, with consequent performances fall.

The factors of chemical degradation have particularly interested the exposed stones and plaster surfaces. The deterioration of these materials however has been often individualized in the concomitant presence of physical and biological factors, as well as the thermal and the higrometric unbalances that create the favourable conditions for chemical attack.
Other factors are air damp and air pollution level. About air pollution it is important to underline, how degenerative phenomena caused by man's actions have modified the behaviour of traditional construction materials, object of investigation, accelerating what, once, could be the artefact natural ageing process. The different and unnatural level of concentration of some of the natural air components have caused on the exposed surfaces salty efflorescence, corrosions, black crusts, break-ups, and exfoliation. These alterations have created precarious environmental conditions and sometimes unhealthy, causing technological and esthetical decline of the buildings, as well as corrosion of the wall structures and separations of coverings, besides, acting as nutrient substratum for some groups of micro-organisms
The bio-deterioration depends on the life cycle of the micro-organisms and, particularly, on chemical trials triggered off by their organic deposits on the materials surfaces, it is besides, favoured by the rate of damp, when it is superior to the norm, by the presence of particular mineral salts inside the materials, and often, by the presence of organic substances applied on surfaces whit the purpose of protection or renovation. In particular in the samples analized the biological factors have caused the alterations of stone surfaces.
In many buildings, pathologies are also due to defects to the use of non fit materials, to wrong project choices and to scarce executive levels. With reference to the results of the survey campaign it has been frequently recorded: absence of drainage and consequent rising damp in ground floor slabs, use of stones available in the zone, of easy workability but unfit to bear shear and bending efforts, use of mortars for masonry with scarce percentage of binders and consequent rapid erosion that has caused separations and collapses, use of seismic roof structures, use in restoration interventions, of not traspirant paints which has prevented the evaporation of water in excess, rainwater pipes, eaves gutters and pitches that carry great masses of water in located points and that trigger off a series of degrade phenomena, use of freezing stones, and so on. (Annex 2)
The contribution, directed to a first level interpretation of phenomena (pre-diagnosis), has offered an express evalutation tool of the real degradation state of the considered element, delegating to the use of methodic and suitable instrumentations the cognitive close examinations (diagnosis).

5 SUITABILITY OF THE BUILDING PATHOLOGY DATABASE IN THE MAINTENANCE APPROACH: THE ONGOING OPPORTUNITY OF DEFINITION OF FACTOR G

The evaluation of building components durability assume a strategic role in the planning of buildings maintenance programme, in fact, it could offer an exact indication to cadence the controls and interventions to execute correct management of building and its parts during life cycle.
According to this concept, one of the aims of the present work is to introduce the knowledges deriving from the building pathology database into the principal management maintenance instruments regulated by the National Laws with particular reference to the Maintenance Book, in order to to try out the suitability of Factor Method and in particular the ongoing definition of Factor G.
In reference to what it is expressed in ISO 15686 about Factor G – Maintenance Level, the location of the value range of Factor G depends on a well-defined planning of maintenance operations, on the qualitative levels, on the feasibility evaluations, on the technical compatibilities of the estimated operations and on the happening of unexpected conditions regarding the planned maintenance of the building. According to the knowledges deriving from the Building Pathology Database assume a strategic role both in choosing interventions necessary to stop the growth of pathological mechanisms, and in planning future buildings maintenance programme. More specifically in the National contex, Outline Law 109/94, provides specific guidelines on working instruments and on the aims of maintenance programme. Furthermore these guidelines embrace contents expressed in the national standard UNI 10874 "Standards for drafting in use and maintenance manuals"concernig the structuring of each one of the working documents that make up the Maintenance Plan: In Use Manual, Maintenance Manual and Maintenance Programme

Among the mentioned instruments, the Maintenance Plan is the main management tool of maintenance activities. It highly controls the planning of interventions during the time, the location and allocation of avaible resource, allowing besides rising performance levels of buildings, according to the property strategies fixed by estate, and controlling economic management demands.

6 CONCLUSION

Calabrian historical centres constitute a cultural and environmental resource, as well as a patrimony of great importance from the quantitative point of view. In according to these considerations appears necessary to move the attention to compare with the assessment of building durability and the promotion of maintenance.

The paper presents a methodological proposal for ex-post evaluations of the durability of building materials and components. The investigation activity has allowed to structure informations about the behaviour, during the time, of a particular typology of buildings realized in bearing wall. The building pathology database proposed is beginning to look as an useful instrument for intervention to existing buildings, from choosing interventions necessary to stop the growth of pathological mechanisms, to planning future maintenance approaches, or drawing maintenance plans, as well as a predictive tool in new design projects, useful for reliable prevision of durability of building materials and components, for finding out in a objective and repeatable way the probable damage state of elements.

On the other end it is also beginning to look as an useful propaedeutic instrument for application of "Factor Method" in an experimental way, In particular, according to this results an ongoing work of a research group at DASTEC deals with the aims to associate the results derived from Building Pathology Database with a preliminary contribution to try out the suitability of Factor Method and in particular the steady definition of Factor G through the principal management maintenance instruments regulated by the National Laws.

Therefore, the improvement and the implementation of working methods for a reliable prevision of Service Life to be applied in a concrete way, represents the challenge and the commitment from researches for years to come. This study and the recent works aimed to the introduction of Protocols for the diagnosis and the control of the different phases of the plan, are directed towards these goals.

7 ACKNOWLEDGMENTS

The present contribution is an essential part of the research project:
Tools for guide and technical control of the project and the realizative processes in interventions in Calabrian historical centres (1994/1999)
Criteria and tools for evaluating reliability of buildings and buildings component (COFIN 2001)
financed by UE, carried out by the research group S.T.O.A. coordinated and directed by Prof. Attilio Nesi, at Department DASTEC Art - Science and Technique in building.

8 REFERENCES

CEE 89/106, (1989), *Construction Products Directive*, European Commission, Bruxelles

CIB W86 ,1993, *Building Pathology a state of the art report;* CIB report publication n. 155, CIB, Rotterdam

CIB, 1993, *Feedback from practice of durability data - inspection of buildings* CIB report publication n.157, CIB, Rotterdam

Lauria M. (a cura di) (1998) *Analisi tecnologica e codici di pratica per gli interventi nei centri storici minori della Calabria - Provincia di Reggio Calabria* - FALZEA Editore, Reggio Calabria

Lauria M. (2002), "Failure cases in buildings of calabrian historical centres" *CIB W086 Meeting,* 7-8 Ottobre, Budapest.

EOTA GUIDANCE DOCUMENT, *GD 002* (1999), *GD 003* (1999), *GD 004* (1999), *GD 005* (1999), European Organization for Technical Approvals, Bruxelles

EU REPORTS (1996-1999) *EU Commission Green Paper on Urban Environment*, 1996; *European Sustainable Cities Report*, 1996; *Sustainable Urban Development in the EU: A Framework for Action,* Communication, 1998; *ESDP European Spatial Development Perspective*, 1999.

ISO 15686 Part 1 (2000) *Buildings and constructed assets. Service life planning. General Principles*

Haagenrud, S.E. & Stordahl, P. & Eriksson, B. & Riks, E. & Krigsvoll, G. & and Garofolo I., 2001 *EU-project ENV4-CT98- 0796 MMWood*. Final report, NBI, Oslo.

Hovde, P. J. 2002 *Factor Methods for service life prediction. A state of the art.* Final draft report

Nesi, A. (a cura di), 2002, *Normativa tecnica locale per il progetto dell'esistente premoderno. Guida e controllo tecnico delle azioni di recupero nei centri storici della Calabria*, Gangemi, Roma

Harris, Samuel Y., 2001 *Building Pathology: Deterioration, Diagnostics, and Intervention*, Wiley, New York.

Garrand C., 2001 *HAPM guide to defect avoidance*, London, Spon Press.

Nypan T., 2002 *Norwegian Directorate of Cultural Heritage STUDY 2000*, Internal working paper, Oslo.

Law 109/1994 *Law Merloni – Outline Law about Pubblic Works.*

D.P.R. 554/1999 *Carrying out regulation of Law 109/94 - Outline Law about Pubblic Works*

UNI 10874, 2000 *Standards for drafting in use and maintenance manuals.*

UNI 8290-1, 1981 - Edilizia residenziale. Sistema tecnologico. Classificazione e terminologia.

UNI 8290-2, 1983 - Edilizia residenziale. Sistema tecnologico. Analisi dei requisiti.

UNI 8290-3, 1987 - Edilizia residenziale. Sistema tecnologico. Analisi degli agenti.

A Sustainable Approach for the Conservation of the Timur Shah Mausoleum in Kabul, Afghanistan

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ABSTRACT

The Timur Shah Mausoleum is one of the landmarks of the Afghan capital Kabul. Erected in 1816, it consists of an octagonal brick construction with a dome on its top. The building was damaged by a misfired cannon ball during a British attack in the 19th century. As a consequence approximately one third of the dome collapsed. The gap was closed by Indian masons but collapsed again. Another temporary repair solution was applied in 1936/37 consisting of sheet metal covering the hole in the dome. To restore the original appearance of the monument it was decided recently to complete the missing part of the building permanently and to reinforce its structural integrity, in particular considering the frequent occurrences of earthquakes. Before the actual interventions were performed the structural state and the building materials used for the mausoleum have been analyzed in detail. The bricks used exhibited a high variety of physical properties such as total porosity or total water absorption. Due to the mineralogical composition it could be concluded that the bricks were fired at a low temperature, probably not above 700°C. The bedding mortar consisted of a lime binder with weak hydraulic properties. Notable was the high amount of clay and silt and the presence of charcoal. The latter one was in particular the main reason for the low apparent density of the mortar. A high number of lime lumps directed to the use of quick lime as a binding material. The conservation efforts required an approach, which could be realized and sustained on site. That meant the use of traditional and on site available materials and techniques. The structural stabilization of the masonry and the drum as a basic intervention had to be carried out with contemporary technology. But the actual restoration of the dome was done by Afghan craftsmen using low fired bricks from a local brickworks and lime mortar made from aged lime putty with brick dust as pozzolanic addition and char coal, in composition similar to the original bedding mortar. Using this approach it was possible to complete the dome in a time period of six month. In particular the workability and performance of the new lime mortar was so successful that the local supervisors and craftsmen declared their intentions to use the material for other restoration projects as well.

KEYWORDS

Structural intervention, cultural heritage, micro structure, lime mortar, brick

1 INTRODUCTION

The mausoleum of the king Timur Shah, who reigned between 1773 and 1793, is an important witness of the past of Afghanistan. His father Ahmad Shah was able to pacify and unite the tribal Afghan leaders and to enlarge his territory. He fought successfully the Persians and the Moghuls. Under Timur Shah, however, the opposition of local tribal leaders increased and he was forced to move the Afghan capital from Kandahar to Kabul. The mausoleum was erected in the center of an extended garden in the year after the death of Timur Shah.

During the first Anglo-Afghan war (1839-1842) the monument was severely damaged. A misfired projectile launched by a British cannon hit the dome of the building. As a result over one third of it collapsed. Although repaired by Indian craftsmen on behalf of the British authority the reconstructed part collapsed again probably due to an earthquake. After that the masonry of the dome was in this condition until the year 2003. Solely in 1936/37 the entire dome including the hole was covered by metal sheeting in order to prevent any further rain water penetration.

The Timur Shah mausoleum exhibits a similar architectural style as the resting place of Ahmad Shah in Kandahar. The lower level of the building consists of an octagonal construction followed by a drum and dome (Fig. 1). The entire structure is built in brick, which delicateness of craftsmanship is in particular impressively exhibited in the cambered inner vault of the monument (Fig. 2). The drum below the dome has an inner radius of 6,85 m with a wall thickness between 120 and 130 cm. The dome itself consists of several shells of masonry, which appear in section like an onion skin. The base of the dome is formed by five shells and the number of shells is successively reduced at the top of the dome to two. The thickness of the shells is only one brick each and have not been joined together by binder bricks.

In 2002 it has been decided to improve the structural state of the building since it was suspected it would not survive another winter. Therefore options were discussed in how to preserve the monument ranging from completely dismantling of the dome and reconstruction in concrete to a restoration of the damaged part under consideration of the structural integrity of the entire building. Eventually the latter option has been realized after a thorough and careful assessment of the structural condition. A restoration consistent with the idea of using locally existing resources and craftsmanship as much as

Figure 1. The Timur Shah mausoleum in a historical photograph after the dome was damaged and repaired (Photo with courtesy of the Biblioteca Afghanica).

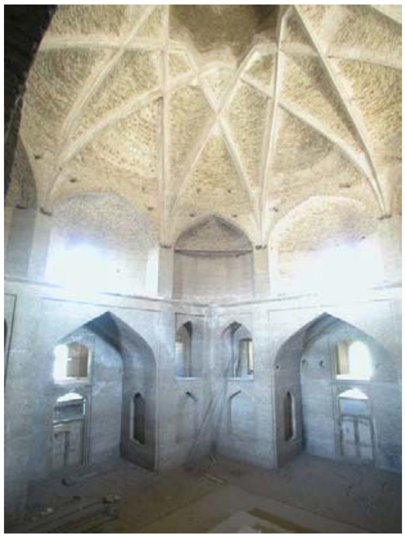

Figure 2. Inside the mausoleum with the cambered inner arch.

Figure 3. The damaged area from inside the dome. Clearly visible are the different shell layers (arrow)

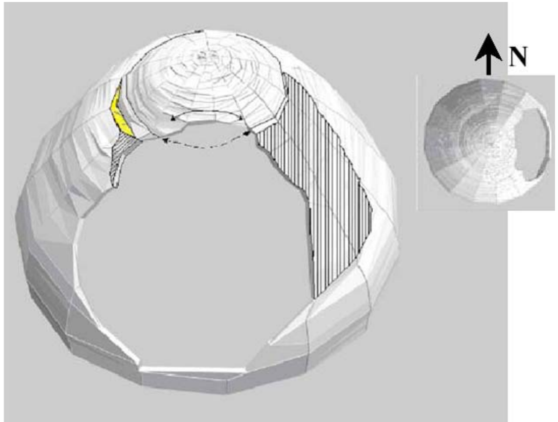

Figure 4. A CAD-reconstruction of the dome's geometry. The hatched areas indicate existing material from the first repair

possible in order to ensure a trouble free intervention and a sustainable maintenance afterwards. In this context it was crucial to use repair materials which were on the one hand compatible with the existing building materials but also locally available and easily worked by the hired craftsmen. For this reason the existing materials had to be examined in detail and based on the results appropriate repair materials had to be developed or chosen.

2 CONDITION OF THE MONUMENT IN THE YEAR 2002

The most critical problem concerning the structural condition of the building was the hole in the dome. It faced eastward and covered almost one third of the dome's total surface area (Fig. 3). Figure 4 shows that a substantial amount of material from the first repair was still existing at the time of the assessment. A visual examination conveyed that the bricks and mortar used for the repair were very different from the original ones: Both appeared to have a much higher strength and a different coloration. The contact joints of the remnants of the old repair to the original shells was clearly kinked. Obviously due to the impact the edges of the original masonry was deformed inwards. The early repair, however, could not restore the original geometry of the dome. The repair followed essentially the geometry of the deformed original parts of the dome. In the course of time many bricks did fall from the edge of the hole inside the dome and some were still loosely attached to it. The bricks were 20 x 20 x 4 cm in size with a average joint width of 2,5 cm.

Surprisingly most of the major cracking occurred in the drum under the hole and not at the dome. Many cracks went completely through the masonry. The crack widths on the inside showed values of up to 30 mm and opened to the outside according to the thickness of the wall. Sometimes the real width of the cracks was obscured by earth mortar from the last repair intervention in the 1940's. Some of the cracks were monitored by gypsum marks applied in 1998 or 1999. Unfortunately most of them fell off but the ones which stayed well attached showed no movement. Only at an open joint with a width of 3 cm a small crack appeared and indicated a horizontal movement of about 1 mm, which developed probably during the earthquake in fall 2001. But it seemed the majority of the cracks were old without any significant change.

The roofing of the dome showed many damages. A part of the metal sheeting was missing at the south. Most of the remaining sheets were deformed or exhibited bullet holes. The wood frame support for the metal sheeting was bleached by sun and water in exposed areas and many of the bonds of the laths and beams were weak because the nails were corroded. Exposed bricks and mortar were sometimes deteriorated due to frost exposure. But the overall condition of the bricks and mortars was essentially quite good, despite the fact that the original bedding mortar showed only a low strength.

3 ORIGINAL BUILDING MATERIALS

The analysis of the original building materials used for the mausoleum was essential in the process of developing appropriate repair mortars and for the choice of suitable brick materials. In the course of the analysis it was also evident, that traditional methods of mortar analysis (which are essentially not only for the analysis of cement mortars) could only be applied to a certain point due to the composition of the binders and aggregate of the mortars. In essence samples were taken from the original bedding mortar and from brick material. Additionally some samples were taken from a plaster capping the cambered vault and from remnants of a wall plaster of the octagon.

The bedding mortar was of gray color with many white and black inclusions of several millimeters in size. The black inclusions comprised of pieces of charcoal. The mortar exhibited many air voids and showed a fairly low strength. The micro texture of the mortar was very inhomogeneous (Fig. 5). Many inclusions of clay and charcoal were embedded in a lime binder matrix. The amount of capillary pores and air voids was high. Coarse aggregate above 1 mm existed almost entirely of charcoal fragments (Fig. 6). Other aggregate consisted of quartz, plagioclase, rock fragments, mica, organic matter and the clay lumps mentioned before. In the lime binder many lumps of lime were also present (white inclusions by naked eye observation) indicating the use of quick lime or freshly slaked lime together with poor mixing [Hughes et al., 1997, Hughes et al., 2001]. Remarkable was the high content of water soluble chloride, which ranged from 0.8 to 2.8 mass-%. XRD measurements showed that most of the chloride could be attributed to sylvite (KCl). Despite the fact that many clay lumps were worked into the mortar no direct evidence could be found that a pozzolanic reaction with the calcium hydroxide of the binder had occurred. The results of XRD designated kaolinite and illite as major clay minerals. Both are known for their non existent or only weak pozzolanic activity [Liebig and Althaus, 1997]. There was also no direct evidence from light or electron microscopy of reaction rims around clay lumps, which would have indicated a pozzolanic reaction. Therefore the mortar was characterized as a high lime product or a mortar with only weak hydraulic properties. The binder/aggregate ratio and the grain size distribution of the bedding mortar was determined according to Knöfel and Schubert [1993]. The original binder/aggregate ratio was with 1:1.8 (calculated for $Ca(OH)_2$) quite low but not surprising for a historical mortar, which is in accordance with findings from other authors [Callebaut, 2000; Sand and Franken, 1993; Wisser and Knöfel, 1988; Winnefeld, 1998]. The grain size distribution (Fig. 7) exhibited a very high amount of fines smaller than 63 µm. This was attributed to the presence of clay. Obviously an unwashed fine sand with a high silt and clay content was taken from the river bank nearby. It seems likely that charcoal was used in order to optimize the grain size

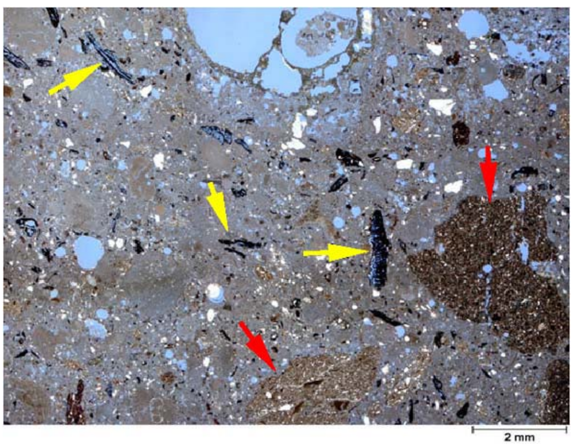

Figure 5. The micro texture of the bedding mortar in thin section with many inclusions of clay lumps (red arrows) and charcoal (yellow arrows). Pore space is indicated by a blue dye.

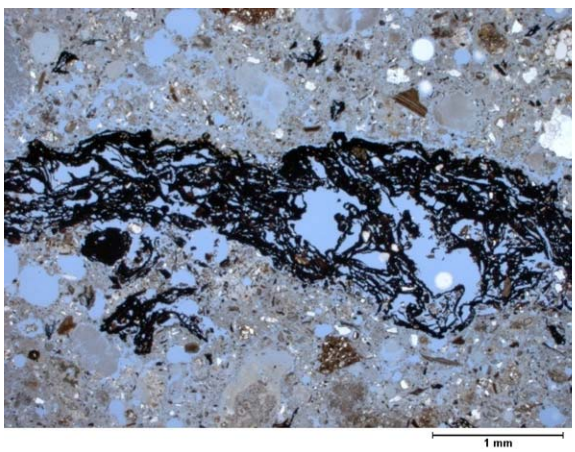

Figure 6. Close-up of a charcoal fragment of several millimeters in size (thin section photo micrograph).

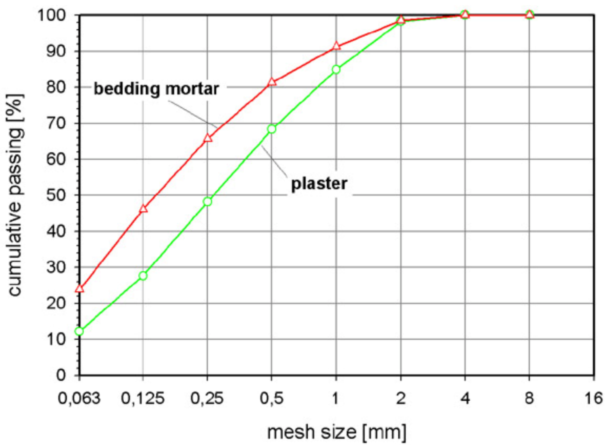

Figure 7. Grain size distribution of the bedding mortar and the plaster.

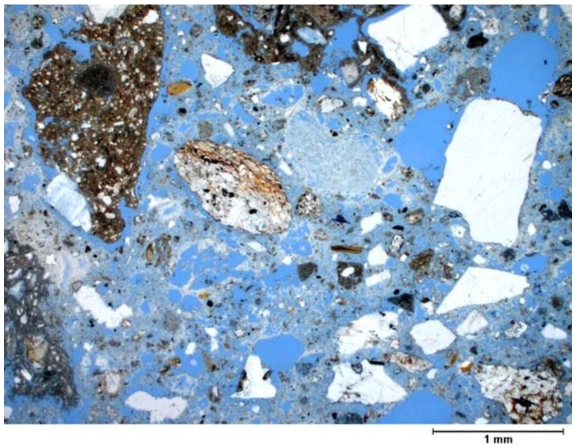

Figure 8. Photo micrograph showing the micro texture of a plaster sample.

distribution towards the coarser fraction. The high amount of fines had certainly caused a high water demand during mortar preparation which may explain the observed high number of capillary pores in the binder matrix. Uncommon were the physical properties. The bedding mortar exhibited a very low apparent density in the range of 1100 to 1150 kg/m^3. Total porosity was at 57 vol.-%. The total water absorption was exceedingly high with values between 38 to 48 mass-%. These data may be attributed mainly to the charcoal. It has been known that charcoal increases the capillary water absorption considerably [Goodman, 1998; Winnefeld et al., 2001]. Used in bigger aggregates it seemed to act as a lightweight aggregate in this mortar.

The plaster samples from the two locations were all similar in appearance and properties. Figure 8 illustrates a sample. The general texture was dominated by a high binder content of gypsum and some lime as well as a low amount of aggregate consisting mostly of limestone and siliceous compounds. Air voids and capillary pores were frequent but not as many as in the bedding mortar. The grain size distribution is shown in Fig. 7 with a similar shape to that of the bedding mortar (carbonate aggregate was not considered). It was not clear at what time the gypsum-lime plaster was applied. It might be possible that it derived from a later time period of the mausoleum. In total three brick samples have been investigated. The material was soft and of low strength. The mineralogical composition showed quartz, and feldspar as main constituents but also calcite and clays. The micro texture revealed additionally a high amount of organic matter in form of plant fibers, which were still in good condition. This indicated low burning temperatures of the bricks. Otherwise organic matter would have been burned and the constituents, in particular calcite, would have reacted to ceramic phases such as wollastonite, gehlenite or mullite as described in Cultrone et al. [2001]. As expected for hand made bricks the physical properties varied in a broad range. Total porosity was between 38 and 50 vol.-%, apparent density between 1430 and 1660 kg/m^3. Total water absorption amounted between 21 and 35 mass-%. According to Egermann and Mayer [1989] the variation in properties is due to the burning techniques in field kilns and the manual casting process.

4 CONSERVATION CONCEPTS

4.1 Repair materials

The results from the material analysis conveyed quite peculiar properties of the bedding mortar, such as the high porosity and the high amounts of fines in the grain size distribution. Due to the very low strength of the bedding mortar it was not possible to get meaningful results from mechanical testing. But the composition and the porosity suggested a compressive strength, which should not have exceeded 1 MPa. The bricks were also of low strength and partially very soft. The properties excluded therefore from the beginning the use of cementitious materials or a natural hydraulic lime as a mortar component. The strength would have been to high and with a lack in ductility where it was needed. Alternatively, the use of high lime as a binder for the bedding mortar seemed most appropriate.

Fortunately, there was a good local source of quicklime, which was manufactured in an archaic appearing brickworks (Fig. 9). The quick lime from this works was slaked and afterwards aged in barrels under water for at least eight weeks. After aging the putty was a smooth malleable paste which was used as a binder component for new bedding mortar. Field tests were performed in order to find a mortar formulation, which provided a good workability and adhesion as well as enough but not too much mechanical strength. Different trial mixtures were tested on site. Since equipment for the testing of workability and adhesion was not available on site small brick structures have been erected with inclined layers in order to observe the performance of the mortar mixes. This was also an opportunity for the local craftspeople to get used to processing the lime mortar. The final mortar mixture chosen consisted of binder-aggregate proportion of 1:2.5 (by volume) using crushed limestone. Charcoal has been added in order to reduce apparent density and enhance early mortar properties, such as water retention, adhesion, stiffening rate [Goodman, 1998]. Brick dust was added in low proportion as a pozzolanic addition for slightly increasing strength and durability in compensation to the addition of charcoal. Bricks were acquired from the local brickworks where also the quick lime originated and were in composition and physical properties similar to the original material. The new bricks could be manufactured in the size of the original material.

4.2 Structural conservation

The structural system of the dome has been approached by the membrane theory [Pflüger, 1981; Girkmann, 1936]. The resulting forces on top of the drum provided a magnitude of the horizontal forces (about 20 to.) straining and shifting the wall coping in the past and causing the cracks in the drum. The results allowed us to dimension and position “safety”-belts as temporary measures around the drum in order to keep the masonry stable for further work. Prefabricated belts with ratchets for loads about 10 to. each were installed. One essential concept was to establish a concrete bond beam inside the drum in order to redistribute the load evenly in the masonry. The masonry was tied back to the bond beam by anchors. The anchors were grouted into the wall using geo-textile socks between the grout and the masonry in order to prevent excess moisture from penetrating the wall sections (Fig. 10). Between the bond beam and the masonry foam boards were applied to give space for activating the anchors by turning the screws with a defined torque moment. As an accompanying measure all the cracks were cleaned and afterwards closed by pushing lime mortar as deeply and tightly as possible into the openings. Grouting of the cracks was not an option since there was no control to what part of the masonry the grouting material would flow.

Figure 9. The brickworks near Kabul.

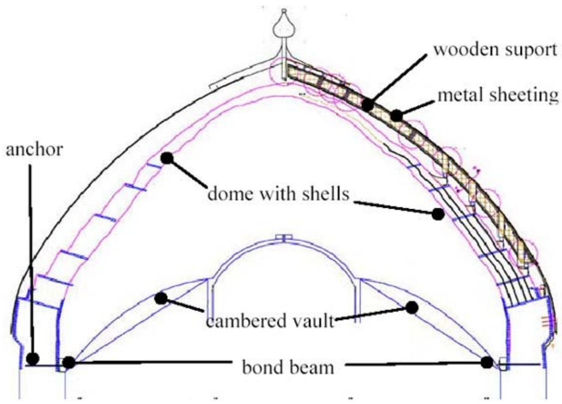

Figure 10. The repair solution with bond beam, wooden suport and metal sheeting.

The dome has been restored without a centering using lime mortar and the bricks and following the original shell structure (Fig. 11). Before starting the work all the edges of the existing masonry were cleaned and loose bricks were removed. The joints between old and new erected masonry were stuffed with mortar 6 months after closing the gap. However, it was not possible to regain a complete flow of force between new and old masonry. Therefore, additional heavy weight belts were applied on top of the ends of masonry shells in order to pre-stress the connecting joints and enhance the load bearing capacity for the new wooden support of the metal sheeting.

A new "shell" made of timber in form of a light construction, following the traditional geometry, covered with metal sheets will give sufficient and even more appropriate shelter to the old masonry in the future (Fig. 12). Proper ventilation under the wooden support is ensured in order to protect the timber from moisture, fungus and animal pests. The timber construction was calculated with the maximum of regular snow loads of 3,50 kN/m², which corresponds to the known maximum height of 90 cm fresh dry snow. All rafters are supported by vertical posts, which are set on sole plates. The horizontal forces will be transferred to the masonry only on top of the drum. No special regulations addressing design loads for domes exist for wind. Wind will not effect single members of the timber construction, because they are bonded by the boarding. But wind will effect the dome as a whole and try to lift it. The maximum wind loads according to DIN 1055, part 4, were used for the calculations. The resulting global safety factor of 2,66 is sufficient to prevent the timber construction from lifting in any case.

5 CONCLUSIONS

The Timur Shah mausoleum in Kabul, Afghanistan was restored under consideration of the existing structure. It was possible to recreate a much higher state of structural integrity as it was before. This was carried out using appropriate conservation materials, which were locally available and with the knowledge and ability of local craftspeople. No special or heavy equipment was used. Compared to a complete dismantling and reconstruction not only costs have been saved but also most of the original structure could be preserved.

6 ACKNOWLEDGMENTS

Figure 11. During the restoration of the dome.

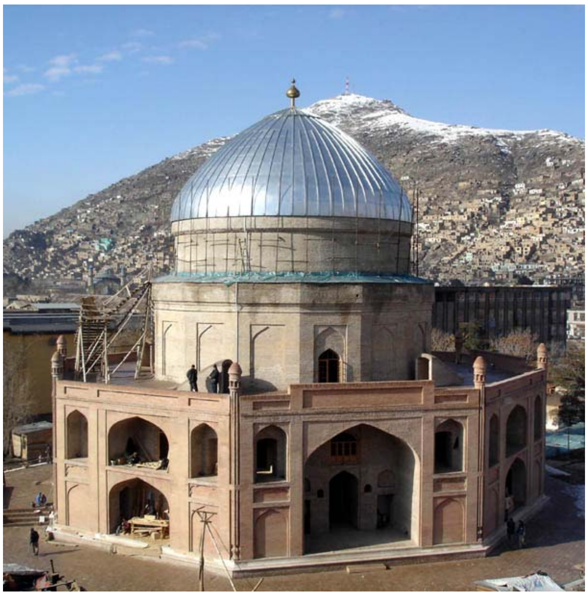

Figure 12. The mausoleum shortly after finishing the metal sheeting.

We would like to thank the Aga Khan Trust for Culture for their financial and logistic support and their permission to publish the results of the work. We want also thank Mary Hardy from the Getty Conservation Institute for reviewing the paper. And the authors are all aware that without the enthusiasm and skills of the local craftsmen and engineers the project would not have been performed successfully. Therefore many thanks to them!

7 REFERENCES

Callebaut, K. 2000, *Characterisation of historical lime mortars in Belgium: Implications for restoration mortars*, Thesis, Catholic University of Leuven, Leuven, pp. 239.

Cultrone, G., Rodriguez-Navarro, C., Sebastian, E. M., Cazalla, O. and de la Torre, M. J. 2001, Carbonate and silicate phase reactions during ceramic firing, *European Journal of Mineraolgy,* **13,** 621-634.

DIN 1055-4 1986, *Lastannahmen für Bauten; Verkehrslasten, Windlasten bei nicht schwingungsanfälligen Bauwerken*, Beuth Verlag, Berlin, Germany.

Egermann, R. and Mayer, K. 1989, Die Entwicklung der Ziegelherstellung und ihr Einfluss auf die mechanischen Eigenschaften von Mauerziegeln, In *Erhalten historisch bedeutsamer Bauwerke - Baugefüge, Konstruktionen, Werkstoffe* eds, Wenzel, F., Ernst & Sohn, Berlin, Germany, pp 107-130.

Girkmann, K. 1936, *Flächentragwerke,* Springer Verlag, Berlin.

Goodman, M. 1998, The effects of wood ash on lime plaster, thesis, University of Pennsylvania, Philadelphia, pp. 144.

Hughes, J. J., Bartos, P. M., Cuthbert, S. J., Stewart, R. N. and Valek, J. 1997, Microstructures in historic Scottish lime mortars, in *Aspects of stone weathering, decay and conservation: proceedings of the 1997 SWAPNET,* eds, Jones, M. S. and Wakefield, R. D. Imperial College Press, The Robert Gordon University, pp. 125-137.

Hughes, J. J., Leslie, A. B. and Callebaut, K. 2001, The petrography of lime inclusions in historic lime based mortars, in *Proceedings of the 8th Euroseminar on Microscopy Applied to Buildings Materials,* Athens, Greece, pp. 359-364.

Knöfel, D. and Schubert, P. 1993, *Handbuch: Mörtel und Steinerganzungsstoffe in der Denkmalpflege,* Ernst & Sohn, Berlin.

Liebig, E. and Althaus, E. 1997, Kaolinite and montmorillonite as pozzolanic components in lime mortars - untreated and after thermal activation, *ZKG international,* **50,** 282-290.

Pflüger, A. 1981, *Elementare Schalenstatik,* Springer Verlag, Berlin.

Sand, J. and Franken, S. 1993, Die Mörtel der Burgruine Hohenrechberg, in *Erhalten historisch bedeutsamer Bauwerke - Baugefüge, Konstruktionen, Werkstoffe,* eds Wenzel, F., Ernst & Sohn, Berlin, pp. 265-280.

Winnefeld, F. 1998, *Mauer- und Putzmörtel zur Restaurierung historischer Ziegelbauwerke,* Shaker Verlag, Aachen, Germany.

Winnefeld, F., Böttger, K.-G. and Knöfel, D. 2001, Historische Kalkmörtel - Restaurierungsmörtel für die Torhalle Lorsch, *Restauro,* **2001**[1], 40-45.

Wisser, S. and Knöfel, D. 1988, Untersuchungen an Historischen Putz-und Mauermorteln. Teil 2: Untersuchungen und Ergebnisse, *Bautenschutz + Bausanierung,* **11,** 163-171.

The Experience of *Sirena* Project: a case-study of a Building of Historic Center of Naples

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ABSTRACT

The maintenance of buildings and of single building components represents actually one of the most significant instruments for the care of building heritage, during the years.
One of the assumption of building maintenance and particularly of planning of maintenance activities is the evaluation of durability of building components, that is the capability of the single component to maintain, during the time, the technological services of its assemblage.
The individualization of single element service life cycle is fundamental part of the process of assessment of the time to intervene to realize the correct care of buildings.
In this period of big affirmation of programmed maintenance and of maintenance building instruments aimed to guarantee conservation, recovery and value of buildings, an initiative of the Municipality of Naples through Sirena Society is inserted to promote a recovery program of common parts of buildings with contributions without restitution.
The paper discusses *preventive maintenance* and maintenance instruments and focuses upon the analisis of defects and solutions of the floor covering element of a typical concrete building of the historic centre of Naples. It proposes an example of building *maintenance booklet*, with a *mainenance handbook* that suggests how to intervene, and a *maintenance planning*, that suggests when to intervene, to preserve building quality during the years.

KEYWORDS

Programmed Maintenance, *Maintenance booklet*, Diagnostic schedules

1 INTRODUCTION

The planning of all activities carried out to the building during its service life, with the aim to preserve its quality, the programmed maintenance, is a relevant argument, in the building national overview.
The maintenance idea is a part of the "project"; it introduces new action criteria, it changes obligations of all the subjects involved in the building project, it defines new maintenance strategies.
It is a real cultural revolution that involves, especially, engineers, that are obliged to obtain competence to plan maintenance. Time is a very important element to determine all the things which define different maintenance strategies, control and intervention time and decisional models of building maintenance.
The result is that building maintenance cannot represent a marginal phase of the productive process. Maintenance is not to be considered like a summary of reparative intervention but the totality of all the activities, which are inside the building productive cycle, able to maintain quality and functionality of buildings.
Programmed maintenance is based on scomposition of the building in single building components; here UNI 8290 introduces and sets out a classification of the technological system, dividing components in: technological units classes, technological units and technical elements classes.
With particular reference to floore coverings, which are closings (how buildings are separated from the external context), the Regulation differentiates, regarding to technological units, vertical closings, inferior horizontal closings, horizontal closings on external spaces, superior closings.
A superior closing is a technological element, of a horizontal and sub-horizontal type, that separates the building from the external spaces; these are intended as various external agents that are causes of degrade of buildings and of building components. For example: sun, rain, wind, snow; and other unexpected events or things that may cause defects and anomalies of building system.
At the end of the paper, a program of maintenance building activities criteria will be presented; in this program, these activities have to be done during building service life, starting from the analysis of pathologies, through the measurement and the analysis of the correspondent anomalies, arriving to the preview recurrence of the same pathologies.
Maintenance activities (controls and interventions) have to be object of prevision and planning with the definition of an actions plan aimed to maintain building quality during the years.
Generally, that means to realize maintenance plans or, more specifically, maintenance instruments such as the *maintenance booklet*, made in the occasion of the *Sirena Project*, a program of restoration of common parts buildings, of Municipality of Naples.
This instrument is composed by two different sections:

- the Anagrafe section;
- the maintenance planning section.

In the first one there are building characteristic data (geometrical, morphological, technological).
The second one is divided into other three sections, that are:

1) maintenance handbook, (which defines how to intervene), constituted by control and intervention schedule, about maintenance actions;
2) maintenance planning (which defines when to intervene), that is a planning, a calendary of control and intervention times;
3) use handbook, that is an instrument that contains an analysis of the principal anomalies for building components, the frequency of time they happen, a photographic documentation, a reference to UNI Code.

After this introduction on maintenance and maintenance instruments, the paper aims to describe the most important pathologies on floor coverings of a building of historic centre of Naples, to define the most adequate intervention adopted to repristinate the performance efficiency of the floor covering element, to individuate all the maintenance activities that have to be done, during building service life, to preserve quality of the whole building and of its single elements.

2 CAUSES THAT MAKE NECESSARY MAINTENANCE ACTIONS

Before reporting about building maintenance activities, concerning coverings, it can be useful a notice to the principal causes that make necessary maintenance actions.
Physiological obsolescence of a building, that is a natural aging due to physiological factors, could be considerate as a consequence of other factors that determine, in different times, the progressive lowering of materials and components quality levels.
In particular, the most common types of building obsolescence are: physical obsolescence that concerns physical, chemical and mechanical materials performances, functional obsolescence and technological obsolescence. The functional obsolescence concerns buildings components functions.
Particularly, these components can't do their traditional functions, for which the were designed: this phenomenon is due to transformation not foreseen in the project or in case of modification of use destination of the building.
Technological obsolescence is connected to technological innovation processes; this is a component substitution that can cause pathological phenomenon but can determine also advantages because of the introduction in building market of new products, which are more satisfactory and with the same prices.
Unlike of natural aging, that is a foreseeable process, pathological degradation, manifests itself with different times and effects, that are not prevedible during the element service life.
In this category there are defects, damages, caused by degradation agents which provoke building service life, reducing its length during the years.
Sometimes, pathological defects and natural aging forms are two parts of the same process: the first one as a degeneration of the second one.
The components characteristics and the materials properties allow an evaluation and prevision of degradation phenomenon, connected to the building aging process, so that control and intervention activities could be programmed to contrast effects which are negative to building functionality.
As a consequence, preventive maintenance intervention are better than after breakdown maintenance because of the fact that the first ones are able to anticipate the pathology manifestation; in the second case, these types of actions are following to the pathologies manifestation that can cause damages rather serious.
The more recurrent degradation factors are related to atmospheric agents action, to accidental events, to maintenance intervention absence or to the same building utilisators that are not able to utilize well the building, causing damages to materials and building components.
Other factors that can cause building pathologies and degradation phenomena are:
- technological solutions not adequate to the building service conditions;
- not suitable materials and products;
- wrong evaluation of environmental factors;
- defects about constructive details information.

Concerning degradation factors due to maintenance defects, it is important to observe that this type of factors can cause a worsening defect, the appearance of new pathologies, the damaging of components.
As a consequence, there is a very strong connection between building components service life and programmed maintenance intervention. In particular, the knowledge of building elements duration suggests the datum about planning of buildings maintenance actions (controls and intervention; the temporal frequency to do actions has to be introduced in the maintenance planning.

3 COVERINGS BUILDING PATHOLOGIES AND INTERVENTION CRITERIA: A CASE STUDY

The building has a good state of conservation; however there are some degrade situations, referred to the floor covering element, that cause problems to the whole building.
With reference to coverings, it is important to define that the technical solution (photo 1) is composed by a waterproof membrane with a superior coat of polyurethane; particularly a zone of the terrace is lagged by another waterproof membrane coat.
Regarding to polyurethane, it caused volumetric expansion phenomena that generate firstly a loss of capability to guarantee impermeability to the whole solution, and also the impossibility to execute maintenance action because of the fact that polyurethane is an inflammable material and so it is impossible to do partial substitutions of membrane (generally a membrane is placed with a gas propane blowlamp).

Figure 1. A view of the terrace

Other problems are:

- decrease of the union (for rainwater discharge) section, caused by the polyurethane inside;
- wrong positioning of parapet elements;
- wrong configuration of weathering;
- lack of adequate coat of waterproof membrane under the marble batten that runs through the terrace perimeter;
- diffuse crocodiling (surface cracks) and chromatic alteration on large parts of the terrace.

With reference to the intervention criteria proposed, it is important to precise that the polyurethane coat, applied with insulating materials and with waterproof functions, has to be removed.
The removal result will allow the evaluation of weathering characteristics.
Particularly the following measures are suggested:

1) adequate organization of inclination along the terrace modifying the watershed lines;
2) increase of the entrance sections of pluvious that present a section of 80 mm while generally pluvious have a section of 125 mm.

Concerning the floor covering solution, considering the necessity to guarantee the adequate water insulation, the desirable solution are of two types: one, a more classic intervention, is the following (starting from floor and going on toward):

- an expanded clay slope footing;
- a slim footing (cement);
- double expanded polystyrene layer (20 mm);
- a slim footing;
- a double waterproof membrane layer;
- an acrylic paint layer.

The second one is a *reverse-roof* type:

- an expanded clay slope footing;
- a slim footing;
- a double waterproof membrane layer;
- a double expanded polystyrene layer (20 mm);

- an NT layer put down without adhesive;
- a gravel layer.

In conclusion, the adopted solution is the *reverse-roof* one.
This type of solution presents the waterproof membrane under the insulating layer; particularly the external surface is represented by a coating in ceramic, stone, gravel, else.

4 MAINTENANCE ACTIVITIES

It is evident that controls and interventions maintenance activities, and after these a maintenance planning, are strictly connected with the technical solutions adopted for each building component, as individuate in UNI 8290-1. Particularly, for floor coverings, and with specific reference to the building in centre of Naples, the technical solution adopted for the covering element is reported in the sequent table (see UNI 8290-1).
The first column contains the definition of the layers functions, starting from the intrados of the floor, the second one refers to the materials of each layer.

FUNCTION	TECHNOLOGICAL CARACHTERISTICS
Support	Expanded clay slope footing
Waterproof coating	Double waterproof membrane layer
Heat-insulating element	Double expanded polystyrene layer (20 mm)
Filtering layer	NT layer put down without adhesive
External coating	Concrete floor (tiles)

With reference to this technical solution the most recurrent anomalies are as following:

1) Surface alteration
2) Crackles, scores
3) Marks
4) Superficial deposits
5) Anchors and joints degrade
6) Surface vegetation
7) Limes
8) Biological attacks
9) Partial element detaches
10) Elements loss

With reference to all these anomalies, and in this case to the superficial coating in concrete floor, there have been disposed diagnosis and planning schedules, described as following.

4.1 Diagnosis Schedules

The diagnosis schedules (fig. 1) contains:

- a description of the examined technical solution;
- the anomaly definition;
  near the photo,
- the current localization;
- the identifiable causes;
- the caused performance degrades;
- the most important connected UNI Regulation;
- notes.

SCHEDA DI DIAGNOSI

COPERTURE (RIFERIMENTO NORMA UNI 8290: 3.2.4.1)	CO2/VA D4

E

A

DESCRIZIONE DELLA SOLUZIONE TECNICA RILEVATA:

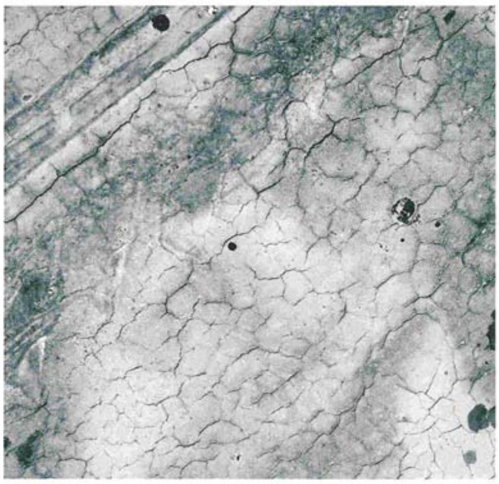
A. (STRUTTURA PORTANTE): calcestruzzo
B. (MASSETTO DI REGOLARIZZAZIONE E PENDENZE): cls alleggerito
C. (FISSATIVO PER MANTO IMPERMEABILE): *Primer* bituminoso
D. (STRATO DI TENUTA): MEMBRANA IMPERMEABILE
E. (STRATO DI PROTEZIONE DEL MANTO): vernice acrilica

ANOMALIA: COCCODRILLATURE
(Formazione di cavillature e fessure superficiali)

LOCALIZZAZIONE RICORRENTE:
diffuse lungo lo strato superficiale del manto impermeabile

CAUSE INDIVIDUABILI:
obsolescenza fisiologica
difetti di produzione e/o esecuzione
atmosfere aggressive
agenti atmosferici

DECADIMENTI PRESTAZIONALI:
aspetto
funzionalità della membrana impermeabile



PRINCIPALI NORME DI RIFERIMENTO:
UNI 9307-1 30/09/88 Coperture continue. Istruzioni per la progettazione. Elemento di tenuta.
UNI 8178 30/11/80 Edilizia. Coperture. Analisi degli elementi e strati funzionali.
UNI 8627 31/05/84 Edilizia. Sistemi di copertura. Definizione e classificazione degli schemi funzionali, soluzioni conformi e soluzioni tecnologiche.
UNI 8202-1 30/09/81 Edilizia. Membrane per impermeabilizzazione. Generalità per le prove.
UNI 8202-20 02/10/87 Edilizia. Membrane per impermeabilizzazione. Determinazione del coefficiente di dilatazione termica lineare.
UNI 8202-21 31/03/84 Edilizia. Membrane per impermeabilizzazione. Determinazione dell'impermeabilità all'acqua.
UNI 8202-24 31/07/88 Edilizia. Membrane per impermeabilizzazione. Determinazione della resistenza all'azione perforante delle radici.
UNI 8202-28 30/04/84 Edilizia. Membrane per impermeabilizzazione. Determinazione della resistenza all'ozono.
UNI 8202-30 30/04/84 Edilizia. Membrane per impermeabilizzazione. Prova di trazione delle giunzioni.

NOTE: questo tipo di anomalia si manifesta sotto forma di microfessure a ragnatela che investono lo strato superficiale del manto di tenuta

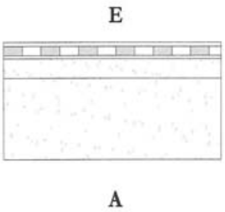
Figure 1 – a diagnosis schedule example

4.2 Planning Schedules

The planning schedules (fig. 2) refer to other schedules, the maintenance action description schedules (controls and intervention) that represent, with the planning of all the actions during the time, the contents of the maintenance file (*Sirena* Project). The schedule has two fundamental fields: the first one concerning the illustration of the examined technical solution. The second one, for each anomaly individualized on the external coating of the technical element, are reported:

- the anomaly manifestation during the years;
- the maintenance actions typology (controls and intervention) that can be don in relation to the observed anomaly and to its importance;
- the intervention verge during the time, referring to each maintenance action.

DESCRIZIONE DELLA SOLUZIONE TECNICA RILEVATA – TIPO CO2-VS:



A. (STRUTTURA PORTANTE): calcestruzzo
B. (MASSETTO DI REGOLARIZZAZIONE E PENDENZE): cls alleggerito
C. (FISSATIVO PER MANTO IMPERMEABILE): *Primer* bituminoso
D. (STRATO DI TENUTA): membrana impermeabile
E. (STRATO DI PROTEZIONE DEL MANTO): vernice acrilica

SCHEDA N.

CO2 VS

ANOMALIA	FREQUENZA DI ACCADIMENTO (ANNI)	AZIONE			FREQUENZA CONSIGLIATA (ANNI)
ALTERAZIONE SUPERFICIALE	2-3	CONTROLLO	VS-*cn*	Controllo a vista per verificare la presenza di depositi e/o alterazioni superficiali	1-2
		INTERVENTO	VS-*in*	Pulizia dello strato superficiale	6 mesi
		INTERVENTO	VS-*in*	Rinnovo parziale dello strato di pittura	2-3
		INTERVENTO	VS-*in*	Rinnovo totale dello strato di pittura	4-5
BOLLATURE E RIGONFIAMENTI	---	CONTROLLO	VS-*cn*	Controllo a vista per verificare la presenza di bollature e rigonfiamenti	2
		CONTROLLO	VS-*cn*	Controllo per verificare le condizioni della membrana	2
		INTERVENTO	VS-*in*	Eliminazione delle bolle superficiali	AO
		INTERVENTO	VS-*in*	Asciugatura dell'umidità	AO
		INTERVENTO	VS-*in*	Sostituzione parziale dello strato di membrana	5
		INTERVENTO	VS-*in*	Sostituzione totale dello strato di membrana	15-20

Figure 2 – a planning schedule example

For example, with reference to the surface alteration, that is in the first column of the figure 2, the time of manifestation is about two or the years; the maintenance actions, that are controls and interventions, suggested are:

for the controls,
- a view control to verify the surface deposits presence

for the interventions,
- a surface layer cleanliness
- a partial renovation of the layer paint
- a substitution of the layer paint

Finally, on the right side of the table, the intervention verge during the time is suggested; for the control, about one or two years; for the interventions, respectively, six months, two or three years, four or five years.
Naturally, all the elements of the table, anomalies, times of manifestation maintenance actions and the intervention verge, changes with different surface layer (concrete tiles, ceramic tiles, etc).

5 CONCLUSION

Preventive maintenance aims to preserve building quality and building quality components, during the years; it means that maintenance actions, controls and intervention, made at fixed dates, make possible to intervene before breakdown manifestation, with advantages of economic and management type.
With reference to floor coverings, and particularly to floor coverings maintenance actions, it is important to underline that maintenance controls and interventions are related to the surface layer of the technical solution, that doesn't mean necessary the waterproof membrane. Particularly, in the case of the *reverse-roof*, adopted for the examined building in Naples, this type of solution presents the waterproof membrane under the insulating layer, the external surface is represented by a coating in ceramic, stone, gravel, else.
As a consequence, maintenance actions are connected to the surface tiles; *preventive maintenance* of this surface layer has to guarantee a good preservation of the waterproof membrane. However, the maintenance plan, in this case, has also to establish maintenance actions which concern the waterproof membrane, considering its more probable duration in time. A partial renovation and a total substitution of the membrane layer have to be done respectively within five years, in the first case, fifteen or thirty years, for the last case.

6 REFERENCES

AA.VV., *Manuale di progettazione edilizia Fondamenti, strumenti, norme, Vol. IV: Tecnologie – Requisiti, soluzioni, esecuzione, prestazioni,* HOEPLI Editore, Milano

AA.VV., *Impermeabilizzazioni: coperture continue, codice di pratica,* BE-MA Editore, Milano

Arbizzani E. 1991, *Manutenzione e gestione degli edifici complessi*, Hoepli, Milano,

Croce S. 1981, *Affidabilità del sistema tecnologico edilizio. Patologie in edilizia*,

D'Alessandro M. (a cura di) 1994, *Dalla manutenzione alla manutenibilità*, Milano

Daniotti B., Maggi P. N. 1993, *Valutazione della qualità tecnologica caratteristica dei prodotti complessi per l'edilizia,* Quaderno DISET, vol. 1 e 2,

Di Giulio R. 1991, *Qualità edilizia programmata,* HOEPLI, Milano,

Di Giulio R. 2000, *Manuale di manutenzione edilizia,* Maggioli,

Lee R. 1993, *Manutenzione edilizia programmata: Strategie, strumenti, procedure*, Hoepli, Milano,

IMPER Italia S.p.a., *Impermeabilizzazione ed isolamento termico – Schemi esecutivi*

Maggi P. N., Gottfried A., Morra L. 1992, *Valutazione della qualità tecnologica dei prodotti complessi per l'edilizia*, Quaderno DISET, vol. 1,

Molinari C. 1989, *La manutenzione in edilizia*, Angeli, Milano,

Nicolella M. 1997, *Valutazione semi-probabiliatica del comportamento in servizio di elementi costruttivi per la determinazione dell'affidabilità* (tesi di dottorato), Napoli,

Nicolella M. 2000, *Affidabilità e durabilità degli elementi costruttivi in edilizia*, CUEN, Napoli,

Nicolella M. 2003, *Programmazione degli interventi in edilizia - Guida al libretto di manutenzione del fabbricato*, UNI, Milano

Rejna M., *Valutazione della qualità tecnologica utile dei prodotti complessi per l'edilizia* 1995, Quaderno DISET, vol. 4

Russo M. G., Iovino R., Fascia F. Marinelli D. 17-21 marzo 2002, *Programmed maintenance procedures of staircases historical buildings in the Ancient Center of Naples IX DBMC International Conference on Durability of Building Materials and Components, Brisbane Convention & Exibition Centre, Queensland, Australia*

Talamo C. 1998, La manutenzione in edilizia, Maggioli

UNI 8088 - Lavori inerenti le coperture dei fabbricati. Criteri per la sicurezza

UNI 8089 - Edilizia. Coperture ed elementi funzionali. Terminologia funzionale

UNI 8090 - Elementi complementari delle coperture. Terminologia

UNI 8091 - Coperture. Terminologia geometrica

UNI 8178 - Edilizia. Coperture. Analisi degli elementi e strati funzionali

UNI 8290-1/1981 - Edilizia Residenziale – Sistema tecnologico – Classificazione e terminologia

UNI 8290-2/1983 - Edilizia Residenziale – Sistema tecnologico – Analisi dei requisiti

UNI 8290-3/1987 - Edilizia Residenziale – Sistema tecnologico – Analisi degli agenti

UNI 8627 - Sistemi di copertura. Definizioni e classificazioni degli schemi funzionali, soluzioni conformi

UNI 10838 - Edilizia – Terminologia riferita all'utenza, alle prestazioni, al processo edilizio e alla qualità edilizia

A Powerful Simulator of Moisture Transfer in Historic Building

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ABSTRACT

Moisture is one of the most deteriorating factor of buildings. New tools are needed for the ability of building, its parts, components and materials to increase their durability against moisture action. In this work a simulator of moisture transfer in building is presented, when main source of moisture is ground water. Main objectives of this tool are the prognosis of moisture content and monitoring of masonry over time, as well as the decision-making on compatible restoration materials and techniques in order to increase mansory durability.
This simulator takes into account (a) the moisture transfer mechanisms to and from the building (capillary rise, drying, etc); (b) the wall configuration (materials and size); (c) the construction materials properties (d) the seasonal region meteorological data (air temperature, humidity and velocity); (e) the ground water salinity; and calculates: (a) the seasonal wall moisture content along with the corresponded equilibrium moisture height; (b) the capillary rising water flow rate; (c) the wall drying flow rate (d) the salt deposition flow rate; etc. The simulator has been developed in an Excel platform in a user-friendly environment and consists of four units: (a) the process model; (b) the problem solution algorithms; (c) the database; (d) the graphics interface.
The proposed simulator is a powerful tool in decision making concepts concerning the building deteriorating evolution and the selection of the appropriate protecting strategy, e.g., the plaster selection (material, size, replacing time). With the use of this tool, the assessment of the effectiveness of restoration materials before their real application can be made, contributing to the extension of masonries lifetime, but as well to the attenuation of cost and time waste of restoration works.

KEYWORDS

Moisture transfer mechanisms, materials properties, decision making tool

1 INTRODUCTION

Premature failure of building envelopes, prior to achieving their design service life, has become a major problem within the last twenty years. The service life of masonries is affected by a numerous factors, such as micro/macro environmental conditions, materials properties, and envelope design. Moisture, in all its forms, consists one the most deteriorating and decay factors of buildings. Moreover, moisture is the most prevalent cause of deterioration of historic masonries [Durability guidelines for building wall envelopes, 1997].

Moisture accumulating, not only reduces the durability of building materials, but also degrades indoor air quality and thermal performance [Karagiozis, 2003].

Thus, the building envelope restoration suffering from moisture problems is one of the critical key issues, in sustainable conservation strategies. The incompatible materials and techniques that are used in many cases accelerate the degradation process. There is a growing need for new tools development, which will contribute in long-term innovation planning, for historic preservation research.

Moisture transfer in buildings is a very complex process and is influenced by a lot of physical phenomena. In the literature there are a lot of computer –based tools, aiming to predict the long-term hydrothermal performance of buildings. A review made by the Canadian Mortgage and Housing Corporation, found forty-five computerized hydrothermal modeling tools [Review of Hydrothermal Models for Building Envelope Retrofit Analysis]. Suggestively are referred the following:

The **WUFI-StOpStar** and its family of predecessor **WUFI** and **WUFI-ORNL/IBP** is a menu-driven PC program that calculates the transient hydrothermal behavior of multilayer building components exposed to a set of climatic conditions. The **LETENITE-VTT** is an enhanced version of the original **LATENITE** model. **LETENITE-VTT** includes not only the building envelope solver, but also a capability to simulate the interactions between the building envelope and the indoor air by solving the whole building energy and mass balance [Geving et al, 1997]. The **MOIST** is a PC program, for predicting the one-dimensional transfer of heat and moisture in building envelopes. It enables user to define a wall, cathedral ceiling, or low-slope roof construction and to predict the temperature and moisture content (or relative humidity) of the individual construction layers as a function of time of year [Burch & Chi, 1997]. The **UMIDUS** is a PC program, for the prediction of heat and moisture transfer in porous building elements [Mendes et al., 1999].

In this work a simulator of moisture transfer in buildings is proposed. Rising damp is considered as the main source of moisture. The simulator is based on a mathematical model and uses the advantages of the Excel software. The user interface contains input data for the defined parameters of the problem, and reviews the results. These results concern the masonry moisture, as well as, the salt content of masonry, depending on the construction materials properties, and the environmental conditions. Therefore this simulator is an effective tool for decision-making on the compatible restoration materials and techniques for the protection of masonries and expansion of their lifetime.

2 SIMULATOR SCOPE

The development of this simulator intends to contribute to the solution of masonry moisture problems effectively. More specifically aim of this tool is the:

- prognosis of the seasonal wall moisture content along with the corresponded equilibrium moisture height
- decision-making on effective restoration materials and techniques concerning moisture problems
- increase of masonry service lifetime
- comparison between alternative restoration materials.

3 SIMULATOR PROCESS MODEL

The simulator model has been developed for a masonry with two layers. The main masonry layer consists of brick or stone and the second layer of plaster. The moisture wetting mechanisms taking into consideration are capillary suction and sorption, while the drying process evaporation and absorption.

The process model uses simple parameters and can calculate the masonry moisture content profile, depending on the material used as well on the environmental conditions. More specifically the Simulator takes into account:

(a) **The construction materials properties**. The Simulator process model uses experimental values for materials; porosity, water absorption-desorption isotherms (10-90% of relative humidity), capillary rise coefficient, drying kinetics. The materials properties data form look-up tables.

(b) **The moisture transfer mechanisms to and from the building**. A mathematical model is developed and it describes the moisture transport phenomena taking into consideration; capillary rise and drying kinetics and the water sorption-desorption isotherms.

(c) **The wall configuration**. For the masonry description is needed, its dimensions (height and width), and the construction materials used.

(d) **The seasonal, regional meteorological data (air temperature, humidity and velocity).** The masonry moisture equilibrium is depended strongly on the environmental conditions. Weather files are created that contain daily data of temperature, relative humidity and wind speed.

(e) **The ground water salinity.** The soil moisture is usually high in salts; consequently rising damp increases progressively the salt content of masonry. The action of soluble salts is very destructive and manifests during the wetting-drying cycle of masonry [Theoulakis & Moropoulou, 1999]. The salinity of water is described in ppm varying from 0-30000ppm for seawater.

4 SIMULATOR ARCHITECTURE

The Simulator has been developed on Excel workbook Microsoft Excel spreadsheets, which offer sufficient process hospitality. They are connected easily and on-line with charts and graphic objects, resulting in powerful and easy-to-use graphical interface. Excel also supports mathematical and statistical tools. Databases are effective and easily accessed. In addition, Visual Basic for Applications (which is included in the new version of Excel) offers a powerful object-oriented programming [Maroulis & Saravacos 2003].

Four different units can be distinguished, developed in different sheets:

- Databases worksheet
- Process model worksheet
- Problem solution algorithms
- Graphics interface worksheet.

4.1 Databases worksheet

This sheet contains all the data needed for calculations in the form of Data lists. The data could be extended or modified via appropriate dialogue boxes. The following databases are developed:

- Construction materials properties; microstructural data, capillary rise kinetic data, drying kinetics data, sorption-desorption isotherms

- Meteorological data; variations of air temperature, relative humidity and velocity. Data concerning rain and solar radiation are not included, because the present process model does not take these kinds of data in consideration.

4.2 Process model

This is the heart of the system calculations. It contains the process model. The model solution uses only worksheet functions. When any changes in input variables occur, the solution is obtained automatically on this worksheet.

4.3 Problem solution algorithms

The solution of different problems is based on the operational program of the process model worksheet above, and uses the Solver of Excel via Visual Basic program, to obtain the solution.

4.4 Graphics interface

This is the only method for man-machine communication. The graphic interface will essential consist of three parts:

- Problem formulation. The specifications and the required data for the problem to be solved are entered by the user or estimated from the databases. Data are inserted via dialogue boxes or buttons for changing some important magnitudes. The specifications will consider the wall configuration that means the type and dimension of the masonry, the ground water characteristics, meteorological data etc.

- Results presentation. The results will be obtained automatically and are presented in the form of tables or graphs.

The Simulator interface is shown in Fig. 1.

As it is shown in Fig. 1 the Graphics interface contains charts (symbolized as C in comments), drop down menus (symbolized as DDM in comments) and scroll bars (symbolized as SB in comments). The graphics interface is friendly to the user.

(I)Problem specifications / Problem type selection. With the use of **DDM1** and **SB1** the following five parameters are specified: the masonry construction material (now is selected BRM brick), the restoration plaster (now is selected the PRL), the masonry width (now is 45cm), the plaster application width (at the moment is 5cm) and the masonry height (currently is 400cm). The selected materials are marked automatically in chart **C1** with circles, while the selected masonry dimensions are presented in chart **C2**. With the use of **SB2** the ground water total soluble salts concentration in ppm can be selected (now is selected 25000ppm). The **SB3** scroll bars are used for input of ambient conditions. Also the month of the year, the temperature, the relative humidity and the air velocity can be selected. The changes of air temperature and relative humidity per month are shown in chart **C4**.

Chart **C1** describes the moisture transport coefficients, the capillary rise height time constant, t_c(d), and the drying constant t_d(hrs) for all materials of the database. This specific chart gives a first idea about for moisture performance of building materials and contributes decisively on the selection of the more hygrometric compatible restoration material. All the materials codes starting with a P are plasters, S stones and B bricks.

The two constants derive from first order kinetic models describing capillary rise phenomenon and drying kinetics [Karoglou et al., 2004].

(II) Results presentation. The results are presented in the form of charts. The variation of moisture content is represented in chart **C3**, while the height of moisture front, as well as the salt concentration profile in chart **C2**.

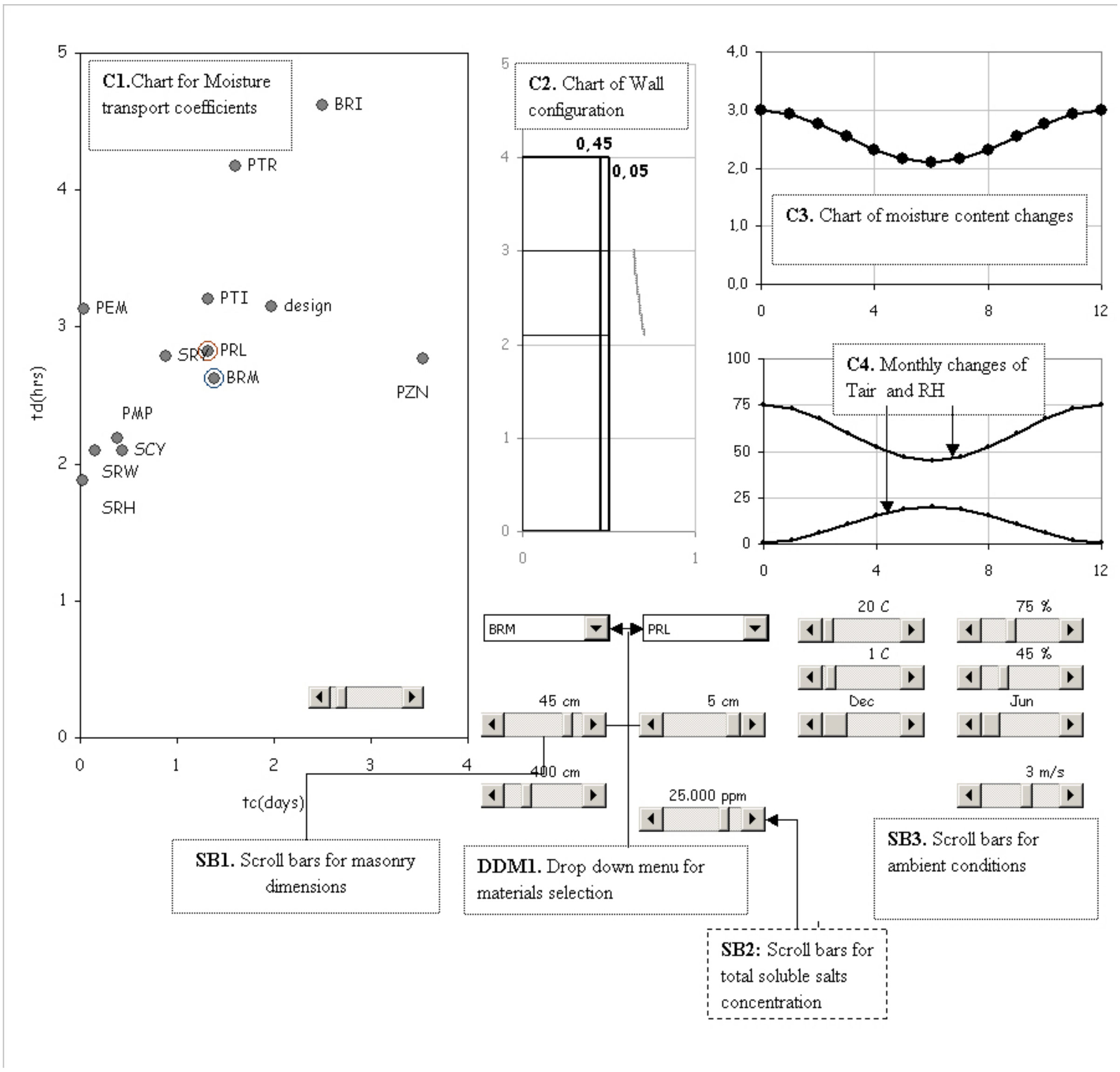


Figure 1. Simulator interface and comments

5 CASE STUDY

Development of moisture content could lead to improvements in the durability of the materials and components. If a balance between wetting and drying is maintained moisture will not accumuclate over time and moisture related preblems will mitigate [Straube, 2002]. For assesing the risk of moisture damage the wetting and drying process should be considered. The appropriate restoration materials choice is one way to control the masonry moisture equilibrium.

A case study for selecting the most appropriate restoration plaster for a specific masonry follows. A historic masonry is considered. The masonry is constructed either with brick or stone. It is supposed a masonry M_1 made with brick BRM and a masonry M_2 made with stone SCY. The two masonries have the same dimensions. Their moisture height before the plaster application is shown at Fig.4 (i), Fig.5 (i). The environmental conditions are shown in Fig.3 and the air velocity selected is 3m/s.

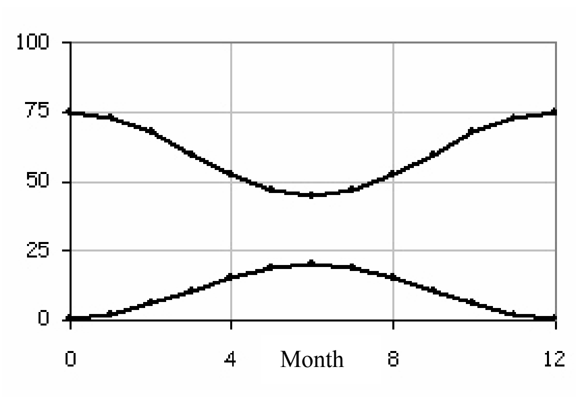
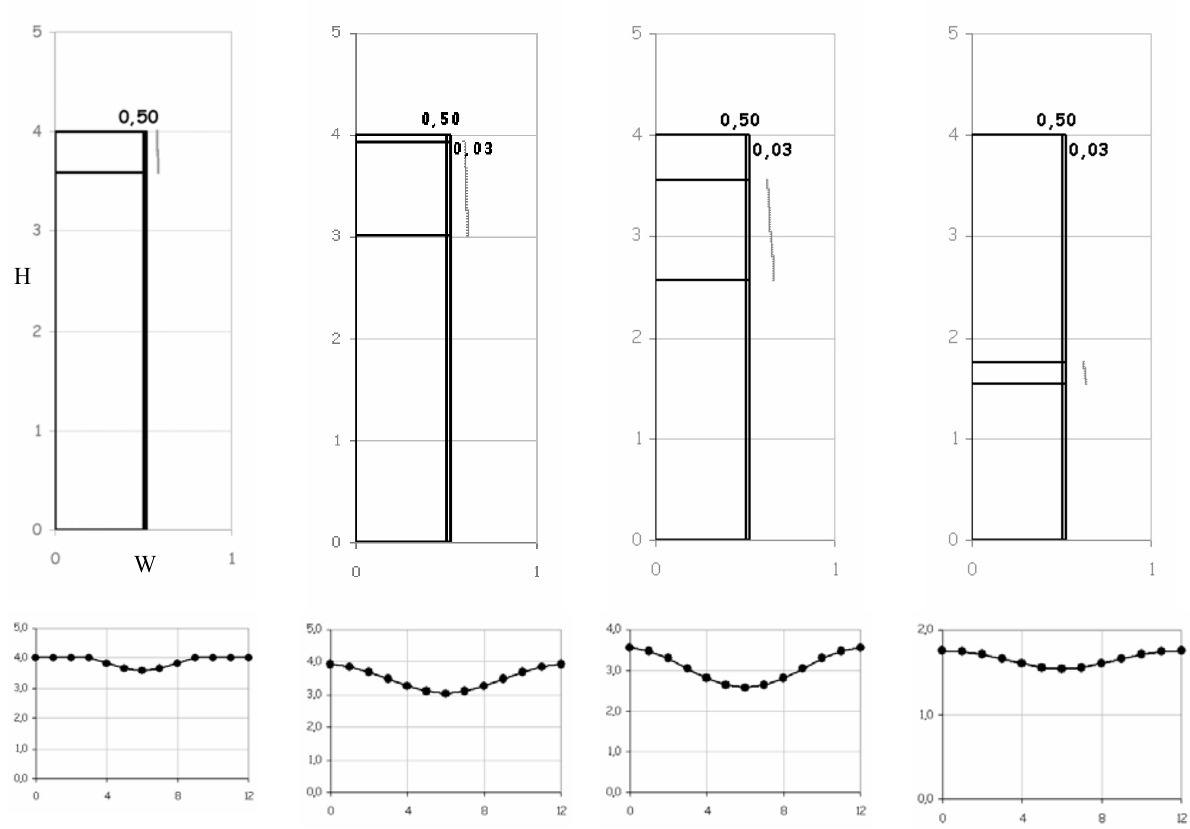


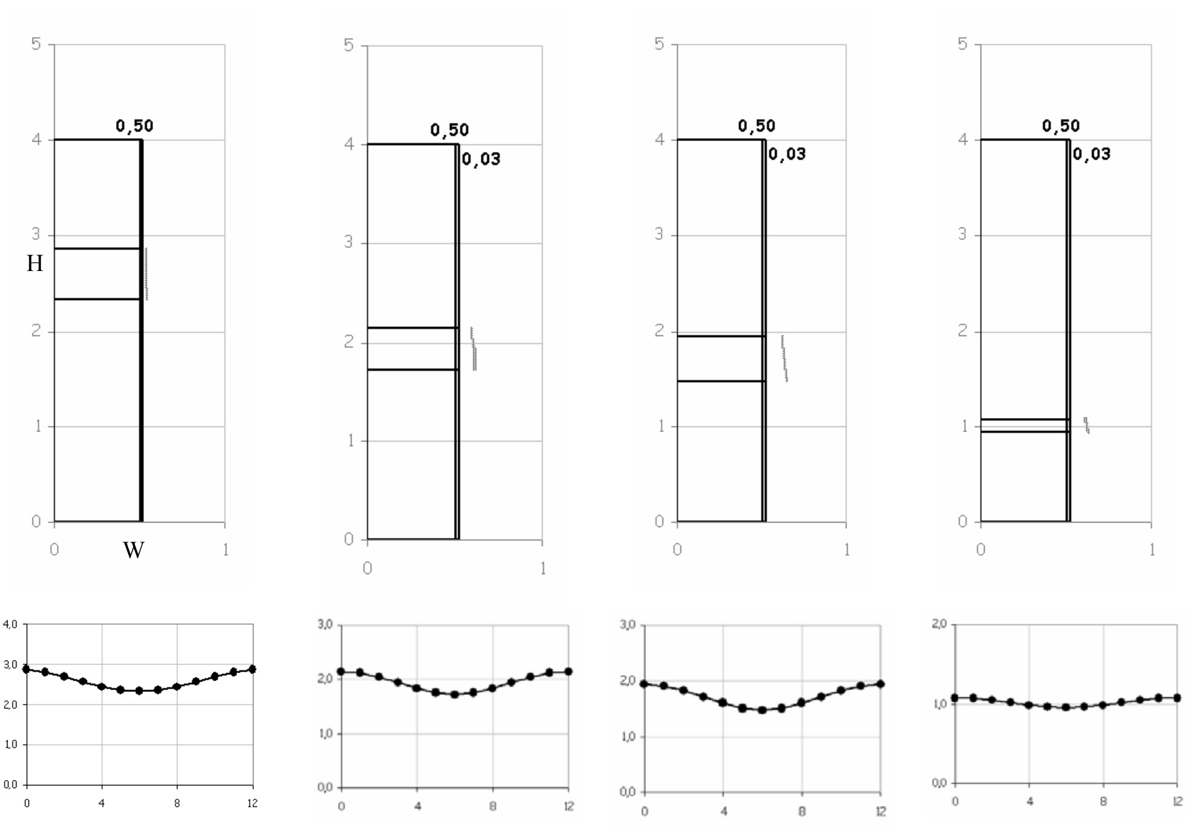
Figure 3. Meteorological data changes per month

At the following figures are shown the two masonries M1, M2, with the application of different plasters for the same environmental conditions. M1 is constructed with brick, while M2 with stone. The masonries have the same width, W equal to 50 cm.



(i) BRM **(ii) BRM-PMP** **(iii) BRM-PZN** **(iv) BRM-PEM**

Figure 4. M1 masonry minimum and maximum moisture height, the month that occurs and the changes with the application of different plaster



(i)SCY **(ii) SCY-PMP** **(iii) SCY-PZN** **(iv) SCY-PEM**

Figure 5. M2 masonry minimum and maximum moisture height, the month that occurs and the changes with the application of different plaster

Fig.4(i) and Fig.5(i) show that the moisture height(minimum and maximum values), symbolised with H, of the masonry constructed with brick is greater than the one constructed with stone, which is expected based on the fact that brick has larger t_d and t_c values (**C1**, Fig.1). The maximum values of moisture height, are observed for December and January, while the lowest values at July, for both masonries.

Regarding the application of plasters first of all it is evident, that the use of plaster contributes to the decreace of moisture height in all cases. The degree of height reduction depends on the plasters characteristics. In the Fig.4(ii-iv) and Fig.5(ii-iv) are shown the same type of masonries using a different type of plaster each time. For assesing the behaviour of plasters is useful to observe the charateristic moisture transport coefficients of each material (**C1**, Fig.1). As it is resulted from Fig.4(ii-iv) and Fig.5(ii-iv) for both masonries PEM plaster gives the lowest moisture height values. PEM is a plaster, as shown in **C1** chart of Fig.1, with the lowest value of capillary rise time coeffiecient among plasters. This might easily drive to the conclusion that the lowest capillary coefficient a plaster presents, the better for the masonry moisture balance is. But looking at Fig.4ii and Fig4iii where two others systems for each material are indicated, the BRM-PMP and BRM-PZN, it is evident that the use of PZN plaster leads to greater decrease of moisture height, although PZN presents greater capillary coefficient than plaster PMP. The same is observed for the systems SCY-PMP and SCY-PZN (Fig.5(ii) and Fig. 5(iii)). Moreover PZN compared to PMP presents greater drying time constant values, but lower than PEM.

As a consequence for the specific materials (BRM, SCY), the appropriate plaster that is the one with lower values of capillary rise time coefficient and with higher values of drying time constant.

6 CONCLUSIONS

The proposed simulator is a powerful tool in decision making concepts concerning the building deteriorating evolution and the selection of the appropriate protecting strategy, e.g., the plaster selection (material, size, replacing time). With the use of this tool, the assessment of the effectiveness of restoration materials before their real application can be made, contributing to the extension of masonries lifetime, but as well to the attenuation of cost and time waste of restoration works.

7 REFERENCES

Burch D.M., Chi J. 1997, 'MOIST: A PC program for predicting heat and moisture transfer in building envelopes', Release 3.0.

Durability guidelines for building wall envelopes, PWGSC, PRS, Technology & Environment, Technical Report, March 1997.

Geving S., Karagiozis A., Salonvaara M. 1997, 'Measurements and two-dimensional computer simulations of the hygrothermal performance of a wood frame wall', *Journal of Thermal Insulation and Building Envelopes*, v 20, pp. 301-319.

Karagiozis, A. N. 2003, 'Importance of moisture control in building performance', Canadian conference on building energy simulation, September 11th - 13th, Montreal, Canada.

Karoglou M., Moropoulou A., Giakoumaki A., Krokida M.K., 'Capillary rise kinetics of some building materials', *Journal of colloid and interface science*, (in press).

Karoglou M., Moropoulou A., Krokida M.K., Maroulis Z.B., 'Drying kinetics of some building materials', *Journal of drying technology*, (in press).

Theoulakis P., Moropoulou A. 1999, 'Salt crystal growth as weathering mechanism of porous stone on historic masonry', Journal of Porous Materials, v 6, pp. 345-358.

Maroulis, Z.B. & Saravakos, G.D. 2003, *Frood process design,*Dekker, New York.

Mendes N., Ridley I., Lamberts R, Philip P. C., Budag K. 1999, 'UMIDUS: A PC program for the prediction of heat and moisture transfer in porous buildings elements', http://www.pucpr.br/pesquisa/lst/.

Review of Hygrothermal Models for Building Envelope Retrofit Analysis, Canada Mortgage and Housing Corporation (CMHM), Technical Series 03-128, http://www.cmhc-schl.gc.ca/publications/en/rh-pr/tech/03-128-e.htm.

Straube, J. 2002, 'Moisture in Buildings', *Journal of ASHRAE*.

WUFI (0), Moisture transport in building materials-computer simulation with the WUFI model, http://www.hoki.ibp.fhg.de/wufi/wufi_frame_e.html.

Experiences of the Durability of Wooden Houses in Finland

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ABSTRACT

Finland is one of the leading timber construction countries in the world. Finnish houses have traditionally been wooden houses. The danger of fire has led to restrictions which have allowed a wooden frame only in one or two story high buildings. That's why wooden buildings in Finland have mainly been single-family houses. Since the 1990s Finland has strongly invested in timber construction and tried to increase the use of wood in all types of buildings. Also the Finnish fire code has been renewed and today it allows wooden strucures in multi-storey buildings.

The experiences of wooden houses have not been only positive in Finland. Serious moisture damage has been found in 82 % of single-family houses built after the 1950s. The worst damage has been leaking of low-pitched roofs, base floor damage as a result of building too close to the ground, damage of wet areas and damage resulting from leaks in the plumbing.

The large amount of damage indicates that timber structures are very susceptible to damage. However, the damage has resulted more from changes and experiments in the way of building than from the use of wood as a building material. Before timber construction can become common also in larger scale buildings it will be necessary to eliminate design faults and learn to build sound and lasting timber structures. When properly structurally protected, wood is a lasting and reliable material that creates beauty, warmth and pleasantness in its environment.

KEYWORDS

Timber structures, moisture damage, long-term durability, healthy buildings.

1. INTRODUCTION

Finland is one of the world's leading timber construction countries. This is a consequence of the abundance of Finland's wood resources. Finland is the most forested country in Europe. Finland is situated in the northern coniferous forest belt and its most common wood species are pine and spruce. 45 % of the forests are covered with pine and 37 % with spruce. Until recently, the growth of Finland's forests has been greater than removal, and the annual utilisation of wood can clearly be increased. Increasing the use of wood creates new jobs, boosts Finland's exports and improves the national economy. According to estimates it would be possible to even double the current export income from Finland's forest industry. That's why the Finnish government has invested significantly in the development and research of timber construction since the 1990s. The goal of this effort has been to increase the wood expressly in construction. [Heikkilä 2003].

Wood is an excellent natural building material with a long tradition of use. The long history of timber construction demonstrates that durable and long-term houses can be built from wood. However, wood also has its weak qualities. Thus there have also been doubts about increasing the use of wood. The strongest misgivings concern the combustibility and durability of timber structures. Along with fire, the other significant risk of timber construction is moisture. About 80 % of all damage in wooden houses is a result of moisture. As a consequence of long-term moisture, timber structures become mouldy, rot and spoil. There are plenty of examples of this, too, in the history of timber construction. This presentation concerns the durability of wooden houses expressly from the standpoint of moisture damage.

A precondition for the success and increase of timber construction is the ability to make durable and lasting structures from wood. When building from wood, one must recognise the biological and physical qualities of wood and the behaviour of timber structures under different conditions. Special attention must always be directed at protecting wood from moisture in construction.

The most important wood species in Finnish construction is pine. Spruce is the second one and it is used in the same way as pine. The durability of wood against biological damage is varying between wood species, individual trees and different parts of each tree trunk. Heartwood is normally more durable than surface wood. According to international classifications (EN350 1-2) pine and spruce are quite sensitive to biological damage. Wood begins to be damaged if its moisture stays above 20 % for a long time. In that case the relative humidity of the surrounding air is usually above 80 %. A relative humidity of 70 % can be considered the critical value with respect to mould in wood. When the relative humidity of air exceeds 90 % wood begins to rot. Another precondition for the moulding and rotting of wood is that the temperature is between 0 and +40 °C. Mould spores and rot fungus require oxygen and nutrients to function; normally there is a sufficient amount of these in wood as well as the surrounding air.

From wooden buildings and structures hundreds of years old, we can see that wood does indeed last for a long time if its structural protection is taken care of. The precondition for the durability of wood is that the moisture of the wood stays continuously under 20 %. Wood is a demanding building material that requires meticulous design, building and maintenance.

2 FINLAND'S WOODEN BUILDING STOCK

Wood has always been a natural building material in Finland. In old construction wood has been used in a very versatile manner because there were no other building materials available or they were too expensive. Until the beginning of the 1900s, wood was almost the only building material used in residential as well as public buildings, in load-bearing as well as supplementary building elements.

Up to the mid 1800s Finnish towns were also wooden house towns. As a result of the great town fires of the early 1800s, stone houses began to be favoured in the construction of towns. Gradually stone houses grew into multi-storey buildings and were larger than wooden houses. The way of building was controlled with fire safety regulations that permitted the use of wood only in low one- or two-storey buildings. As a result, the use of timber structures all across Finland during the 1900s was practically limited to single-family houses. [Heikkilä 2003]. New Finnish wooden houses are mostly one- or two-storey single-family houses. Wood has also been used to build row houses and small commercial or public buildings. When assessing the experiences of the durability of wooden houses, one can lean on the experiences of single-family houses. No comprehensive study of the damage of single-family houses has ever been conducted in Finland. Instead a large amount of empirical data has been collected, and based on that we can deduce the most common damages and their causes.

Before drawing conclusions regarding the durability of timber structures and faults in wooden houses, we must remember that wooden houses have been quite different in different times. Damage cannot be discussed without knowledge of this background. One must know the ways of building commonly practised in various times. Experiments have also been carried out in the history of timber construction; feedback has been received from these experiments and ways of building changed. This chain of trials and errors must also be known. Next is a description of the typical way of building in Finland over different time periods.

2.1 Log house tradition (-1940)

The traditional Finnish wooden house has been a log house, in which horizontal logs have simultaneously comprised the load-bearing structure of the walls, thermal insulation and, often, the interior and exterior cladding. Log houses contained a timber-framed floor that had a ventilated crawl space; for insulation of the floor, natural materials such as moss were used at first, later sawdust. The ceiling has been similar in structure. It must be remembered that originally there were no wet areas at all in these buildings. The sauna and related washing facilities in traditional construction have always been located in separate outbuildings. Log houses have been wood-heated; fireplaces that generated strong ventilation were built into the rooms. Efforts were also made to make log houses air proof. That is why people began to coat the interior with cardboard and wallpaper and cover the exterior with boards. These log houses have proven to be long-lived if their roof is kept in good condition. Damages have been found in the crawl space when ventilation has been insufficient or surface water has gone under the house or the ground has been exceptionally wet. This way of building prevailed in Finland until the beginning decades of the 1900s. [Heikkilä 2003], [Kääriäinen *et al.*1998].

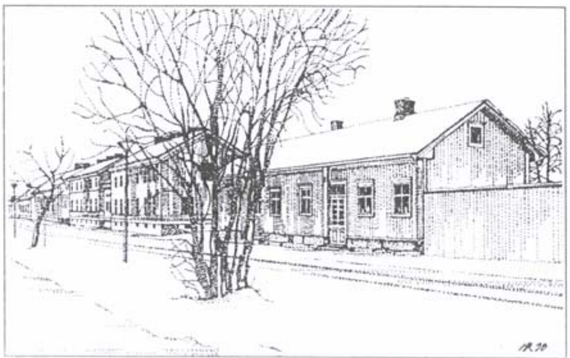


Figure 1. Log houses of wooden towns are one- or two-storey. Their roofs are steep, facades are boarded over, and they have a high base.

2.2 Period of reconstruction (1940s and 50s)

In the early 1900s American-style lightweight timber frame was imported to Finland; it fairly quickly replaced the log structure thanks to its affordability, quickness to build and better thermal insulation. Sawdust and plane chips were used as insulation in the frame structures and cardboard was used as a sealing material, whereupon the structures functioned in the same way as traditional log walls with respect to the physics of moisture. The height of popularity for this structure was during the reconstruction of the Second World War. The timber-framed "veteran homes" of the 1940s and -50s are one-and-a-half-storey and have a cellar. The houses are high and shaped like a cube, and have a steep roof. Owing to the cellar the wooden floor structure is ventilated and lies clearly above the ground. The sauna and washing facilities are located in the stone-structured cellar. The woodwork of these houses is in good condition if the roof has been kept in good condition. However, a substantial amount of moisture damage has occurred in the cellars of these houses due to the poor water-tightness of the cellar walls. [Heikkilä 2003], [Kääriäinen *et al.*1998].

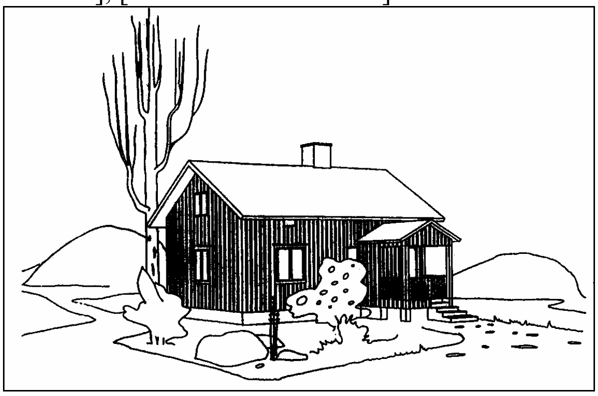


Figure 2. A house of the reconstruction period is one-and-a-half-storey and its roof is steep. The house always has a cellar.

2.3 Period of modernism and structural experiments (1960s and 70s)

At the onset of the 1960s the way of building wooden houses changed dramatically. When it was discovered that a low foundation could be laid without danger of frost damage, the practice of building cellars was abandoned and dwellings, including service spaces, began to be built in single storey. As a result, architectural ideals also changed and designers aspired to make houses low and to make them accentuate the horizontal direction. Floors became ground-supported concrete slabs and were pressed as close to the ground as possible. In wooden houses the result was that the bottom of the exterior walls extended underground, which has proven to be one of the worst mistakes in timber construction. This way of building has not even yet been completely abandoned even though its erroneousness was demonstrated as early as 25 years ago. Patterned on the same architectural goals, designers also wanted roofs to be as gently sloped as possible. In the beginning of the 1970s new wooden houses changed into entirely flat-roofed houses, and eaves, which protected the façade, went away at the same time. In a short period of time, flat roofs proved to be a fateful mistake in Finland's harsh climate.

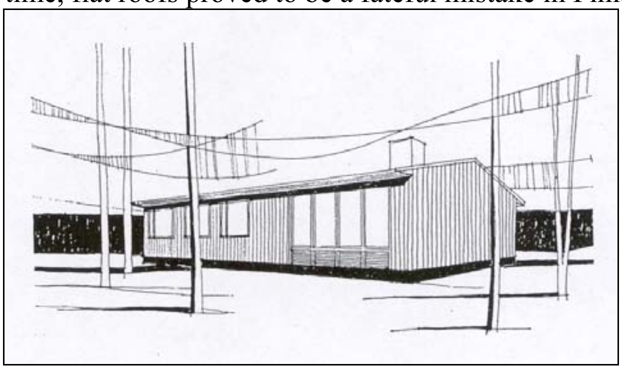


Figure 3. Wooden houses from the 1960s were built into one storey and cellars were no longer built. The houses were made low and the roofs gently sloped.

When cellars stopped being built, the sauna and washing facilities were moved inside the building. This has resulted in a significant increase in the moisture stress of dwellings and also the placement of piping, susceptible to leaks, into structures. This increase in the use of water took place without developing the structural solutions of wet areas or boosting the ventilation of houses. Another fateful mistake was embedding the pipework into structures. The hidden pipes could leak for a long time before the damage was noticed.

The operating principle of wooden structures with respect to the physics of moisture changed in the 1960s when industrial mineral wool products replaced the earlier wood-based insulating materials. With the use of mineral wool, which insulated heat well and absorbed moisture poorly, plastic sheeting was required in the wooden structures to ensure air- and vapour-tightness. The more impervious wall solutions and poor ventilation led to quality problems in the indoor air of single-family houses.

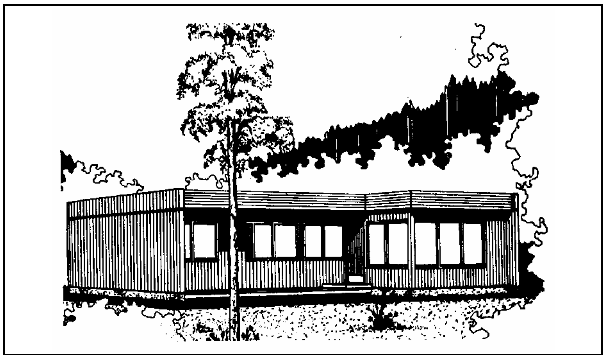


Figure 4. Wooden houses from the 1970s were made entirely flat-roofed.

The wooden houses of the 1960s and 70s have proven to be very problematic. Some of the solutions used at that time were abandoned quickly. Most houses of that period have received a new roof and flat roofs have not been built on single-family houses since then. Plumbing was also no longer embedded, and since then piping has been installed in casing pipe so that their condition could be monitored and they could be replaced without demolishing the surrounding structure. As a result of the evident indoor air problems, later houses have most often been equipped with mechanical ventilation. [Heikkilä 2003], [Kääriäinen *et al.*1998].

2.4 A step backwards (1980-)

In the timber construction of the 1980s, designers returned back in the direction of tradition and worked to avoid the problems caused by the excesses of the previous decades. Since the 1980s Finnish single-family houses have again had inclined ridge roofs. Efficient thermal insulation and improved air-tightness of the envelope are typical of new houses. The houses are, to a large extent, equipped with mechanical ventilation. Water pipes are situated in casing pipe so that leaks can be detected and pipes can be replaced without demolishing structures. Walls of stone materials are often used in wet areas. Some of the damage typical in wooden houses of the previous decades has thus been prevented. A problem even in new houses is building too close to the ground, which has continuously resulted in moisture damage in the base floor and the lower part of exterior walls. [Heikkilä 2003], [Kääriäinen *et al.*1998].

Wooden structures have maintained their dominance even in the newest Finnish single-family construction of the 21st century. In 2002, 87 % of new Finnish single-family houses, 80 % of row houses and 100 % of holiday houses were wooden buildings.

In the 1990s Finland's fire safety restrictions were renewed to be in accordance with pan-European principles. According to fire safety restrictions in force, wood can also be used to make 3-4 storey residential and employment buildings. Construction of wooden apartment buildings has been tested in Finland, but it appears that the wooden share of multi-story housing construction will remain fairly small despite the removal of fire technical barriers.

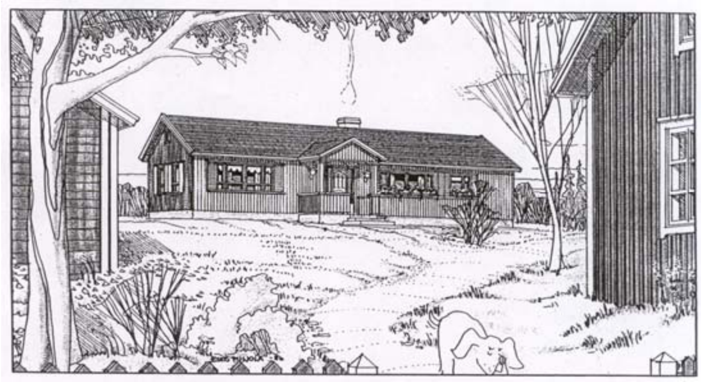


Figure 5. Lessons were learned from tradition in the construction of wooden houses in the 1980s. Once again houses have steep roofs.

3 CONDITION OF FINNISH WOODEN HOUSES AND TYPICAL DAMAGE

The moisture damage of Finnish wooden buildings can be evaluated based on sampling research conducted in 1995 on single-family houses. The amount of moisture damage in Finnish single-family houses is alarmingly great. This study shows that moisture damage exists in 82 % of single-family houses built after 1950. The fact that nearly 500,000 or 55 % of Finnish single-family houses are in need of prompt repair is an indication of the seriousness of moisture damage. [Partanen *et al.*1995].

The study indicates that there is moisture damage in wooden houses of all ages. However, most moisture damage clearly exists in houses from the 1960s and 70s, in which new types of materials and architectural solutions were used. Based on the study it appears that something has been learned from the mistakes, as the amount of moisture damage in houses built after the early 1980s has somewhat decreased. [Partanen *et al.*1995]. One can conclude from the study that the danger of moisture damage in wooden buildings is always great.

When studying the moisture damage in wooden houses of different ages, we can see that the types of damage are closely linked to each way of building. The biggest moisture technical problem of houses from the 1950s is the leaking of stone-structured cellar walls. Other moisture damage is related to the ageing of roofs and plumbing. The actual timber structures in these houses are in good condition. The reason for this is the location of timber structures high above ground level, steep roofs and protective eaves as well as the absence of actual wet areas inside the wooden frame. Damage in houses from the 1960s is plentiful and exists in all parts of the buildings. Most of the damage is in the timber-framed ceiling structures due to the leaking of gently sloping roofs. There is also a lot of damage in base floor structures. The damage develops from leaks from embedded plumbing and outdoor surface water. A large amount of moisture damage is also located in the floors and walls of wet areas, which have not been made waterproof. The worst moisture damage in houses of the 1970s is damage in ceilings resulting from leaks in the flat roofs. Another significant type of damage is wall and floor damage. Base floor damage is also caused by outdoor surface water. As late as the 1970s, it was customary to sink pipework into base floor structures, and plumbing damage is a significant problem group. In wooden houses from the 1980s, moisture damage in ceilings no longer exists because sufficiently steep roofs have been built on the houses. On the other hand, wall and base floor damage of wet areas continue to be common. Base floor damage caused by outdoor surface water also continues to occur. [Partanen *et al.*1995].

Moisture damages in houses built after the 1960s are connected with four areas:

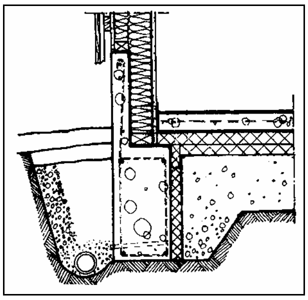
1) leaking of gently sloped roofs
2) base floor damage caused by building too close to the ground
3) wet areas
4) plumbing leaks.

More than 70 % of flat roofs are damaged. There is damage in the wet areas of nearly 42 % of all houses. Of all base floor damage, 50 % is caused by outdoor surface water. Plumbing damage appears to be proportional to the age of the pipework. [Partanen *et al.*1995]. Leaks in pipes are also common in new houses. Water leakage was found in 17,000 single-family houses in Finland in 1995 [Määttä *et al.*1997]. The total sum of the cost of repairing the moisture damages in single-family houses is estimated to be 600 million euro, or approximately 1,200 euro per house. [Partanen *et al.*1995].

The large amount of moisture damage suggests that timber structures would be particularly problematic. In this respect, however, one should be critical. Moisture damage is more related to the ways of building in use than the use of wood as the principal building material. The cellar walls of houses from the 1950s are stone-structured and their problems with water-tightness are not related to wood. The roof leaks of the houses from the 1960s and 70s are caused by too slight an inclination and poor waterproofing materials. The damage as such is not related to the use of wood as a building material. The experiences demonstrate that damage has occurred in the flat roofs of stone buildings in the same way. Neither can the damages in wet areas be considered only the fault of timber structures. The prime cause of the damage is inadequate waterproofing. Moisture damage has also occurred in stone-structured wet areas whose structures lack waterproofing. Plumbing leaks are caused by the ageing of piping materials and are thus are not only a problem of wooden buildings. Pipes also leak in stone houses. Base floor damage is a result of outdoor surface water and building too close to the ground. Damaged base floors have been concrete slabs. Damage has also naturally occurred in the timber structures that are in close contact with the concrete. Based on what has been presented, we can conclude that wood material itself has contributed little to the worst damage of wooden houses. This is confirmed by the fact that serious moisture damage has been found in 60 % of stone-structured apartment buildings built in Finland over a comparable time period.

4 WAYS TO PREVENT MOISTURE DAMAGE

During the past decade in Finland there has been continuous and extensive public discourse about the quality of construction. The background of this discussion is the large amount of moisture and mould damage in buildings and the resulting indoor air problems. In addition to wooden single-family houses, multi-storey residential buildings and public and commercial buildings have suffered serious moisture damage. On the other hand it must be noted that Finland is not alone with its construction quality and moisture problems because in international assessments the quality of Finnish construction is considered fairly decent. In the discussion on quality it has been stated that it appears that know-how in building physics will become an important key domain of study. Moisture damage can be prevented only through significant investments in research and development of building physics. [Heikkilä 2004]. This particularly applies to timber construction, which is nationally important to Finland.



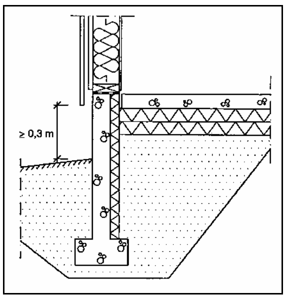


Figure 6. Left. A dangerous solution from the 1970s. The floor and the ground are on the same level. The bottom of the timber frame extends underground. Right. A safe joint of the floor and the exterior wall.

Moisture damage in wooden buildings has not gone away even though information about healthy solutions has been available already for a long time. The same mistakes are continuously repeated in the construction of single-family houses. 60-80 % of the risks of moisture damage in new wooden buildings fall on base floor structures and wet areas. Damages in roofs and plumbing have mostly been controlled. Base floor structures, the joint of the base floor, foundation and bottom of the exterior wall, and wet areas constitute the most demanding details for the moisture technical design of a wooden house. By working on the careful design, construction and maintenance of these structures it will be possible to ensure long-lasting and safe wood construction. [Heikkilä 2003].

A basic requirement of base floor structures is that the floor area is lifted sufficiently high from the surrounding ground level. An effective layer that cuts off capillary rising of water is needed under a ground-supported base floor. The moisture stress of the base floor and footings can be reduced by sufficiently inclining the ground away from the building and making a subsurface drain around the building. Thermal insulation in ground-supported base floors should be situated under the concrete slab. Wet areas should be built with stone structures where possible even in wooden buildings. In wet areas, not only the floors but also the walls need to be reliably waterproofed. With these quite simple solutions the majority of the moisture problems in timber-framed single-family houses can be prevented. [Heikkilä 2003].

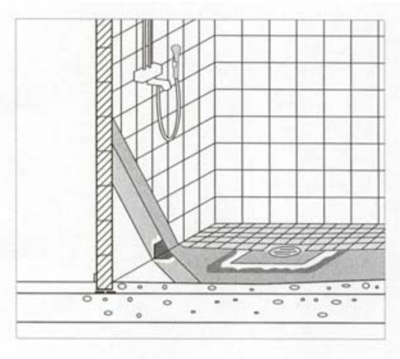


Figure 7. Wet areas should be made with stone structures and there must be reliable waterproofing in their floor and walls.

Moreover, when care is taken that only stone structures are used in possible cellars, piping is installed in casing pipe, a sufficiently inclined roof and façade-protecting eaves are made on buildings, builders are close to achieving a healthy and risk-free way of building with wood. [Heikkilä 2003]. After builders clamp down on structural mistakes, wood, the traditional Finnish building material, will be freed from the groundless suspicions aimed at it. When properly structurally protected, wood is a lasting and reliable material that creates beauty, warmth and pleasantness in its environment.

5 REFERENCES

Heikkilä, J., 2003, 'Learning from Defects of Timber Buildings in Finland', Proceedings 2nd International Symposium on Building Pathology, Durability and Rehabilitation, Lisboa, Portugal, November 6-8, 2003, pp. 351-359.

Heikkilä, J., 2004, *Hometalo ja miten se tehdään – syntyykö kosteusongelma jo suunnittelijan pöydällä? (How not to build a house – Are problems with dampness born on the drawing board?)*, The Finnish Architectural Review, No 2.

Kääriäinen, H., Rantamäki, J. & Tulla, K. 1998, *Puurakenteiden kosteustekninen toimivuus. Kokemustiedot (Moisture technical functionality of wooden structures. Experience data),* VTT, Espoo.

Määttä, J. & Kannisto, T. 1997, *Pientalojen talousvesiverkostojen vuotovahingot (Leak damages of household water networks in single-family houses)*, VTT, Espoo.

Partanen, P., Jääskeläinen, E., Nevalainen, A., Husman, T., Hyvärinen, A., Korhonen, L., Meklin, T., Miller, K., Forss, P., Saajo, J., Röning-Jokinen, I., Nousiainen, M., Tolvanen, R. & Henttinen, I. 1995, *Pientalojen kosteusvauriot - yleisyyden ja korjauskustannusten selvittäminen (Moisture damage in single-family houses – clarification of frequency and repair costs),* Kansanterveyslaitos, Kuopio.

Practical Approach towards harmonised Databases using the concept of hierarchical product category rules

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ABSTRACT

We propose to describe procedures and results achieved when applying the concept of hierarchical PCR documents, from a European to a regional tier, for harmonising databases in the construction sector.

The framework of LCA is an internationally accepted methodology for systematic and scientific assessment of a product's performance during all stages of its life. Life cycle data of construction products is useful for many different applications, such as product development, supply chain management, environmental labels and declarations or assessment of buildings.

LCA methodology has been translated into a good number of software tools rendering the method accessible to a large community in Europe and worldwide - also in the construction sector.

During the past decade of LCA studies a large amount of data has been collected and documented for specific products as well as for basic processes e.g. energy provision, production of plastics, metals, cement, etc. While processing raw data into LCA results has become an affordable task, measuring and collecting this data is still an expensive exercise. Harmonised requirements on data definition and quality will increase its applicability and acceptance of LCA. The lack of comparability of data from different origins and application contexts has already resulted in a lack of credibility of LCA based applications. This lack has different reasons: for comparable product systems a number of obviously necessary conventions like the description of system borders, of assumptions made or calculation rules applied, have not been harmonised. Furthermore most data is not yet documented with appropriate transparency. Harmonised requirements on meta-data would be helpful.

The German Ministry for Traffic and Construction today introduces sustainability aspects into its own construction activities. Among others it finances a project to facilitate networking of different national and international approaches at harmonised databases for construction products. Such a database is expected to be publicly available and should support a number of different applications as mentioned above.

One major task for a harmonised LCA databases is ensuring data consistency. On a regional, German level a number of networks, associations etc. are active in defining such rules for either groups of products in the construction sector or basic industrial processes across all sectors.

Using the concept of Product category specific rules (PCR), which is recommended in ISO standards dealing with Type III environmental product declarations (EPD), a hierarchical sequence of documents has been developed breaking down those general requirements from the international level to specific requirements for a specific product category. These documents describe how to measure and collect data in compliance with specific requirements for a specific product category. Transparency via systematic documentation of product system description, calculation rules as well as data sources etc. allows for comparability with respect to other data sets derived under the same set of conventions. Independent control of such requirements including data quality enhances credibility. Examples from the construction sector and – as an important basic process - the energy sector will be presented. The typical requirements of the PCR concept have found considerable acceptance in the LCA community already. However, a lean and reliable procedure to arrive at a consensus for concrete requirements with sufficient acceptance by industrial competitors who will have to supply the data and interested parties like government, environmental NGOs or consumers' organisations, who will have to believe it, is still under discussion. The procedure we will describe and show examples for is applicable in a European context. The potential of arriving at international conventions beyond Europe using the PCR concept and the platform of an third party network for communicating LCA data, GEDNet (Global Environmental Declaration Network) is also described.

KEYWORDS : Environmental product declarations; LCA; Harmonisation.

1 INTRODUCTION

LCA is an effective instrument for modelling the environmental impact of products and is thus useful to build understanding and knowledge in the pursuit of the politically and societally important issue of "sustainable development." "Life Cycle Thinking" is the central theme of European integrated product policy. The integration of environmental aspects into product standardization and into public procurement is an example of the use of LCA central for the building sector. Environmental product declarations (EPD) are considered a reliable way of communicating LCA results for such purposes.

EPD are based on LCA and additional technical, ecological and toxicity data. They provide information about the environmental aspects of a product without evaluating it. From the data sets, environmentally relevant properties of a building can be determined. Under certain methodological requirements, the declarations can be used for an assessment of building products in the context of a building. The declarations are also intended to build a database for the assessment of buildings.

The practical application of such declarations consists in the comparison of various products based on the declared data. The comparability of EPD requires the comparability of the LCA on which they are based, as well as the comparability of other information declared in the EPD. This comparability is achieved through identical requirements on the conditions under which the LCA and other information are produced. These requirements are contained in a document of product category rules (PCR). The role, contents, and testability of the PCR document are described, with emphasis on the requirements on the quality of data sets. Credibility is an essential precondition for a successful application of EPD. Credibility is promoted by testable requirements on data quality. Some methodological aspects of data quality and of LCA data application for EPD are discussed.

2 GENERATION OF ENVIRONMENTAL PRODUCT DECLARATIONS

The standard ISO DIS 14025 describes the procedures to be followed when generating an EPD. Program operators provide a framework, which helps companies to develop an EPD. Guidance is given for developing PCR with participation of interested parties, for providing credible control procedures for PCR and for data calculated from it Programs in the construction branch are privately operated. While programs seek to be self-supporting, industry and governments supported most program operators to get started. Generating an EPD however, relies on the initiative of the producer.

An EPD consists of a set of indicators, derived from a LCA of the declared product and any additional information necessary to describe the product’s environmental performance sufficiently:
“The overall goal of environmental labels and declarations is, through communication of verifiable and accurate information, that is not misleading, on environmental aspects of products and services, to encourage the demand for and supply of those products and services that cause less stress on the environment, thereby stimulating the potential for market-driven continuous environmental improvement.” (Objectives for all environmental or declarations ISO 14020, Environmental labels and declarations – General principles)
To be able to encourage demand and supply of products that cause less stress on the environment than others, EPD must enable the users to make a comparison with the help of accurate and comparable data about relevant environmental issues. To implement this ambitious requirement, an EPD program provides a company wishing to generate an EPD with a set of rules. The rules contain mandatory requirements for collecting the data and for processing it into a declaration. After the data is collected, the LC-indicators can be calculated – usually by applying a LCA-software. The data then is checked, thereby ensuring it was collected and processed according to the PCR and the description of the product’s environmental performance is plausible.
Next to the ISO standards, these procedures are explained from a practical viewpoint in the Handbook of GEDNet, the Global Environmental Declaration network for Type III [GEDNet 2002]

The figure below shows how to get from an application to an EPD, exemplified by the German EPD program for construction products, which is operated by a federation of producers, AUB (www.bau-umwelt.de).

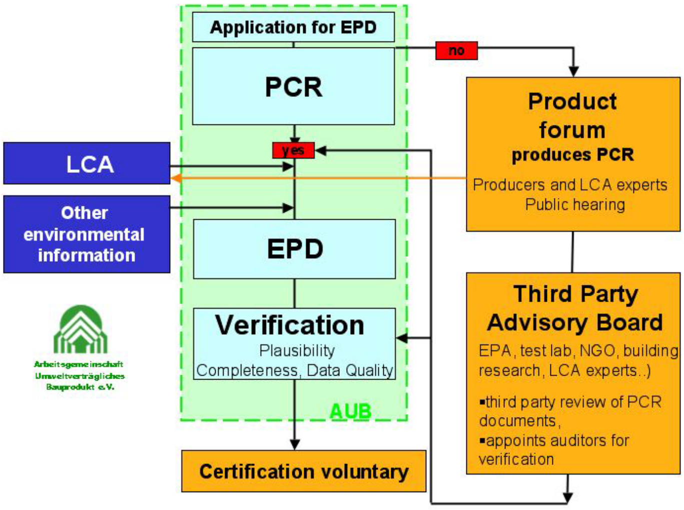

Figure 1. Example of program procedures to generate an EPD from application, taken from the AUB program Germany.

When preparing an EPD the first step is to allocate the product to a product category. A product category can be quite large, for instance the product category "construction products" or it can be more narrowly defined, for instance the product category "construction metals". For an LCA, a product category is defined as a group of products described by the same functional unit. However, EPD in the construction sector often are generated without considering the use phase – since their future application is multifunctional. In this case a product category is better defined as a group of products described by the same modules of a product system. By the same logic, the unit to which the LC-indicators of such "modular" EPD are normalised is not called a "functional unit", but rather a "declared unit". Product categories in a program should be carefully defined. Transparency in the allocation of products to a certain product category is key. In the end, only such EPD will be fully comparable that describe products of the same product category and only when the complete Life Cycle is considered or omitted stages are identical or not relevant.
When a product is allocated to a product category, it is allocated to a set of rules governing the LCA study for that product category. If no product category exists, it has to be defined and the respective PCR developed. The rules by which a product system is described and LC- indicators are calculated are to some extent conventions. They include choices favourable for one product and less favourable for another. Therefore, it is necessary to participate interested parties and to check completeness, relevance and consistency of the PCR by an independent third party. PCR have a number of functions. They ensure for an EPD:

- Credibility, by the third party independent review as well as the participation of interested parties during the development of the rules.
- Transparency, by publishing the PCR document.
- Consistency, by defining the product system, calculation rules, test condition and data quality according to a common methodology.
- Comparability, by using the same definition of the product system and the same rules for calculation, test condition and data quality.

3 PCR

3.1 Contents and structure of a PCR document

An example for the structure of a PCR document can be taken from the German AUB program for construction products. The PCR document for metal products is structured in the following way:

Declaration contents additional to LCA	0) General description of product	Definition of product category and product	
		Application of product, standards, quality	
		Characterisation of the product in delivery condition	
	1) Materials (product)	Composition / configuration of product	
		Hazardous substances	
		Extraction, Origin, Availability of raw materials	
	2) Production	General description of production processes	
		Packaging	
		Occupational health, safety and environment	
	3) Fitting, installation into the building	Recommendations for the construction phase	
		Occupational health, safety and environment	
		Waste at the construction site	
	4) Use phase of the pro-duct when integrated into the building	External and other special influences	
		Aspects of health, safety and environment	
		Reference Service Life, Durability	
	5) end of life of product	General description of recycling and/or deposition processes	
Declaration contents based on LCA	6) LCA	LCA requirements,	Declared unit, system boundaries, LCA stages, allocation rules, calculation etc.
		Choice of LCA Inventory Indicators	- renewable, non renewable raw materials; - renewable, non renewable energy sources; - Land use - Water use
		Choice of LCA Impact Indicators	- GWP - ODP - Acidification - Eutrophication - POPC
	Annex 1 Data collection	Type of data	Documentation format and units

Table 1: Description of structure and contents of a PCR document for metal products in the German AUB program for construction products.

3.2 Data quality

Data quality has been an issue for LCA practitioners for many years. So far no widely accepted method for quantitative assessment of data quality is on the market. However, a number or research projects deal with the problem, which may propel solutions in the near future [Carlson, R.2003 and Schuurmans A. 2003]. Additionally the ISO TR 14048 giving guidance for documenting LCA data could be the basis for a common format for LCA data. A new work item proposal to further develop the technical report is in work.

LCA data of a certain quality should be characterised with respect to being reproducible, consistent, complete and accurate, accuracy including being representative from a local, temporal and technological perspective. Assessing data quality will therefore need time and resources, which is seen as a drawback to the general ambition of Type III programs to be quick, flexible and low cost. To make publicly available LCA data on the market useful for EPD, a filter is needed for the appropriate level of data quality for an EPD. The UNEP Life Cycle Initiative pursues such a project. It plans to set up an Internet portal where such a filter of requirements is applied to publicly available data.

If assessing data quality is not (yet) possible, a step forward could be to define a set of databases to be used for EPD. Such a step was taken when European providers of electric energy, Vattenfall, EdF, and British Energy developed a PCR document for the product: 1 kWh of electricity or combined heat and electricity.

Material	Database	Published
Steel	IISI (International Iron and Steel Institute)	1998
Copper	ICA (International Copper Association)	1998
Copper semi products	ICA (International Copper Association) + IME (Institut für Metallhüttenwesen und Elektrometallurgie, Aachen)	1998 1995
Electricity	ETH (Eidgenössische Technische Hochschule) Data combined with IEA (International Energy Agency) statistics 1998	1996
Aluminium	EAA (European Aluminium Association)	2000
Plastics	APME (Association of Plastics Manufacturers in Europe)	1993-1998
Chemicals	APME (Association of Plastics Manufacturers in Europe)	1993-1998
Electronic components	EIME (Environmental Information and Management Explorer) EcoBilan	1998-2000
Transports	NTM or regional alternatives[1]	
Waste management	ecoinvent data v.1.0 / ETH Data on waste management	2003 / 1996

Table 2: Example for selection of databases in a PCR document, taken from the PCR for Electricity and District Heat Generation of the Swedish Environmental Management Council, 2004.

This document was developed on a European level using the Global Environmental (Product) Declaration Network (GEDNet) as a platform. An independent 3rd party verified the PCR in 2004 within the Swedish EPD-System. It is documented by the Swedish Environmental Management Council under www.environdec.com

Programs in the construction sector usually use one specific database, supplemented when necessary by external data for background processes. The German EPD program for construction products for example requires the database from GaBi, or equivalent data. This procedure at least ensures consistency of the data within a program. For comparability across programs, either the same specified requirements on data have to be followed, or the same data sources must be used.

3.3 Procedure for generating a PCR document

Developing the PCR document is a process of reaching a very detailed consensus for different categories of issues. The process of reaching this consensus can be split into a sequence of different levels. The first level includes the general selection of methodology, a description how the PCR should be developed and how their validity should be controlled. The second level is limited to a certain branch and covers a wide product category, e.g. construction products. At this level the items

1 Network for Transportation and Environment, a non-profit association working for consensus regarding environmental issues in combination with transportation (http://www.ntm.a.se/eng-index.asp).

to be considered at minimum when describing the product system of the respective product category are identified. The third level describes these items in detail. At this level it may be necessary to subdivide the wide product category into more specific product categories, if specific product characteristics require special treatment. For instance, when allocating recycling procedures for different materials like paper, plastics or metals different rules are favoured by the respective manufacturers. They take into account the variations in service life, recycling rates in the market place, energy content etc of the products, in order to describe the respective product system in an optimal way. Common rules should be defined as long as they produce an appropriate description of the product system. If this not the case, sub-categories will have to be defined. The differences between the sub-categories must be carefully analysed and given enough transparency, to avoid misleading the user of the declaration.

In the construction sector this procedure has already been realised to some extent. The first level describing methodology and procedures is realised by the ISO DIS 14025 standard, "Environmental labels and declarations – Type III environmental declarations – principles and procedures". The second level, of identifying typical items to be considered in the construction sector is realised by the ISO CD 21930 standard, Building construction – sustainability in building construction – Environmental declarations of building products. The third level will be realised by the upcoming CEN standards for sustainable construction. In this series of standards, one will have the function of a PCR document for construction products in Europe. In this document, the intention is to detail common rules for all construction products as far as possible, to achieve transparency and comparability where possible. The overall goal is to use these data sets for a consistent environmental assessment of buildings.

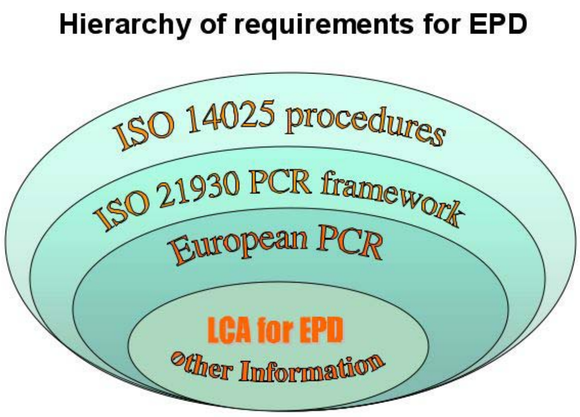

Fig 2: Hierarchy of requirements for the generation of an EPD in the construction sector, from general methodology to the actual LCA of an EPD.

Other industrial branches have chosen a platform different to CEN, to harmonise their database for EPD. The energy sector e.g. used GEDNet, an international association of EPD-program operators and other parties interested in EPD development. GEDNet, which also has Liaison status at ISO TC 207/SC3 (developing ISO 14025) is very well positioned to supply this platform for harmonisation of databases on an European and international scale.

4 ACKNOWLEDGMENT

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5 REFERENCES

GEDNet 2002, International Guide to Environmental product declarations, compiled and edited by Bill Hillier, Lars Jonsson and Sven-Olof Ryding.. © GEDnet 2002

Carlson, R. 2003, LCA Datareview., Contribution to the International workshop on LCI quality, FZK Karlsruhe

Bodlund, B., Settevall, C. Frischknecht, R. Schmincke, E.: Product specific requirements (PSR) for preparing an environmental product declaration (EPD) for Electricity and District Heat Generation (04/2004) p.11 www.environdec.com

Schuurmans, A. 2003,.Contribution to the International workshop on LCI quality, INTRON, Stichting MRPI Forschungszentrum Karlsruhe, Germany, Ed. Chr. Bauer.

Survival functions of buildings and building elements

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ABSTRACT

Buildings and their elements are like all technical systems subject to aging. There are numerous data on the life expectancy of building elements in literature. They vary considerably and both their origin and exact definition are generally lacking. Many of the date represent in fact "personal empirical values" of specialists. There are very little data on repair frequencies and depth.
A first empirical investigation to the life cycle behavior of building elements was performed by ETHZ in Switzerland in 1994. The data were collected on a relatively homogenous and well known housing stock over a long period. The differences in lifetime show clearly that the age of individual building elements is only one of many factors that lead to repair or replacement. In a precedent study, the history of selected buildings was reconstructed and the real repair and replacement operations were established. On this basis survival functions per element were determined. A similar method was applied by the authors to public buildings and showed analogue results. Public buildings are less subject to fashion and short term financial return objectives. Their aging history is determined by regular maintenance, periodic refurbishment (packages) and changes due to building regulations. The use of simple life times of buildings and elements, on the basis of literature values, does not give useful results where as the use of survival functions gives good approximations for the modeling (and probably the prediction) of maintenance and refurbishment operations. The combination of traditional building quality, regular maintenance and repair and efficient refurbishment leads to long life times both of building elements and buildings.

KEYWORDS

Life cycle, building stock, durability, maintenance, service life

1 THEORY

The aging process and the life-expectancy of building elements depends on numerous factors, as for example the composition and construction of the element, the intensity of use, the frequency of inspection and maintenance, the quality of workmanship and the climatic exposition of the element. To ensure the usability of an element it must be periodically maintained and/or repaired allowing to extent its lifetime. After a certain time most elements have to be replaced, despite maintenance and repair: they have reached the end of their lifetime.
The term of "life time" is somewhat ambiguous and has to be defined. The common use designates the "technical lifespan" or “service life”, i.e. the time during which an element performs its function. Alternatively "life time" can also designate the period of presence of the element in the building. This period can be either shorter or longer than the "service life".

1.1 Aging of building elements

For planning and managing buildings knowledge about the aging process of elements is necessary. The simple adoption of literature values, without evaluating the various factors of influence, leads to arbitrary results.

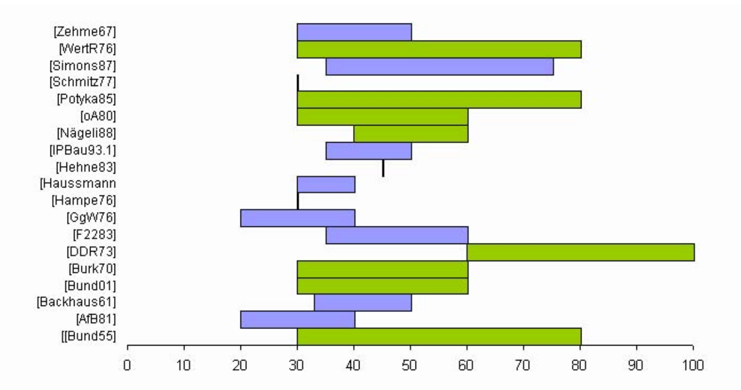


Figure 1. Data concerning the technical service life of a building element according to different sources [Impulse 1994].

A more analytical method to estimate the service life of an element is the factor method described in [ISO 15686-1]. This method is based on reference lifespan and a series of modifying factors that relate to the specific conditions of the considered case. The service life is a function of seven parameters (A to F) depending on inherent quality, the environment and operation conditions.

Agents	*Factor*	*Relevant conditions and examples*	
Agent related to the inherent quality characteristics	A	Quality of elements	Manufacture, storage, transport, materials, protective coasting
	B	Design level	Incorporation, protection
	C	Work execution level	Site management, workmanship, climatic conditions during work
Environment	D	Indoor environment	Aggressiveness of environment, ventilation, condensation
	E	Outdoor environment	Building height, microenvironment conditions, traffic emissions, weather
Operating conditions	F	In-use conditions	Mechanical impact, category of users,
	G	Maintenance level	Quality and frequency of maintenance, accessibility for maintenance

Table 1. Factors for degradation of materials and elements according to ISO 15686-1

The combination of the seven factors can lead to very different results. The ISO refers to national and international standards that do not exist yet. Besides the reference lifespan are not fixed. Only if guidelines – e.g. test conditions - are given, the application of ISO 15686 will be possible.

The analysis of a large number of existing buildings and elements, gives more reliable results, at least for similar buildings and conditions. In [Impuls 1994] survival functions have been combined with degradation levels for twelve elements. The use value of the element is considered equal to 1 at the beginning. The different indexes a,b,c,d correspond to clearly defined states of degradation. Analyzing a large number of cases three characteristic aging curves have been established. They associate duration in years with a certain state of degradation. The first curve to the left gives the degradation in heavy exterior condition and bad execution and maintenance. The curves to the right on the contrary give mild exterior conditions and high quality of execution and maintenance. The middle curve is an average. By knowing the state of degradation and the exposition the probable residual lifetime can be estimated. The dispersion of the curves roughly corresponds to the dispersion of the value in literature.

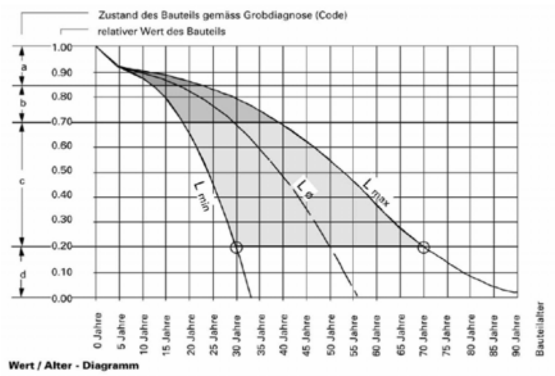


Figure 2. Survival functions for an element according to [Impuls 1994]

Apart from the material aging processes, immaterial factors can cause the early replacement of an element. Triggers are social, fashion, technical or legal developments. Furthermore elements which are not yet at the end of their service life can be replaced as part of general refurbishment actions e.g. when scaffolding is available.

1.2 Life time of buildings

Most simulations of existing building stocks are based on renewal cycles either of elements or of packages of elements. The renewal by element does not correspond to the observed reality, for practical reasons (tenants, scaffolding, sanitary system constraints etc.) refurbishment is almost always done on the building level. A first group of elements (typically lamps, paint) is repaired or replaced in the context of current maintenance; a second group (Typically burners, coatings) is replaced in partial renovation (every 20-30 year) and a third group (typically roofs) in complete renovation (every 30-50 years). The duration of the intervals depends on the quality of the buildings, both durability and adaptability. Partial renovations are often triggered by the condition of the vertical sanitary system, which cannot be replaced independently apartment by apartment. The trigger of complete renovation is often a change of functions.

2 AN EMPIRICAL INVESTIGATION OF BUILDING HISTORIES

In an empirical investigation the maintenance, repair and replacement operations of several buildings over a longer period have been reconstructed from the building owners records. As the data were quite detailed going down to element information, it was possible to proceed to a survival analysis of building elements. The external triggers could be restituted as well and it was possible to evaluate the maintenance strategies as such.
The analyzed objects were public buildings. This is why they were no changes in use and no sale during a long period. Short-term triggers like the income, under-occupation etc. did not appear.
As the same authority managed the buildings over long period information about costs, cleaning etc. were available. But even under these very positive circumstances it was long and difficult to extract the information from the building archives and the maintenance records. Some data recorded on electronic devices (recent data) was already lost where as the older data was well protected against loss. Finally three buildings of the same use (tribunals) but of different ages were selected.
While for the last five years the data situation is very good - work proof lies and bills exist - the measures of the further past could only reconstructed on the basis of archived correspondence. Partially and entire renovation measures are also well documented, because an approval of an appropriation committee was necessary.

2.1 Maintenance strategies

In general the maintenance strategy for public buildings aims at long-term value conservation. The refurbishment decision is based on a thorough examination of the building. Concerning the current maintenance there were no fixed inspection cycles but the surveyor visited the building frequently and he knew the building condition very well.
Due to the financial situation of the state administration at present only urgent measures are accomplished. There are no funds for preventive measures anymore. At the same time the Surveyor's Offices are eager to use the buildings as long as possible. For that reason, elements are still well maintained and adapted from time to time to new standards. Repair is still preferred to replacement when possible.

2.2. Survival analysis of building elements

Repair and replacement of building elements were examined by a survival analysis. Due to the very small number of buildings, the selection was limited to current elements.

The examined buildings had a particularly long lifetime. The reasons can be the regular maintenance and the requirement to keep building elements as long as possible. Furthermore decisions due to fashions were not common in public buildings.

By analyzing the replacements in detail the importance of exterior causes was noticed. Thus at Offenburg the windows were replaced on the occasion of an energetic improvement and in all buildings the internal doors were exchanged because of new regulations concerning fire and noise control.

In general the replacement was part of a refurbishment package on the occasion of a period refurbisment. In addition, some replacements were due to acute damages.

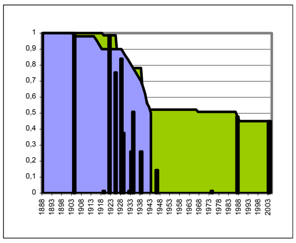
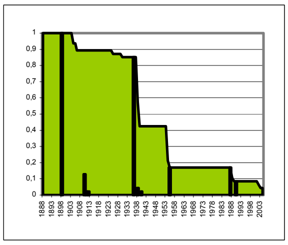
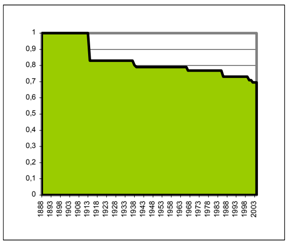
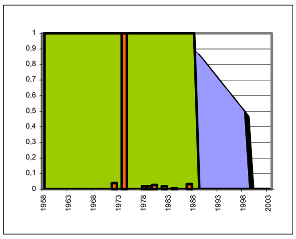
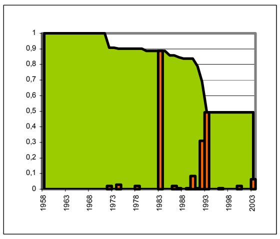
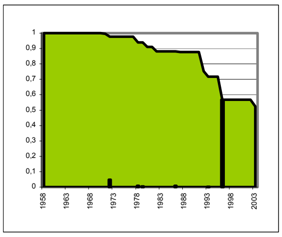
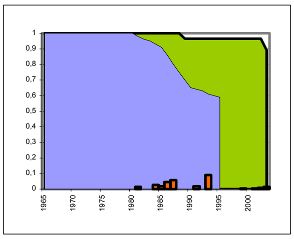
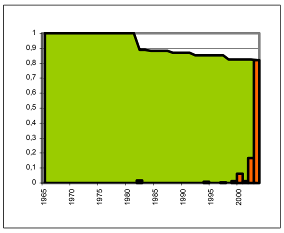
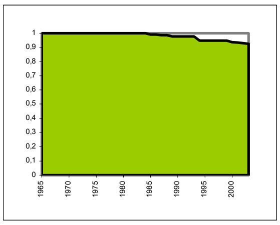
Building	*Windows*	*Internal doors*	*Floor coverings*	
Gengenbach				Original construction units Survival function according to [1] Repairs
Offenburg				
Freiburg				
Average technical lifespan (Literature)	40-60 Years	40-100 Years	Stone: 50 - 80 Years PVC/Linoleum: about 20 Years Wood: 30 - more than 100 Years	

Table 2. Survival functions of selected elements

2.3 Life cycle of buildings

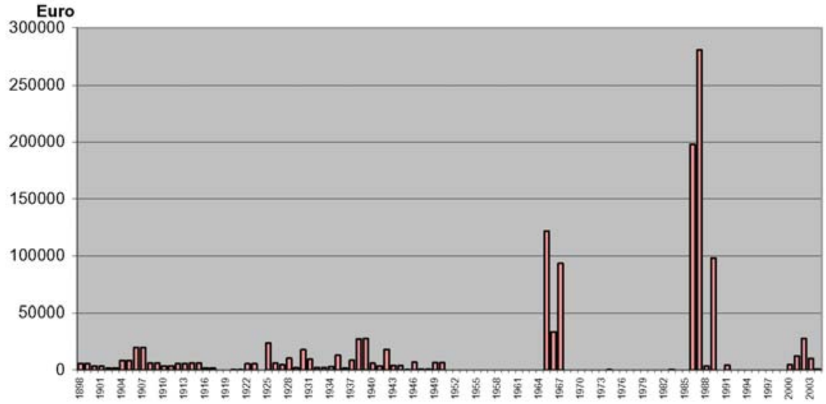
When evaluating the diagrams, missing data has to be taken into consideration. Costs are not known for all operations and there are gap in older data.

Building	*Date of construction*	*1st renewal*	*2nd.renewal*
Amtsgericht Gengenbach	1888	1946[1], 58 years	1985/86, 86 years
Justizgebäude Offenburg	1955-57	1991/99 about 35 years	
Landgericht Freiburg	1962-65	Period. Between 1985-2002; 20-38 years	

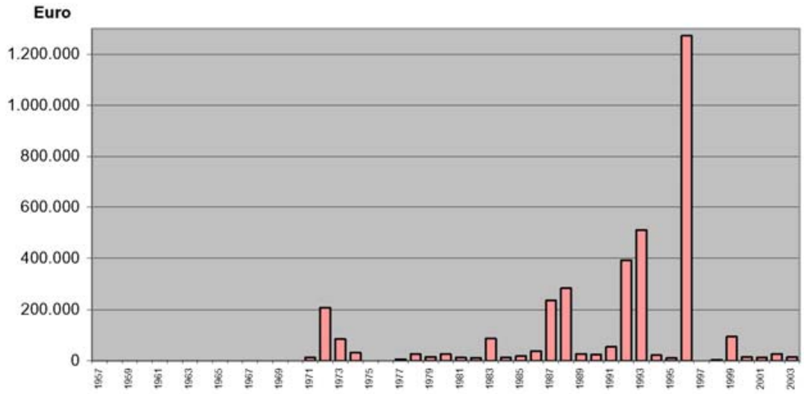
Table 3. Renovation cycles of the examined buildings

[1] no costs available

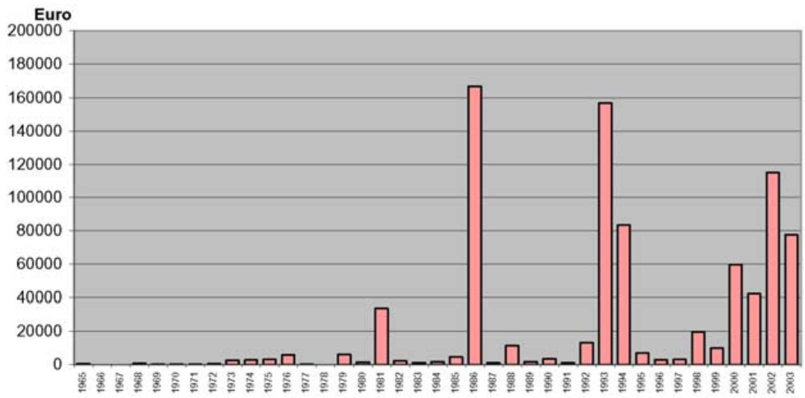
A first analysis concerns renewal cycles. The first step was to examine the time periods of entire renovations, why they were done and which elements had been affected. The course of the inflation-neutral maintenance cost shows well the cycles.



Amtsgericht Gengenbach



Justizgebäude Offenburg



Landgericht Freiburg

Figure 3. Inflation neutral maintenance costs

Usually models of the aging of building stocks are based on the replacement of building elements .By analyzing the renewal packages, the importance of receiving work was identified. So Simulations have to take into consideration this maintenance work.

In addition to the building construction reasons, which result from the general building condition, new building requirements have a considerable influence on refurbishment. The following reasons have identified per period:

20's	modernizations of the sanitary facilities caused by the introduction of water closets
40's	replacement of stoves by heating systems
80's	energy saving measures
90’s	introduction of Information technologies

3 CONCLUSIONS

The differences in lifetime show clearly that the age of individual building elements is only one of many factors that lead to repair or replacement. In a precedent study, the history of selected buildings was reconstructed and the real repair and replacement operations were established. On this basis survival functions per element were determined. A similar method was applied by the authors to public buildings and showed analogue results. Public buildings are less subject to fashion and short term financial return objectives. Their aging history is determined by regular maintenance, periodic refurbishment (packages) and changes due to building regulations. The use of simple life times of buildings and elements, on the basis of literature values, does not give useful results where as the use of survival functions gives good approximations for the modeling (and probably the prediction) of maintenance and refurbishment operations.
The combination of traditional building quality, regular maintenance and repair and efficient refurbishment leads to long life times both of building elements and buildings.

ACKNOWLEDGMENTS

The Surveyor’s Offices at Freiburg and Offenburg has put at our disposition the achieved documents and the staff has been very helpful in answering in detail all questions about maintenance strategies.

REFERENCES

[BBR 2003] Bundesamt für Bauwesen und Raumordnung, 2003, *Erneuerung älterer Wohnungsbestände in Stufen,* Selbstverlag des Bundesamtes für Bauwesen und Raumordnung, Bonn

[Kohler et al 1999] Kohler, N.; Hassler, U.; Paschen, H. (Hrsg.): *Stoffströme und Kosten im Bereich Bauen und Wohnen.* Studie im Auftrag der Enquete Kommission zum Schutz von Mensch und Umwelt des deutschen Bundestages. Berlin 1999

[Impuls 1994] Impulsprogramm Bau (IPBau) - Erhaltung und Erneuerung 1994, *Alterungsverhalten von Bauteilen und Unterhaltskosten: Grundlagen für den Unterhalt und die Erneuerung von Wohnbauten*. Author of the study : Prof. P. Meyer at al ETHZ. Bundesamt fuer Konjunkturfragen, Bern

[ISO 2000] ISO 15686-1:2000, *Buildings and Constructed Assets - Service Life Planing* - Part 1: General Principles

[Krings 2000] Krings, E. 2000, *Stufenlösungen bei der Altbaumodernisierung*, RKW-Verlag, Eschborn

Assessment of the overall degradation level of an element, based on field data

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ABSTRACT

Service life estimation methods usually require an in-depth understanding of the degradation mechanisms that lead to loss of performance of the considered building elements over time. Such information can generally be provided through accelerated ageing laboratory tests, in a scientifically controlled environment. However, this approach has often been rightly questioned by many researchers in as much as it fails to consider the full complexity of the environmental context.

As opposed to this approach, field data recollection techniques are often pointed out as a sound method to assess the degradation level of a building element, in real life service conditions. Such methods require a more or less extensive diagnosis, generally starting off with the visual identification of defects and their classification according to different levels of range and severity. In the end, a qualitative overview of the degradation proccess can be reached that may explain the causes of the registered pathologies and its mechanisms until the moment of inspection.

This paper reviews and discusses some of the available methods to assess the overall degradation level of a building element, and focuses on the convertion of qualitative data (the rating of degradation according to different levels) to quantitative numerical data that can be incorporated in engineering service life estimation methods. In this research (still on an experimental stage), a survey of the degradation level of 150 different mortar rendered façades was implemented. For each sample, different degradation levels in six independent areas of the façade have been considered and later converted into a single numeric value: the overall degradation level. Although further research needs to be carried out, different degradation curves can already be established and related to the environmental exposure conditions of each of the three different sites surveyed. This data may soon be incorporated in service life prediction methods.

Future developments include the determination of hierarchical relations between failure modes. Risk analysis will also have to be considered in order to better deal with entropy phenomena between deterioration mechanisms as well as with the uncertainty related to the workmanship and maintenance standards.

KEYWORDS

Degradation Level, Survey, Diagnosis, Defect Characterisation, Rendered Façade.

1 INTRODUCTION

All buildings and constructions experience a deterioration process from the moment they are built. In this process, they tend to loose their performance as they increasingly fail to fulfill users' needs, requirements and expectations. Loss of performance can occur due to functional obsolescence, unfavourable cost balance, physical degradation or a combination of these. Service life prediction has thus become a scientific field of its own, developing methodologies that allow effective rational maintenance management of buildings [Chew *et al.* 2003], especially as far as physical durability and life cycle costing is concerned.

Numerous factors contribute to the deterioration of buildings and their components. The factor method proposed by the Architectural Institute of Japan [1993], later adopted by ISO 15686 [ISO 2001], considers the inherent characteristics of each building element (material characteristics, design quality, construction quality and maintenance level) and environmental conditions that may act as agents of degradation. Accelarated ageing tests in a scientifically controlled environment and field data survey techniques to assess the degradation level of a building element in service conditions are often pointed out as two complementary approaches to estimate the service life of a given element [Hovde 2004]. The latter requires an overall accessment of the element condition, based on a diagnosis of its defects and their classification according to different levels of range and severity.

2 OVERALL DEGRADATION LEVEL

From the point of view of physical degradation, as buildings age, more failures and defects are expected to be found. Accordingly, if a given building element shows evidence of extensive deterioration, there is a strong likelihood that it may have a longer service period than that of an identical element in the same environmental conditions, but with fewer signs of deterioration.

Such degradation process can be depicted by a degradation function in which loss of performance is the dependent variable and time the independent variable. The rate and nature of degradation depend on the degradation mechanisms for the considered element and on the inherent characteristics of the latter. Degradation functions can be linear (for continual degradation mechanisms, such as wind actions) or discrete (for random phenomena, such as a the effect of mechanical agressions due to usage). In field conditions, loss of performance occurs due to a combination of the two.

The Overall Degradation Level ($ODL_{(T)}$) of a surveyed building or component expresses the actual degradation condition of the considered building or component at the time (T) of the survey [Gaspar, 2003]. For a statistically relevant sample, it is possible to identify a degradation path by means of regression analysis of the ODL of the surveyed elements [Shohet, 2002], corresponding to the degradation function $D_{(t)}$. In theory, once the degradation pattern of a building element is established, it is possible to estimate the service life of any similar building element in identical environmental conditions, knowing its' ODL, as shown in fig. 1.

2.1 Degradation Level assessment

Most field survey methodologies adapt a qualitative approach based on distinct degradation levels. Such levels are established according to deterioration parameters that can be used by different surveyors therefore minimizing the risk of subjectivity [Brandt & Wittchen 1999][Florentzou *et al.* 1999].

Qualitative degradation level assessment relies mostly on visually based data gathering methods which, despite all their shortcomings, are generally regarded as easiest to use, rapid to implement and last, but by no means least, relatively low-cost for extensive data samples [Balaras *et al.* 2004].

In these methods, buildings are divided into construction elements that are assessed independently, through degradation / performance levels [i.e. from “a” or “0” (good state) to “d” or “4” (replacement of element)]. Condition levels may vary according to the surveyed element and in order to ensure an uniform differentiation between levels of degradation, for, as Wittchen & Brandt [2002] stress, field work often demonstrates that too many differences may be found between two consecutive levels if samples do not exactly match either one of the two.

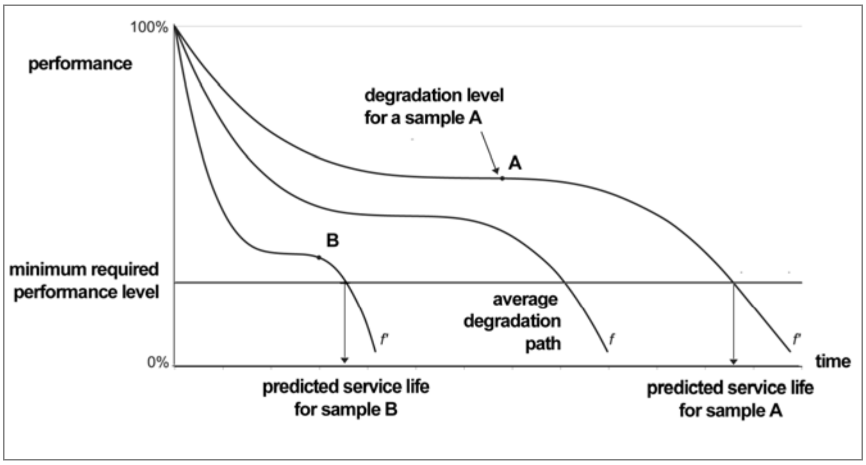

Figure 1. Degradation path for a building element; service life prediction of a given element.

Typical quality charts (as shown in fig. 2) offer a statical image of reality, in which degradation evolves as a discrete variable. Level distribution has limited direct applicability as far as service life prediction is concerned, unless somewhat complex statistical analysis is called for.

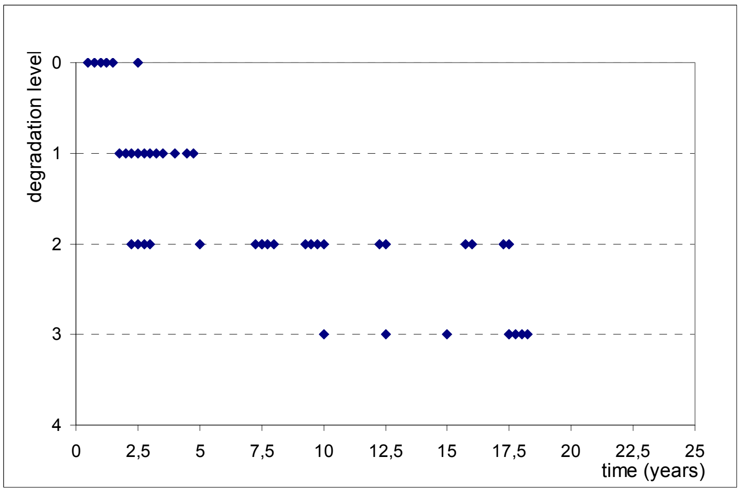

Figure 2. Degradation level of 44 surveyed mortar rendered façades, displayed as discrete variables. Ages have been incrementally increased for easier visualization.

2.2 Quantitative approach

In order to achieve quantitative distributions of degradation levels based on visual evaluation only, some methodologies adopt more detailed criteria in the assessment of deterioration:

a) Marteinsson & Jónsson [1999] suggest four well-defined degradation stages (less than 5%, 5 to 33%, 34 to 66% and 67 to 100% of deterioration) based on the crossing of frequency (which provides an amount of the type of defects: how often, how fast they occur) and scope of defects (ammount of affected surface).

b) Likewise, Shohet [2002] rates loss of performance on a scale from 0 to 100, according to 5 different condition levels. In these ratings, the combined physical (related to a specific failure mode) and visual scores (quality of defects) of defects are weighted into a single value.
c) More complex methodologies incorporate risk, for every building or component is faced with different local environmental and usage conditions. Chew *et al.* [2003] consider lack of quality / performance as a ratio between the registered deffects and the summation of all possible defects, computed with the contribution of poor workmanship. For each sample, the extent of defects is derived from an average between the spread (ammount of affected surface) and depth (physical failure mode) of defects.

3 RESEARCH METHODOLOGY

The presented methodology was developed following a preliminary assessment of the degradation level of cementitious mortar redendered façades [Gaspar & Brito 2003]. In this work it was noted that not only did similar façades showed distinct degradation patterns (due to different exposure conditions, i.e. façade orientation), but also different parts of the same surveyed façade displayed different patterns of degradation. Such differences were related a) to the nature of the failure mode that affected each part of the façade, b) to the extent of degradation (that is, the percentage of deteriorated surface) and c) to the depth of degradation (in other words, the severity of the degradation).

3.1 Data Collection

To understand the possible influence of degradation according to the part of the façade affected, a survey was carried out in which 150 samples were assessed in six different areas of the façade: (1) at or near floor level, (2) on continuous walls, (3) around openings, (4) at parapets, below copings and eaves, (5) below balconies and soffits and (6) in the corners and edges (fig. 3).

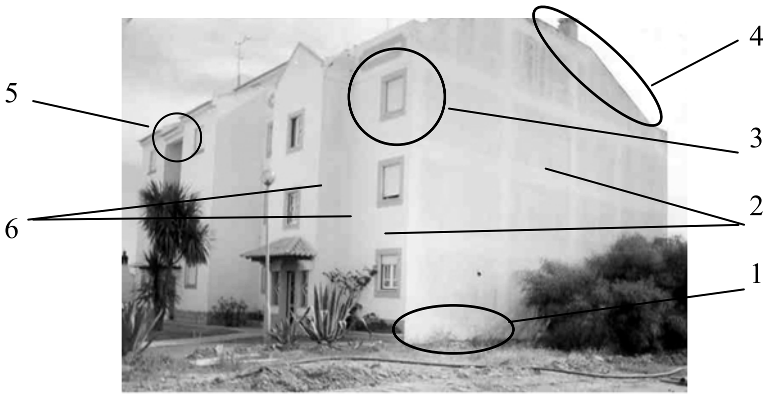

Figure 3. Identification of the surveyed parts.

The samples were randomly chosen in the population of the existing mortar rendered buildings, in three different locations in southern Portugal (Algarve, Lisbon and Alcochete). Each location was chosen for its specific environmental characteristics: mild marine influence (Algarve), strong marine influence (Alcochete) and urban areas, near traffics roads, but with no marine influence (Lisbon). Field work was carried out between August and December 2002. For each surveyed sample, a survey form was filled, with indication of the age of the building, the date and nature of the last intervention on the façade, the description of the building geometry, construction characteristics, environmental / site conditions and a description and characterization of the registered defects. In the end, 23 samples were rejected due to lack of reliable data related to maintenance management.

3.2 Overall degradation level estimation

Each of the six different areas of the façade was visually assessed. Defects were registered and rated into 5 degradation levels (from 0 (no degradation) to 4 (extensive degradation) according to the scope and depth of defects within each area, [Table 1]).

Degradation Level	*Defect characterization*	*Affected area (%)*
Level 0	. No visually detectable degradation	Less than 5
Level 1- good	. Surface staining . Eventual presence of capilarity cracking	5-10
Level 2 - light degradation	. Minor cracking (visible only with magnifying lens) . Localized presence of fungi . Possible way-ins for water or light signs of efflorescence . Light damp and moisture staining	11-30
Level 3 - broad degradation	. Localized cracking (visible with naked eye) . Damaged corners or edges . Localized infiltrations . Efflorescence . Surface damage (colour and texture)	31-50
Level 4 - extensive degradation	. Extensive cracking . Spalling or crumbling of surface . Extensive infiltration and surface damage . Broken or corroded steel elements . Loss of adhesion between layers . Detachment from wall	More than 50

Table 1. Deterioration assessment of cementitious mortar rendered façades

Overall Degradation Level was estimated by means of weighted average of the recorded degradation levels, as indicated in expression (1).

$$ODL = (n_1 \times 1 + n_2 \times 2 + n_3 \times 3 + n_4 \times 4) / \Sigma (n_1 + n_2 + n_3 + n_4) \qquad (1)$$

ODL - overall level of degradation of a façade surveyed;
n_i - Σ of defects of level i (0 to 4) detected (scope + depth).

3.3 Deterioration patterns

Previous investigation [Gaspar 2003] pointed out to the fact that cementitious mortar degradation is not a rectilinear function, but a three stage process (comprising rapid early degradation, a maturing period during which degradation seems to become stable and, in the end, an increasingly fast end of service life) best expressed through polynomial functions. Deterioration patterns for each of the surveyed groups were therefore plotted through third degree polynomial regression analysis (figures 4 to 6).

In the examples depicted, it is clear the still somewhat rather large scatter of data (R^2 around 0,72 for locations under marine influence and 0,62 with no marine influence). This may be explained due to the still small amount of façades surveyed (50 for each location) or to the fact that, due to the randomly character of the sample, local and micro-environmental factors may affect the results. Last but not least, it should be noted that great levels of uncertainty were to expected due to the poor workmanship and maintenance levels registered, especially for mortars executed more than 10 years ago.

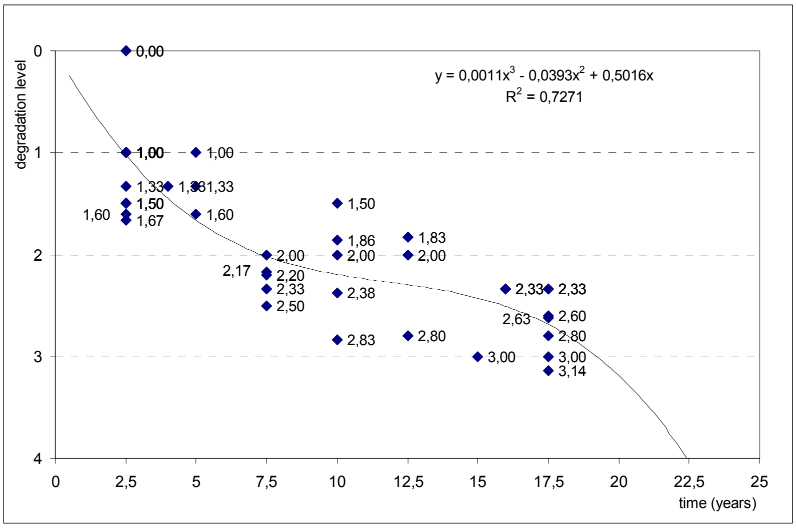

Figure 4. Degradation curve of 45 surveyed façades in the Algarve (mild marine influence).

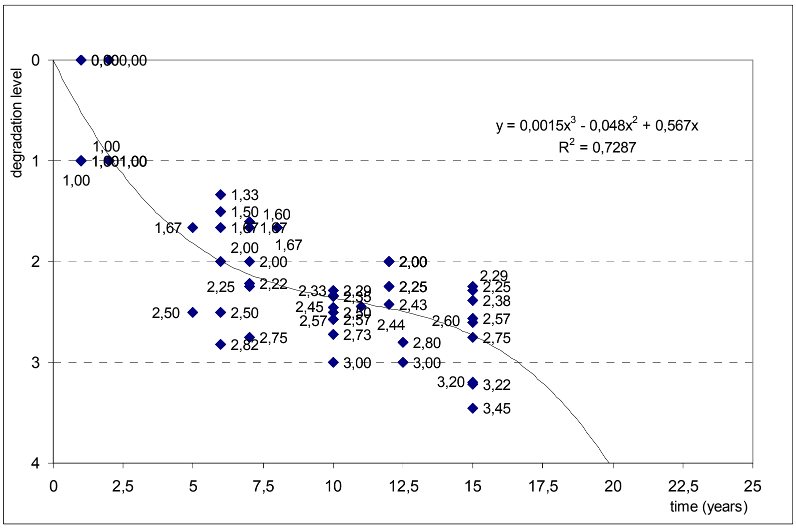

Figure 5. Degradation curve of 52 surveyed façades in Alcochete (a marine environment with direct winds from the sea).

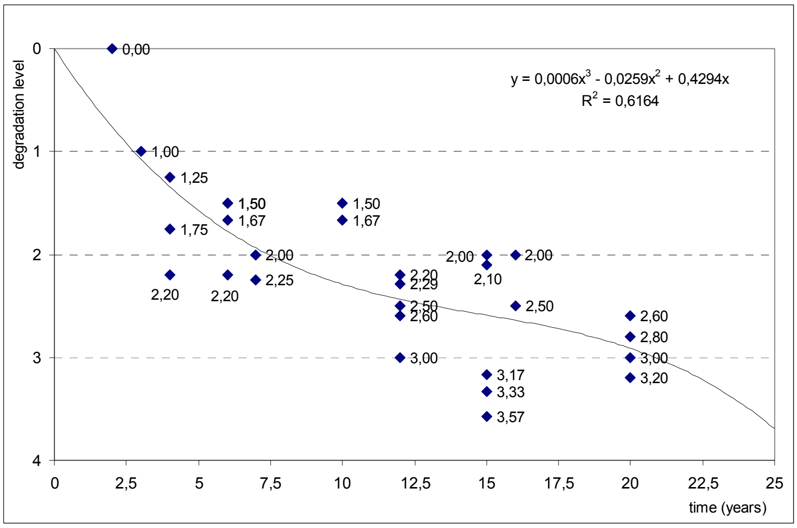

Figure 6. Degradation curve of 30 surveyed facades in Lisbon (urban environment, near major traffic roads, no marine influence).

The deterioration patterns obtained also express entropy phenomena between degradation mechanisms. These happen whenever more than one failure condition occur simultaneously thus increasing the rate of deterioration. Such a situation might happen, among several other possible reasons, due to existing small distributed cracking on the surface of the mortar, allowing in moisture penetration and possible sulfate attack of the matrix of the mortar.

3.4 Service life prediction in the surveyed elements

Although caution should be used in drawing statistically validated conclusions from the field work done, it is possible to establish some principles related to the service life of cementitious mortar renders in façades located in the three surveyed areas:

a) Degradation patterns in mortar renders, in regular field conditions, and with no corrective statistical procedures (such as, considering only mortars with standard workmanship or maintenance levels), seem to follow a three stage deterioration process (initiation - for defects that occur early in service life - maturing and end of service life anomalies);
b) Although a three stage degradation process is especially true for mortars located within marine environmental influence, in the case of mortars located in urban areas with no marine influence a pattern more similar to a logaritmical degradation curve might also be considered. This situation requires further research though, since the field data for this location produced the most inconclusive results;
c) Average expected service life prediction for the surveyed samples, assuming that the minimum required performance degradation corresponds to level 3) [Gaspar 2003] was 17, 19 and 21 years for direct marine influence, mild marine influence and no marine influence, respectively. Naturally renders can have increased or reduced expected service life according to their registered degradation level at the time of the survey (i.e. if they show better or worse performance than the registered average of the ODL, respectively). Likewise, the predicted service life varies according to the required perforce level (refer to fig. 1);
d) The obtained results demonstrate that broad generalizations related to the expected service life of a given building component (particularly if exposed to outdoor conditions) are difficult to establish, even for very similar geographical locations, due to micro-environmental effects that affect each element. From this point of view, every building or component has several degradation patterns, depending on its exposure conditions, that ought to be identified independently, as opposed to one ‘universal’ pattern.

4 CONCLUSIONS

This paper outlines some of the available methods for assessing the degradation level of existing buildings and components, from a quantitative point of view (Overall Degradation Level), and discusses a methodology implemented in a visual survey of the degradation level of 150 cementitious mortar rendered façades, in three distinct locations in southern Portugal, between August and December 2002.

This methodology consisted in an independent degradation level assessment of six different parts of each façade [(1) at or near floor level, (2) on continuous walls, (3) around openings, (4) at parapets, below copings and eaves, (5) below balconies or hanging volumes and (6) in the corners and edges], as means to avoid subjective input from the surveyor, when asked to consider the degradation level of the whole façade. For each part of the façade, a visual assessment was made of the scope and depth of defects, rated into five different degradation levels. These levels were later computed into a weighted average figure that expresses the overall degradation level of the façade on the moment of inspection.

Field work results were deemed still unreliable from a statistical point of view, especially when compared to laboratory ageing tests. However, as opposed to the latter, field surveys may provide the means of understanding entropy phenomena that occur under the influence several simultaneous degradation agents (wind, moisture, UV radiation, driving rain and usage, for instance).

Further research is presently under way to reach the following goals:

a) Develop field work methodologies that are less time consuming, especially when determining the scope of degradation (i.e. the percentage of affected area);
b) Incorporate risk analysis on the results obtained in order to deal with the uncertainty related to workmanship and maintenance standards;
c) Determine hierarchical relations between different failure modes;
d) Establish proof of and quantify entropy phenomena that affect the service life of mortar rendered façades service life so that better statistical models can be developed.

5 REFERENCES

Architectural Institute of Japan, 1993, 'The English Edition of Principal Guide for Service Life Planning of Buildings', Architectural Institute of Japan, Japan, 1993, 98 p.

Balaras, C., Droutsa, K., Daskali, E., Kontoyiannidis, S., 2004, *Building Deterioration Assessment Guidelines*, in Investimmo Report 62-098, March 2004, National Observatory of Athens, Greece, 281 p.

Brandt, E. & Wittchen, K.B., 1999, 'EPIQR - A new surveying tool for maintenance and refurbishment", Proc. 8th Durability of Building Materials and Components, Institute for Research in Construction, Ottawa, Canada, 1999, pp. 1576-1584.

Chew, M., Silva, N. & Tan, S. 2003, 'Design for maintainability', Proc. 2th Int. Symp. on Building Pathology, Durability and Rehabilitation, CIB/W086, Lisbon, Portugal, November 2003, pp. 587-596.

Florentzou, F., Brandt, E. & Wetzel, C., 1999, 'MEDIC - A method for predicting residual service life and refurbihment investment budgets", Proc. 8th Durability of Building Materials and Components, Institute for Research in Construction, Ottawa, Canada, 1999, pp. 1280-1288.

Gaspar, P. & Brito, J. de, 2003, 'Diagnosis and repair of defects on mortar renders in recent constructions' (in Portuguese), Proc. 1º Encontro Nacional sobre Patologia e Reabilitaçao de Edificios, FEUP, Oporto, March 2003, 197-206.

Gaspar, P., 2003, 'Methodology for service life prediction of external cementitious renders', MSc. Thesis (in Portuguese), IST - Technical University of Lisbon, April 2003, 201 p.

Hovde, P., 2004, *State of the Art Report A - Factor Methods for Service Life Prediction*, in Performance Based Methods for Service Life Prediction, CIB W080 Report 294, March 2004, Trondheim, Norway, pp. 3-51.

International Organization for Standardization, 2001, ISO 15686 *Building and construction assets – Service Life Planning, Parts & 2*, International Organization for Standardization, Geneve, Switzerland.

Marteinsson, B. & Jónsson, B. 1999, 'Overall survey of buildings - performance and maintenance", Proc. 8th Durability of Building Materials and Components, Institute for Research in Construction, Ottawa, Canada, 1999, pp. 1634-1654.

Shohet, I., Putterman, M. & Gilboa, E. 2002, 'Deterioration patterns of building cladding components for maintenance management', *Construction Management and Economics,* **20**[4], 305-314.

Wittchen, K.B., Brandt, E., 2002, 'Development of a methodology for selecting office building upgrading solutions based on a test survey in European buildings', *Energy and Buildings,* **34**, 163-169.

Sensitivity of Brittleness of The Concrete To The Pore Structure of The Cement Paste

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ABSTRACT

One of the essential factors which affects the performance of the concrete and reinforced concrete buildings under earthquake loads is the degree of brittleness shown by the concrete, essentially a quasibrittle material, against these effects. It is possible to investigate the brittleness of the concrete directly by dynamic experiments, as well as the static experiments which make it possible to get some numerical values for brittleness. In this work, the second way is choosen and the brittleness of the concrete is expressed as the ratio between the compressive strength and the tension strength of the concrete and the brittleness index.

In this work, the relationship between the brittleness and the grading of the aggregate mixture and the pore structure of the cement paste is investigated; the degree of sensitivity of the values determining the brittleness to the pore structure of the cement paste is also determined; then the relations between these values are found,and the factors determining the correlation constants of these relations are investigated.

The maximum size of aggregate used in concrete is 16 mm. The grading is held within the permitted limits. The cement content is constant as 350 kg/m3,whereas water/cement ratio changes between 0,50 and 0,70. Aggregate is normal concrete aggregate and the cement is Portland Cement 32,5.

These results are obtained from the investigation; (i) While the water/cement ratio increases,the unit weight, the tensile strength, the compressive strength, the brittleness index and the modulus of the elasticity decreases. (ii) The grading of the aggregate does not effect the unit weight and the brittleness index pratically. On the other hand, the tensile strength and the compressive strengths and the modulus of the elasticity have the lowest values compared to the others when the grading is between B16-C16. This situation is probably due to increased air content. (iii) While the compressive strength increases,the brittleness index and the ratio between the compressive strength and the tensile strength increases. It is said that while the strength increases, the concrete becomes more brittle.(iv)The correlations between properties are strong because the sensitivities of these properties to the pore structure of the cement paste are close.

KEYWORDS

Brittleness, correlation coefficents, the pore structure

1.INTRODUCTION

All concrete properties are related to the inner structure of the cement paste. In the modern engineering, understanding the effect of the inner structure of the cement paste or how the materials of the concrete change the inner structure of the cement paste or the effects of the interface between the cement paste and the aggregates is important that concrete is produced according to the requested conditions. The inner structure has an essential place for the determination the properties of the concrete because the properties of the concrete depend on the reactions of the hydration occurring in the cement paste.

From the past to the present, within the help of the developing technology, the inner structure of the cement paste is continuosly researched. The first researches of the inner structure of the cement paste are the crystallization theory of Le Chatelier and the Michealis' colladial theory [1]. Powers and his friends proved that as a result of the cement hydration, the fibres like Labermant, forms in the cement paste and these fibres are in colladial dimension and they have a great inner surface. Also, the structure of these fibres is gel. Besides this gel, in the cement paste, there are calcium hydroxide crystals, the secondary components, unhydrated cement particles and the pores. These pores are surrounded by the gel. Since the size of these pores is capillary, they are called as capillary pores [2].

2.THE STRUCTURE OF THE CEMENT AND THE CEMENT PASTE

There are 4 main components of the cement; C_2S, C_3S, C_3A and C_4AF. After the hydration, when the cement is mixed with water, these components turn into the hydrated components. The hydrated components are C-S-H (Calcium Silicate Hydrate) and CH (Calcium Silicate). C-S-H supply the binding property of the cement.

The inner structure of the cement paste is porous. The pores form as a result of the hydration reactions because of the formed hydrated products. These pores are classified as the capillary and the gel pores. The capillary poıres are formed after the hydration when used water evaporates. The capillary porosity depends on the water/cement ratio and the hydration degree. As the hydration degree increasaes, the more hydrated products are formed and they fill the pores so the numbers of the pores decrease. However, when the water/cement ratio increases, it is impossible that the hydrated products fill the pores because the amount of the water increases and then the numbers of the pores increases,too so the formed hydrated products are not enough to fill these pores. The gel pores are in the hydration products and the numbers and the total volume of the gel pores increase within the the hydration process. The dimension of the gel pores is smaller than the dimension of the capillary pores. Besides these pores, there are air voids,the dimension of which is more smaller . The air voids are formed because of the insufficient settlement of the concrete or some additives which drag air in the concrete.

2.1 The inner structure of the interface between the cement paste and the aggregate

The interface between the cement paste and the aggregate is called as the weakest chain in the concrete. The inner structure of the interface between the cement paste and the aggregate differs from the inner structure of the cement paste. There are various models for the micro structure of the interface.

According to Barnes;
1. A film of CH acculumates vertically on the aggregates.
2. A film of C-S-H particles, like a hair brush, covers the aggregates.
3. Parallel to the aggregate surface, extensive CH crystals acculumate.
4. At the inteface zone, full volume formation of CH is seen.

According to Zimbelmann[3]; when the cement paste is mixed with water, in a few minutes, the etthrengite needles are formed. CH crystals vertically cover the etthrengite needles. The thickness of the level of CH crystals is 2-3 micron. After this level, hegzagonal CH crystals begin to develop. The dimension of these hegzagonal crystals changes between 10 and 30 micron.

At the interface, the porosity is high so the interface is weak and permeable. The thickness of the interface changes between 10 and 50 micron. The interface comprises the huge part of the concrete.

There are some ideas for the formation of the interface. During the mixture of the concrete, the aggregate particles are covered by a film of water. When the hydration starts, forming hydration products connect with this film of water so the water/cement ratio of the interface is higher than the original water/cement ratio.

3. THE PORE STRUCTURE OF THE HARDENED CEMENT PASTE

In the fresh cement paste, the Portland cement paste is fluidy. However, in the hardened paste, there are hard hydration products and the pores. The hydration products form high dense masses (the gel pores) and they have C-S-H. Besides, there are capillary pores and unhydrated cement particles. The total porosity inolves the capillary and the gel pores. The dimension of these pores is microscopic and the gel pores are smaller than the capillary pores. The physical differences of the cement pastes depend on the difference between the capillary pores.

3.1 The water in the hardened cement paste

There are 3 types of water in the hardened cement paste[4],[5],[6];
1. The chemical water is an integral part of the cement gel. This water is not evaporated.
2. The pyhsically absorbed water is absorbed from the surface of the gel particles. The absorption is the surface reactivity of the gel. This water fills the gel pores.
3. The free water is the left water in the saturated paste and fills the capilllary pores.

The physically absorbed water and the free water are able to evaporate.

3.2 The properties of the pore structurte of the hardened cement paste

The total porosity decreases during the hydration process for the all water/cement ratios. At the first periods of the hydration, the decreasing ratio of the total porosity is high, but in 3 months, the ratio is linear. Also, the total porosity increases when the water/cement ratio increases because an increase in the amount of water between the unhydrated particles. The dimension of the distribution of the all pores gets better within the age of the hardened cement paste [7].

3.3 The effect of the pore structure of the cement paste on the mechanical properties of the concrete

The porosity is the most important factor which specifies the strength of the concrete. The radius of the pores which is smaller than 10 nm decreases the strength of the concrete. The constant porosity increases within the increasing amount of the hydration products.

4. THE QUASI-BRITTLENESS OF THE CONCRETE

Aitcin and Mehta proved that the thickness of the loading-unloading curves specify the strength of the the interface between the aggregate and the cement paste [8]. The concretes having the same water/cement ratios and different granulumetries give different results. For example, in the experiments, the elasticity modulus and the compressive strength of the granulumetry between A16-B16 are the highest,whereas the granulumetry between B16-C16 is the lowest. The compressive strength results are shown in Figure 4.1.The right curve is for the granulumetry between A16-B16 and has the highest compressive strength.

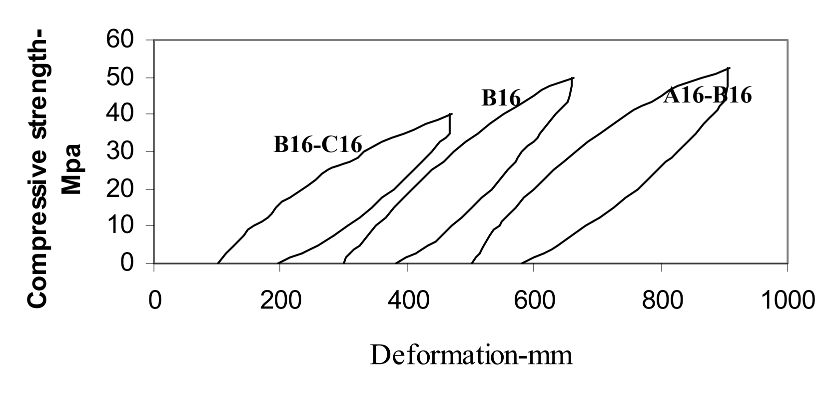

Figure 4.1 The curves of the granulumetries between A16-B16, B16, B16-C16

The increasing thickness of the loading-unloading curves reveal that the bonds of the interface are weak. Also, according to the loading-uploading curves, when the compressive strength increases, the brittleness index increases. With the increasing strength of the concrete, the interface become unporous and homogen.

5. THE SENSITIVITY OF THE PROPERTIES OF THE CONCRETE TO THE PORE STRUCTURE OF THE CEMENT PASTE

If one of the properties of the concrete such as the compressive strength, the tensile strength or the elasticity modulus, only depends on the amount of the pores in the cement paste, the property of the concrete is insensitive to the pore structure of the ceement paste. If the type (or the geometry) of the pores also specifies the property of the concrete, the property of the concrete is sensitive to the pore structure of the cement paste [9]. Therefore, the correlation coefficients are useful to understand whether the property is sensitive or not sensitive to the pore structure of the cement paste.

5.1 The sensitivity of the properties of the fresh concrete to the pore structure of the cement paste- the sensitivity coefficient

If P_{fi} is one of the properties of the fresh concrete such as the unit weight or the slump and n_i is a parameter, the values of n_i are calculated with the obtained values of P_{fi} from the experiments according to the formulation;

$n_i.w + a$ w: the amount of water in the fresh concrete a: the percentage of the air voids

$n_i = n_i^*$,the maximum correlation coefficient is calculated. In the Figure 5.1, the y-axis shows the values of P_{fi} which is obtianed from the experiments, on the other hand the x-axis shows the values of $n_i.w + a$ which are calculated according to the values of P_{fi}. The graphic is drawn according to these values and then the peak point of the graphic is accepted as the maximum correlation coefficient of the property of the fresh concrete.Actually, after the graphic is drawn in Excel, the regration coefficient is found in the tendency of the graphic in Excel and then the square root of the regration coefficient of the graphic ($\sqrt{R^2}$) is accepted as the maximum correlation coefficient (n_i^*).
If $n_i^* = 1$ The property is insensitive to the pore structure of the cement paste
$n_i^* \neq 1$ The property is sensitive to the pore structure of the cement paste
Whether if the properties of the hardened concrete is sensitive or not, $n_i.(w - 1{,}06.\alpha.c) + a$ is used instead of $n_i.w + a$ to consider the hydration of the cement paste. w is the amount of the water in the concrete, α is the degree of the hydration, c is the amount of the cement (the doasge of the concrete) and

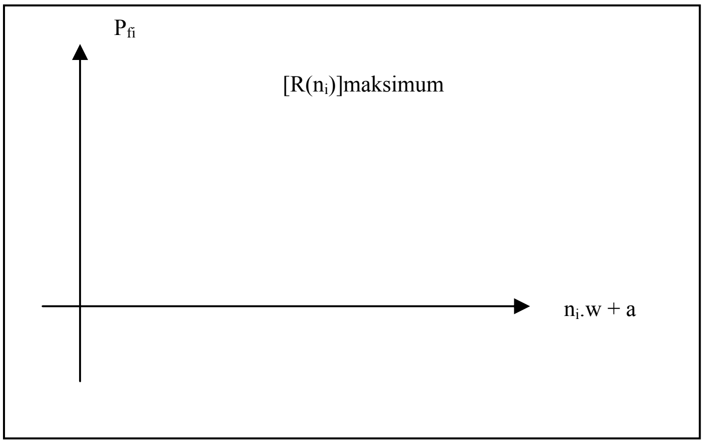

Figure 5.1 The relationship between P_{fi} the property of the fresh concrete and $n_i.w + a$

a is the percentage of the air voids in the concrete. The same processes are done to calculate the maximum correlation coefficient of the property of the hardened concrete.

Another way of calculating the maximum correlation coefficient is comparing with the unit weight of the concrete. The degree of the hydration is hard to be found so an approximate method should be used. The dry unit weight of the concret is calculated in the experiments. The unit weight is insensitive to the pore structure of the cement paste since the effect of the pores on the unit weight is just related to the volume of the pores, not the geometry and the dimension so $n_i^* = 1$ for the unit weight. Therefore the formulation $n_i.(w - 1{,}06.\alpha.c) + a$ changes into $(1-1{,}06.\alpha.c/w).w + a$. However, it is hard to calculate the correlation coefficients with these variants so the formulation is simplified as k.w + a. The sensitivity of the unit weight should be investigated according to this formulationa and the maximum correlation coefficient (k=ko) of the unit weight is calculated. Moreover, k.w + a is used to investigate the sensitivity of a property of the hardened concrete and then a maximum correlation coefficient of the property of the hardened cement ($k=ki^*$) is calculated. Here, ko and ki^* are accepted as the central tendency parameter. The relationship between ko and ki^* determines the sensitivity of the property of the hardened concrete compared to the sensitivity of the dry unit weight.

If $ko = ki^*$ The property is affected by the pore structure of the cement paste like the dry unit weight so the property is insensitive.
If $ko \neq ki^*$ The property is sensitive.

5.2 The first type degree of the sensitivity of the properties of the concrete to the pore structure of the cement paste- (SD1)i

When the first type degree of the sensitvity of one of the property of the hardened concrete to the pore structure of the cement paste is investigated, the ratio of $ki^*$and ko is calculated [$(SD1)i = ki^*/ko$]. The respect of some reseraches, (SD1)i is close to the sensitivity coefficient,n_i^*.

5.3 The second type degree of the sensitivity of the properties of the concrete to the pore structure of the cement paste-(SD2)i

When the second type degree of the sensitvity of one of the property of the hardened concrete to the pore structure of the cement paste is investigated, the ratio of $R(ki^*)$ and Ri(ko) is calculated.[$(SD2)i = R(ki^*)/R(ko)$]. Here, $R(ki^*)$ is the maximum correlation coefficient for the property of the hardened concrete which is calculated from k.w +a. R(ko) is the maximum correlation of the property of the hardened concrete by using ko (the maximum correlation coefficient of the dry unit weight) instead of k in k.w +a. The second type degree of the sensitivity is not more affected by the function of the inner structure so it is necessary to research the correlations between the properties in order to understand whether the secon type degree is significant or not.

5.4 The factors of the correlations between the properties of the concrete

The sensitivity similarity coefficient; tij = (SD)i/(SD)j and tij < 1. Here (SD)i is 1. or 2. type degree of the sensitivity of P_{hi},one of the property of the hardened concrete and (SD)j is 1. or 2. type degree of the P_{hj}, another property of the hardened concrete. tij changes between 0 and 1. If tij is close to 1, P_{hi} and P_{hj} are similar for the sensitivity to the pore structure of the cement paste.

There are other relationships of the similarity of the different sides between the correlations and the properties. The above formulation helps to understand the other relationships [10].

$(P_{hi})' = A(c + w + a)' + B(c/[k.w + a])' + C(m)' + D$

Here ( )' are the ratio of the average in the serie of the variants in the paranthesis and how P_{hi}, the property of the hardened cement is affected by relatively changes in the volume of the cmenet paste, the structure of the cmenet paste and the granulumetry of the aggregates. In order to understand the similarity of the coefficients A, B and C of two different properties of the hardened concrete, the slope of the coefficients should be calculated in T(A)ij = |Ai-Aj| , T(B)ij = |Bi-Bj|, T(C)ij = |Ci-Cj|. If T is more bigger, A, B and C are too different.

6. EXPERIMENTS

The Portland Cement 32,5 is used and the specific gravity of the Portland Cement is 2,97 gr/cm^3. The dosage of the concrete is 350 kg/m^3. The aggregates are broken stone 1, the sand and the stone powder. the specific gravity of the aggregates are relatively 2,72 gr/cm^3, 2,60 gr/cm^3 and 2,70 gr/cm^3.For the granulumetry between A16-B16, the mixture ratios are 70 % the broken stone 1, 20 % the sand and 10 % the stone powder. For B16 granulumetry, the mixture ratios are 65 % the broken stone 1, 25 % the sand and 10 % the stone powder. For the granulumetry between B16-C16, the mixture ratios are 60 % the broken stone 1, 30 % the sand and 10 % the stone powder. For every granulumetry, water/cement ratio begins at 0,50 and it is increased by 0,05 to 0,70. 15 types of the concrete are produced. The beginning value of the water/cement ratio is determined with the help of the VeBe experiment. The water/cement ratio of the fresh concrete which gives 7-8 sn in VeBe machine is accepted as the original water/cement ratio. For one water/cement ratio of every granulumetry, 150*150 mm 4 cubes, 150*300 mm 8 cylinders and 60*150 mm 6 discs are produced.

After 24 hours, the samples are taken out from the moulds and then the samples are held in 23 ± 2^0C water for 21 days. At 21.day, the samples are taken out from the water and waited in the room to the 28.day. At 28.day, the compressive strengths of the cylinders and the splitting strengths of the discs are measured. The cubes are held in 105^0C oven for 4 hours at 35.day and they are weighed and the dimensions of the cubes are measured in order to calculate the dry unit weight. The discs are used to measure the splitting strengths. The capacity of Amsler pressure instrument for the splitting tests is 500 kN. To distribute the load unifomly on the discs, two laths are mutually put on the upper and the lower part of the discs. The half of the cylinders are used to measure the compressive strengths and the capacity of Amsler pressure instrument for the compressive strength is 1000 kN. The speed of the loading is constant and for 2,5 tone, longitunal deformations are measured with the help of a frame. The elasticity moduluses are calculated within the help of these longitunal deformations. The other half of the cylinders are used to obtain the brittleness indexes. Firstly, the samples are loaded to a certain value and then the samples are unloaded. After unloading, the samples are loaded to the fracture. During this process, longitunal deformations are measured. The certain value is theoratically 80-90 % of the compressive strength of the sample. The graphic is drawn according to these results are shown in Figure 6.1. In Figure 6.1, S_I is the permanent deformation energy related to the damage and S_{II} is the elastic deformation energy. The x-axis shows the longitunal deformation (mm) and the y-axis shows the strength of the samples (MPa). The brittleness index is the ratio between S_I and S_{II} (S_{II}/S_I).

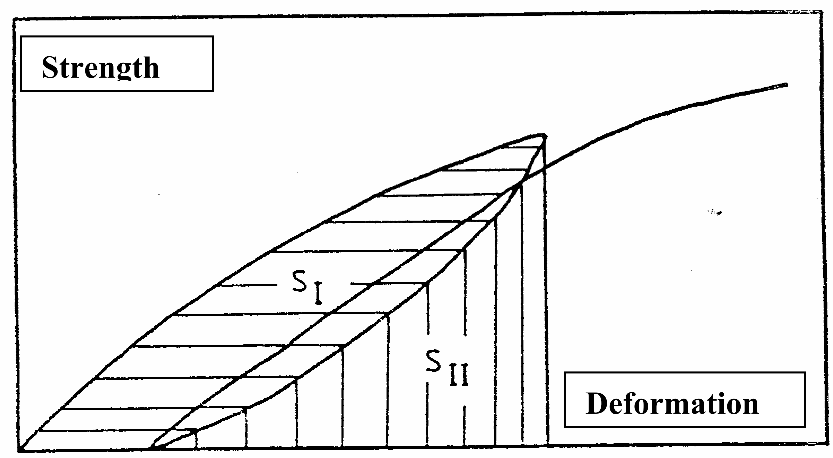

Figure 6.1 The strength-deformation curves of the repetition loading of the concrete

In Table 6.1, the brittleness indexes obtained from the graphics of the tests.The brittleness indexes decrease when the water/cement raito increases. For the lowest water/cement ratios, the compressive strength and the brittleness index are high so it means that the concrete becomes brittle.

The granulumetries	The water/cement ratios				
	0.50	0.55	0.60	0.65	0.70
A16-B16	3.65	2.90	2.71	2.40	1.70
B16	3.21	2.56	2.37	2.30	1.93
B16-C16	2.80	2.52	2.36	2.29	1.68

Table 6.1 The brittleness indexes related to the changes of the water/cement ratios

By using the results obtained from the tests, in the formulations A(k.w + a) + B, A(c/k.w + a) + B,A(c + w + a) + B(c/k.w + a) + C and A(c + w + a) + B(c/k.w + a) + C(m) + D, the correlation coefficients of the properties of the hardened concrete are calculated for every formulations. In Table 6.2, the correlation coefficients of some properties of the concrete obtained from the formulation A(k.w +a) + B are seen.

Property	ki*	(SD1)i	R(ki*)	R(ko)	(SD2)i
Cylinder Compressive Strength-MPa	1,04	0,301	0,9715	0,9321	1,04
Dry unit weight-kg/cm^3	3,46	1,000	0,8924	0,8924	1,00
Splitting Strength-MPa	0,99	0,286	0,8150	0,7758	1,05
Brittleness Index	1,38	0,399	0,9347	0,9192	1,02
Elasticity Modulus-MPa	1,35	0,390	0,9322	0,9154	1,02

Table 6.2 ki*, (SD1)i, R(ki*), R(ko) and (SD2)i of the properties according to the formulation A(k.w + a) + B

7.CONCLUSIONS

With the increasing water/cement ratio, the dry unit weight, the splitting and the compressive strengths, the brittleness index and the elasticity modulus decrease. The granulumetric composition of the aggregate does not pratically effect the unit weight and the brittleness index. However, the compressive and the splitting strengths and the elasticity modulus are a little smaller than the unit weight and the brittleness index for the granulumetry between B16-C16 because of the more air voids in the granulumetry between B16-C16. When the compressive strength increases, the brittleness index and the ratio between the compressive strength and the splitting strength rises. The fact reveals that when the strength increases, the concrete becomes brittle. The correlations between the properties are strong because the sensitivity of the properties to the pore structure of the cement paste is similar.

8.REFERENCES

[1] Brandt,A.M.,1999, Cement and Other Binders, Cement Based Composites, Materials Mechanical Properties and Performance,pp 47-52,London.
[2] Diamond,S.,1995, Microstructure of Cement Based Materials/Bonding and Interfaces in Cementitious Materials, Materials Research Center,Vol:370.
[3] Zimbelmann,R.,1989, A Contribution to The Problem of Cement-Aggregate Bond, Cement and Concrete Research,15,pp 801-808.
[4] Leo,F.M.,1970, The Chemistry of Cement and Concrete,third Edition,Edward Arnold Publishers Ltd.,London
[5] Hansen,C.T., Radjy,F. and Sellevold,E.V.,1973, Cement Paste and Concrete, Annual Review of Materials, Vol 3,pp:233-268,Denmark.
[6] Sereda,P.J.,Feldman,R.F. and Romachnadran,V.S.,1980, Structure Formation and Development in Hardened Cement Pastes,Proc.7th International Congress on The Chemistry of Cement,Vol 1,Paris.
[7] Okpala,D.C.,1999, Pore Structure of Hardened Cement Paste and Mortar,The International Journal of Cement Composites and Lightweight Concrete,Vol 11,No:4,USA.
[8] Mehta,P.K. qnd Aitcin,P.C., 1990, Microstructural Basis of Selection of Materials and Mix Proportions For High Strength Concret,2nd International Symposium onHigh Strength Concrete,pp 265-285,Berkeley,California.
[9] Oktar,O.N., Moral,H. and Tasdemir,M.A.,1984, Sensitivity of Concrete Properties to The Pore Structure of Hardened Cement Paste,Cement and Concrete Research,Vol26,No:11,1619-1627,Istanbul
[10] Oktar,O.N., Moral,H, and Tasdemir,M.A.,1984,Factors Determining The Correlations Between Concrete Research,Vol 26 No:11,16191627,Istanbul.

Evaluation of A Newly Developed Crack Width Measurement Instrument

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ABSTRACT

A new portable instrument that is based on optical principles and is quantitatively able to measure crack width on a structural surface has been developed to deal with the increasing problem of crack width measurement in existing buildings and structures.

The instrument's accuracy and applicability were evaluated to determine its suitability for practical use. The results of the experimental evaluation are as follows:
1) The instrument is generally applied to ordinary building materials but cannot be used for clear glass structures or materials.
2) It is essential to apply an opaque sheet when the material surface being measured around the crack is blue or a deeply colored ceramic tile.
3) Careful measurement is necessary when an undulation on the measured material surface extends over the crack width and the magnitude of the undulation exceeds 1.0 mm.
4) The region around the crack on the measured material surface has to be at an angle of under 41 degrees and must have no defects at the corners.
5) The operator's experience with the instrument and individual skill affect the measurement result. However, they do not affect the results after measurement has been conducted a few times by the same operator. Furthermore, using an auxiliary sheet for the measurement gives a better measurement result.
6) In cases where it is difficult to determine whether the instrument can be applied to some material, the instrument can be used for measurement if it can be confirmed that there are no differences between measurements made with a loupe and the instrument.

As the above shows, the newly developed instrument can measure crack width more accurately and can decrease operator error except under certain conditions related to color, undulation and unevenness of the measured material surface.

KEYWORDS

Building diagnosis, Crack width measurement, Deterioration inspection, Optical instrument

1 INTRODUCTION

In Japan, existing buildings and structures have deteriorated over the time that has elapsed since their completion. Therefore, structure diagnosis is becoming increasingly important and the need to conduct crack width measurement in existing buildings and structures has become more pressing.

Against this background, a new portable instrument that is based on optical principles and is quantitatively able to measure crack width on a structural surface has been developed

The measurement accuracy and the range of application must be clarified if the instrument is to come into wider use, and an objective evaluation is therefore required.

2 OUTLINE OF THE CRACK WIDTH INSTRUMENT

A plastic film with a graduation has generally been used for measuring the width of cracks in concrete structures. It is named the plastic film crack scale in Japan.

This instrument has been developed as a replacement for the conventional crack scale. The instrument is pressed against the crack on the concrete surface and it indicates the magnitude of the crack width instantly. It is portable and can be used to measure cracks from 0.05 – 2.0 mm in 0.05 mm increments.

2.1 Shape and size of the instrument

The dimensions of the instrument are 100 X 70 X 25 mm and it weighs 105 g. The appearance and name of each part are indicated in Fig. 1.

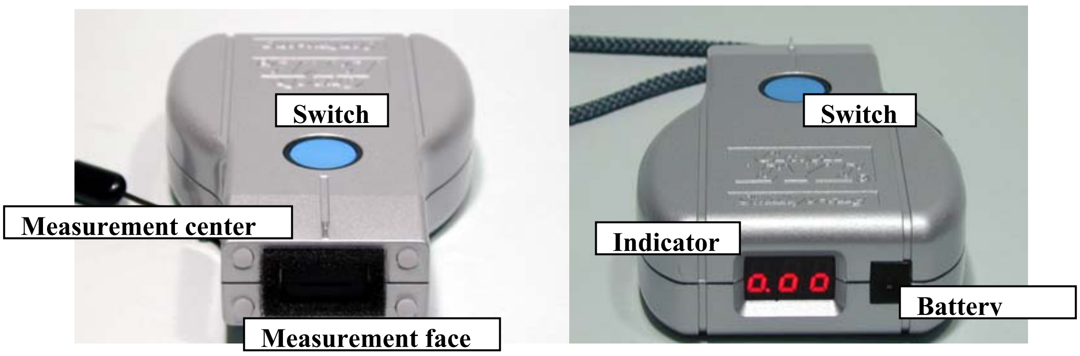

Fig. 1. Appearance and part names

2.2 Measurement method

The measurement method is shown in Fig. 2.
The instrument radiates a beam of light from both sides of the measurement face and the beams cast shadows of each side of the crack onto its opposite face. The shaded part (A in Fig. 2) is detected with a charge-coupled device (CCD) and the crack width is indicated. The instrument measures the difference in contrast between the surfaces illuminated by the beam and the dark depths of the crack.

If, for example, a black line is printed on a white background, the width of the black line can be measured by this instrument.

The sensitivity of the instrument can be controlled with a measurement aid sheet applying the above principle.

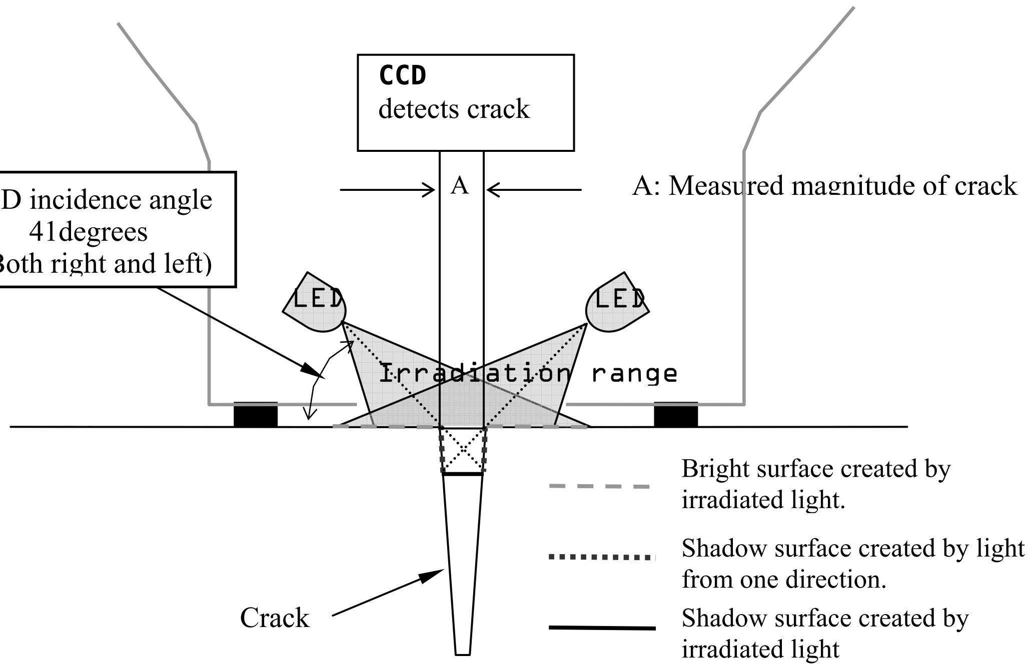

Fig. 2. Crack width detection method

2.3 Control of measurement sensitivity

The measurement sensitivity of the instrument is controlled by using the attached measurement aid sheet indicated in Fig. 3. It provides a standard width of 1.0 $\pm$ 0.01 mm. If the error of three measurements of the standard is within $\pm$ 0.05 mm, the instrument is working normally. Any differences in the power of the light-emitting diode (LEDs) and the sensitivity of the CCD are corrected for by calibration with this 1.0 mm wide line, to minimize any difference between instruments.

The measurement aid sheet is made by a film process, because it is difficult to achieve the required $\pm$ 0.1 mm design accuracy with ordinary printing. The thermal expansion of the measurement aid sheet is 1.0 X 10^{-3} %/C° and its stability are excellent at ordinary temperatures, though it may be degraded at temperatures above 70 C°.

If the instrument is tilted when held against the crack, the tilted width is measured, which includes some depth as well as width, and is thus likely to be larger than the true value.

Fig. 3. Measurement with the aid sheet

3 EVALUATION OF THE INSTRUMENT

3.1 Measurement of several types of specimens

Measurements of cracks in ordinary building materials (concrete, steel and textured sprayed finish) are shown in Fig. 4. It was judged that the results were equivalent to measurements obtained with crack scale or loupe for all the specimens examined. As the crack width grew, the difference in the results increased. However, It is necessary to take into account those errors peculiar to crack scale and loupe measurements, which increase with the size of the reading.

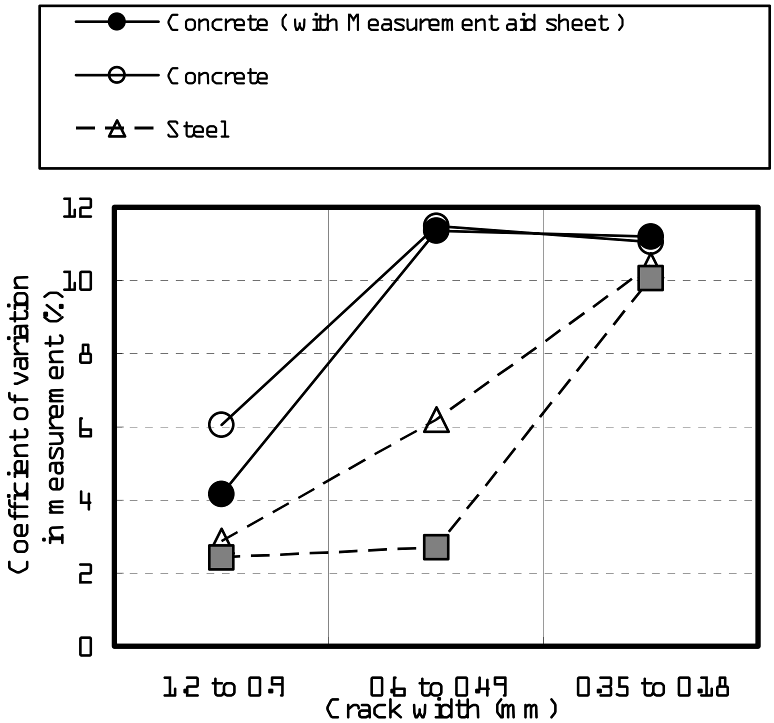

Fig. 4. Measurement result for ordinary building materials

On the other hand, when several operators made 10 measurements at the same place on the specimen in a short time, the results were repeatable, with only a small scatter around the mean value. This takes into account the specific characteristics of the instrument. If the measurement aid sheet is used, the dispersion in measurement is decreased and this may be useful when high accuracy is required from a few measurements.

It is recognized that there will be dispersion in measurements made on a textured sprayed finish, which is an uneven surface, even if the aid sheet is used. Therefore, it is necessary to avoid making measurements on a continuously uneven surface, and the instrument must be placed on a stable surface. Also, measurements should be made only after the measured surface has been tested for smoothness, because a projection angle of over 41° presents problems due to the geometry of this instrument.

3.2 The operator effect

3.2.1 Experience and number of measurements

The experiment used concrete specimens with cracks 0.65 and 0.25 mm wide, and a concrete wall with a 0.7 mm width crack as measured by a crack scale. An operator measured these cracks 5 – 30 times using the instrument, but without using the measurement aid sheet.

The effect of the number of measurements made by the operator is shown in Fig. 5.
Measurements made just 5 times have the largest dispersion, and subsequent measurements tended to become stable.

It is considered that the effect of the number of measurements made by an operator is negligible after the operator has gained practical experience by using the instrument more than 5 times in trial measurements.

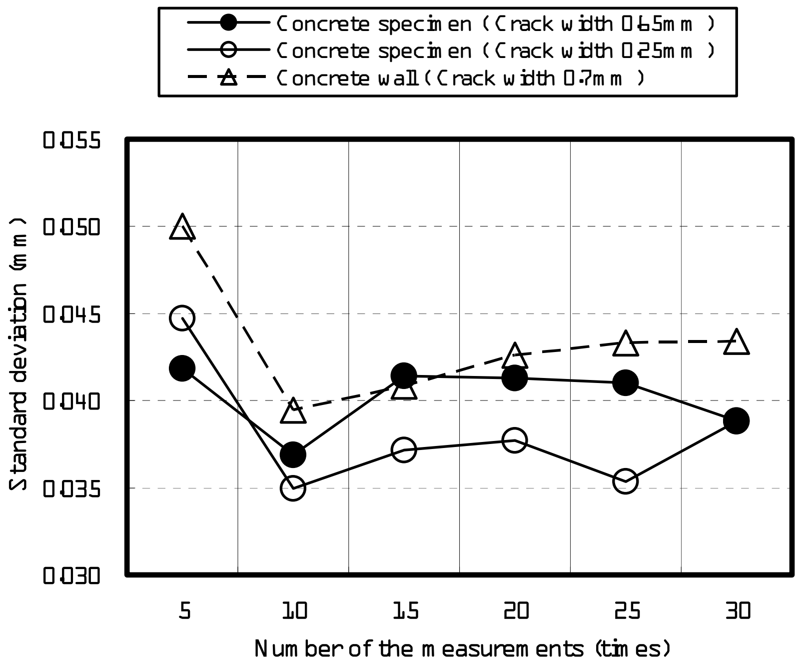

Fig. 5. The effect of operator experience on the measurement

3.2.2 Individual skill

The experiment used concrete specimens with cracks 0.4, 0.55, 0.9, and 0.95 mm wide as measured by a crack scale. Three operators with actual experience measuring crack widths made a mean of three measurements of these cracks using the instrument but without using the measurement aid sheet.

The result of the experiment is shown in Fig. 6. For each specific operator, the dispersion of measurement values and the difference from values measured with a crack scale are recognized, and the dispersion between operators is significant. However, the coefficient of variation is decreased for all three operators when the measurement aid sheet is used.

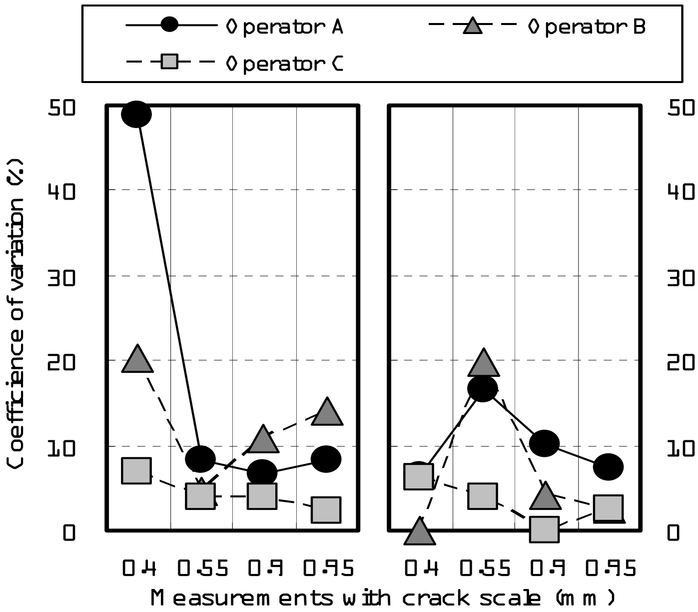

Fig. 6. The effect of the individual operator's skill on the measurement

3.2.3 The calibration tool

The targets were cracks 0.35, 0.6 and 1.2 mm wide as measured with a crack scale, which had been extracted at random from concrete structural elements. An operator with actual experience measuring crack width measured the cracks with the instrument, with and without using the calibration tool.

The results are shown in Fig. 7. The dispersion in measured values can be decreased at all crack widths by using the calibration tool.

It is considered that the usefulness of the calibration tool can be seen in the measurements made by the experienced operator, and also that it helps even an unskilled operator achieve consistent accuracy.

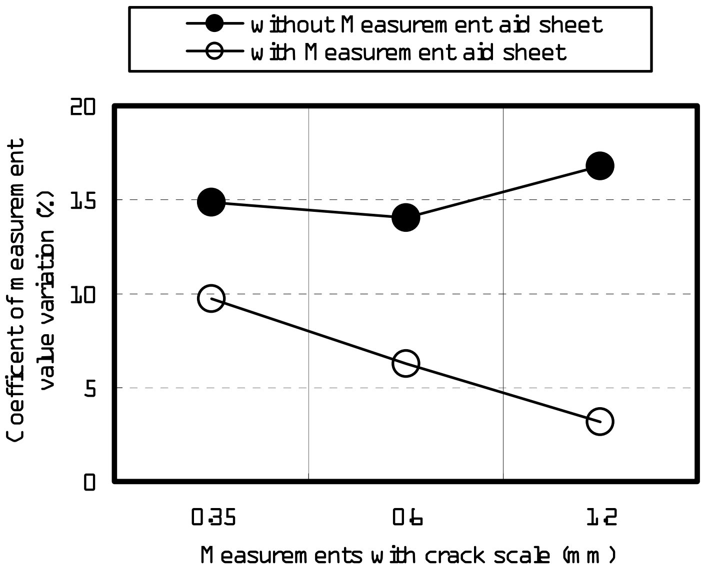

Fig. 7. The effect of the acid sheet on the measurement

3.3 The measurement for structures

It is considered that an uneven surface or lack of a sharp edge on the crack may affect the measurement results with this optical instrument. In this experiment the crack width was measured with both a crack scale and the instrument for two shapes of crack.

Experimental results are shown in Fig. 8.

It is judged that measurements made with the instrument do not differ from those measured with a crack scale, even with unevenness and surface level differences. On the other hand, the lack of an edge did have an effect on the results. It is difficult to measure the crack width exactly because the irradiation depth of the light varies. However, this characteristic may not be a problem unique to this instrument, because the crack scale showed a similar problem in measuring the true crack width.

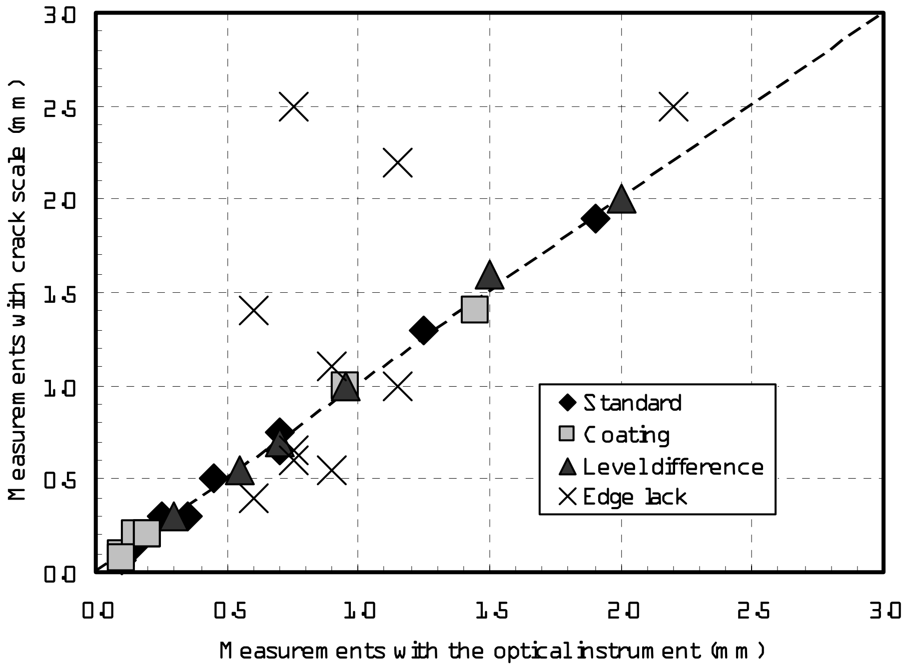

Fig. 8 The effect of the surface conditions on the measurement

3.4 Background Color

The detection error in the crack width tends to increase if the material is blue or green, because these are complementary to the red color of the LED light source, so they absorb the illumination, which decreases the contrast of the illuminated area.

In an extreme case, the background color simply appears as black as the crack, and it is impossible to measure the crack.

Also, in actual practice, if the surface is a dark color but the inner walls of the crack are nearly white, the instrument's error may be larger.

4 CONCLUSION

A new portable instrument has been developed to deal with the increasing problem of crack width measurement in existing buildings and structures. Its function is based on optical principles and it can measure crack width quantitatively on a structural surface.

This paper describes the experimental evaluation of the instrument, as it measures actual cracks. The results show the capabilities and limitations of the instrument.

5 ACKNOWLEDGMENTS

This study was carried out by a research committee on the Japan Society for Finishing Technology.
The authors would like to express their heartfelt thanks to other committee members.

6 REFERENCES

Japan Concrete Institute. 1987, *Guidelines for the Survey, Repair and Reinforcement of Concrete Cracks,* Tokyo.

Kondo, T., Motohashi, K., Nireki, T., & Ogasawara, K., 1998, 'Water Jet System for Pretreating Concrete Surface Before Plastering', Proc. CIB W70 Int. Symp. on Management, Maintenance & Modernization of Building Facilities, Singapore, November 1998, pp. 275–279.

Diagnostic tools and a new building defect registration system

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ABSTRACT

It is an important task of the building activities to maintain the condition of the buildings, to recognise, analyse the failures reducing, limiting the values, the suitability of the buildings, to know and apply the methods of preventing and repairing the defects. The aim of the general building diagnostics is to determine the various visible or instrumentally observable alterations, to qualify the damages from the suitability and personal safety (accidence) points of view. The successful diagnostic activity has an important role in the changes of the repair costs and the efficient elimination of the damages.

The research group of the Széchenyi István University developed a comprehensive diagnostic system which can be applied on the majority of the building constructions, including a uniform testing method and which is suitable for computer aided data recording and data processing. The diagnostic system is primarily based on a visual examination on the spot, its method is suitable for the examination of almost all important structures and structure changes of the buildings.

For using the system a so-called "morphological box" has been created, that contains the hierarchic system of some 2300 constructions, which is connected with the construction components' thesaurus appointed by the correct structure codes of these constructions' place in the hierarchy. The thesaurus was not only necessary because of the easy surveillance of the system, but to exclude the usage of structure-name synonyms in the interest of unified handling.

During the operation of the diagnostic system a large number of data – valuable for the professional practice – was collected and will be collected also in the future, the analysis of which data set is specially suitable for enriching the factual knowledge of durability and the practical application of the experiences later during the building maintenance and reconstruction work. The probation of the diagnostic system has successfully happened in the course of an examination action involving 60 buildings; its installation is in process at one of the biggest building holder-operator organisation in Hungary.

KEYWORDS

Building diagnostic, diagnostic procedure, diagnostic tools, morphological box, thesaurus of components

1. INTRODUCTION

It is important, that people in architecture science give a useful guide – especially concerning questions raised by new trends of the changing, transforming building activities and construction development – to the profession practice. Seeing that in the aspect of adequacy the appearance of new constructions and building materials always raises new problems to be solved, and the experts in practice busy with the daily tasks of the profession 'according to Möller [1945] cannot always pay enough attention to them'. The efficient diagnostic activity as it has been explained before plays a very important part in the formation of maintenance costs and elimination of damage. It has a just as important part in the preparation of a decision, as having a clear picture of the technological conditions of buildings or group of buildings can be of service at the preparation before making financial decisions of great significance.

2. DIAGNOSTIC SYSTEM

A faulty diagnosis can lead to incorrect decisions causing financial loss. The research group of the Széchenyi István University (Győr) worked out such a comprehensive diagnostic system (see Fig. 1.) which contains a common inspection method 'according to Molnárka [2000.] for the vast majority of constructional components (for traditional and actually used constructions in Hungary), and can be used for computer data registration and analysis.

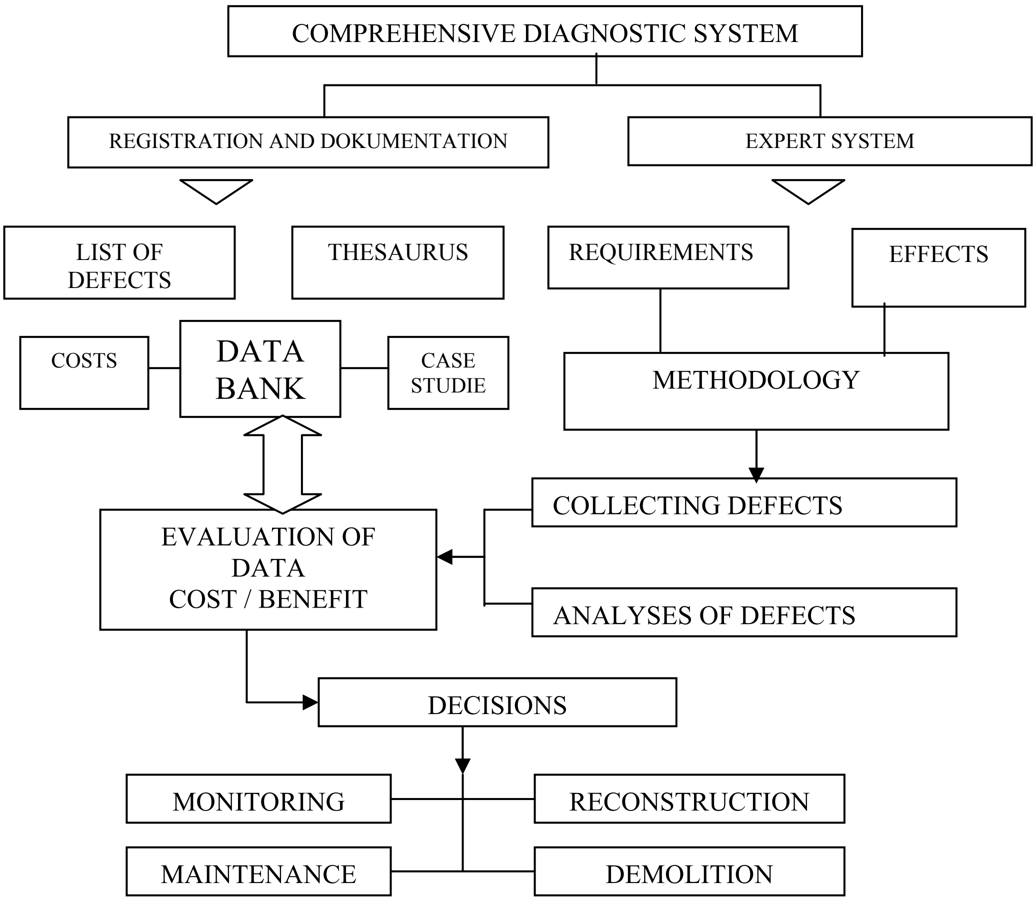

Figure 1.

The morphological box [Zwicky 1966] is connected with the construction components' thesaurus denoted by the correct structure codes of these constructions' place in the hierarchy. The theory of using morphological box for data registration in the process of building diagnostic [Koppány 1977] was published in Hungary seven years ago. The "matrix" construction of our morphological box fits to the methodology of the visual examination and to the hierarchy of the common building constructions in Hungary 'according to Koppány [2002] (see. Fig. 2.).

MAIN CONSTRUCTIONS	CONSTRUCTIONS AND THEIR SUPPLEMENTARY CONSTRUCTIONS			
CODE ⇒ A.	A.X	A.0X	A.00X	A.00X
1. FOUNDATIONS	1.1 Shallow foundations 1.2 Deep foundations		1.001 Insulations against ground-water	
2. VERTICAL LOAD BEARING CONSTRUCTIONS, NON LOAD BEARING CONSTRUCTIONS AND THEIR SUPPLEMENTARY STRUCTURES	2.1 Load bearing walls 2.2 Frames	2.01 Non load bearing walls 2.02 Partition walls 2.03 Doors and windows in internal walls	2.001 Wall-insulations 2.002 Plasters on facades 2.003 Plasters inside 2.004 Claddings inside	2.0001 Grates, balustrades, parapets
3. HORIZONTAL LOAD BEARING CONSTRUCTIONS, NON LOAD BEARING CONSTRUCTIONS AND THEIR SUPPLEMENTARY STRUCTURES	3.1 Ring beams 3.2 Arches, lintels 3.3 Floors, vaults	3.01 Suspended ceilings	3.001 Water-proofings to floors 3.002 Floorings	
4. STAIRCASES, BALCONIES, OPEN CORRIDORS, LOFTS AND THEIR SUPPLEMENTARY STRUCTURES	4.1 Staircases 4.2 Balconies, open corridors,lofts		4.001 Stair-tread covering 4.002 Balconies flooring 4.003 Open corridors flooring 4.004 Lofts flooring	4.0001 Balustrades, parapets, handrails to stairs 4.0002 Balustrades, parapets, handrails to balconies 4.0003 Balustrades, parapets, handrails to open corridors 4.0004 Balustrades, parapets, handrails to lofts
5. ROOFS, ROOF ACCESSORIES, CHIMNEIS, VENTILATION	5.1 Pitched roofs 5.2 Flat roofs 5.3 Roof superstructures 5.4 Chimneis 5.5 Air shafts 5.6 Vent pipes	5.01 Roofings	5.001 Water-proofing 5.002 Thermal insulation 5.003 Vapour-proofing 5.004 Flat roofs tiles and other functional layers 5.005 Roof edges and joining accessories	5.0001 Roof accessories
6. FACADES		6.01 Skirting board	6.001 Surfacings	6.0001 Grates, balustrades, parapets on the facades

Figure 2.

The main task is of our thesaurus (graph-version) to help the visual survey. It can be very useful to understand the hierarchy and the connections in the field of building constructions (see Fig. 3.). The thesaurus is not only necessary the easy surveillance of the system, but to exclude the usage of structure-name synonyms in the interest of unified handling. We have another tool too for the quick survey of the results of the visual examination, it is the hexagonal morphological box. The box shows the actual checked constructions or all constructions of the building. The various conditions of the building constructions can be marked with corresponding colours in the box-fields (see Fig. 4.).

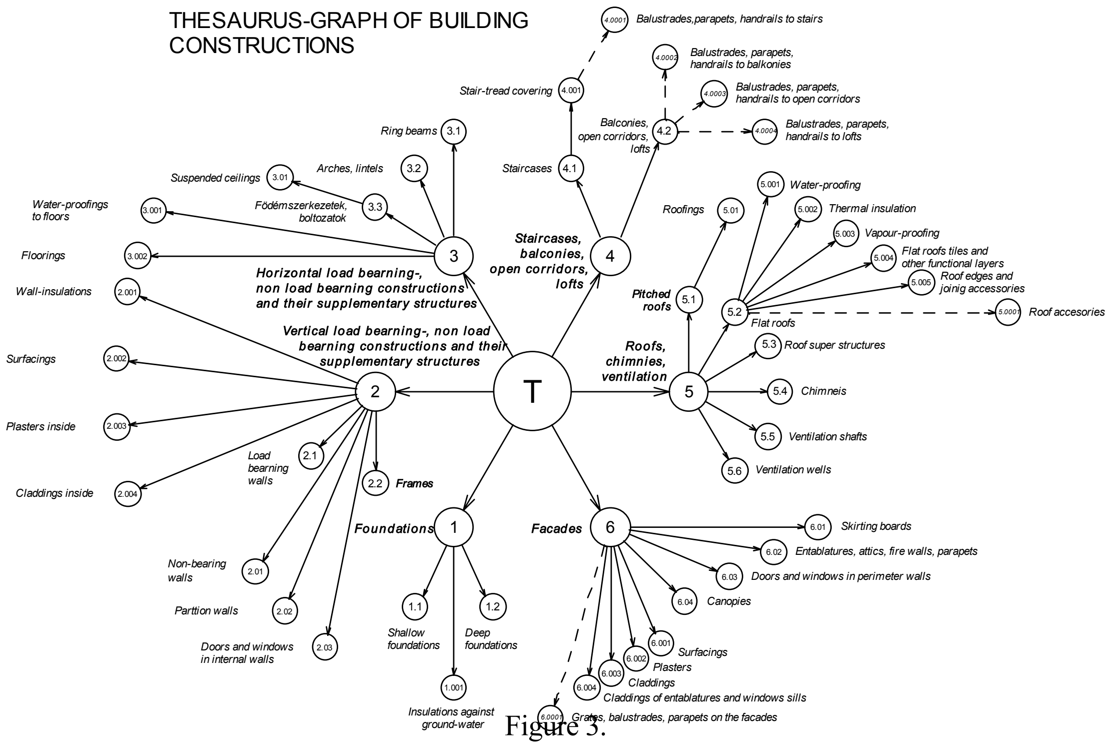

Figure 3.

Figure 4.

The main details of our diagnostic system are shown on Fig. 5. The diagnostic system has been successfully checked in the course of a course of an examination involving 60 buildings.

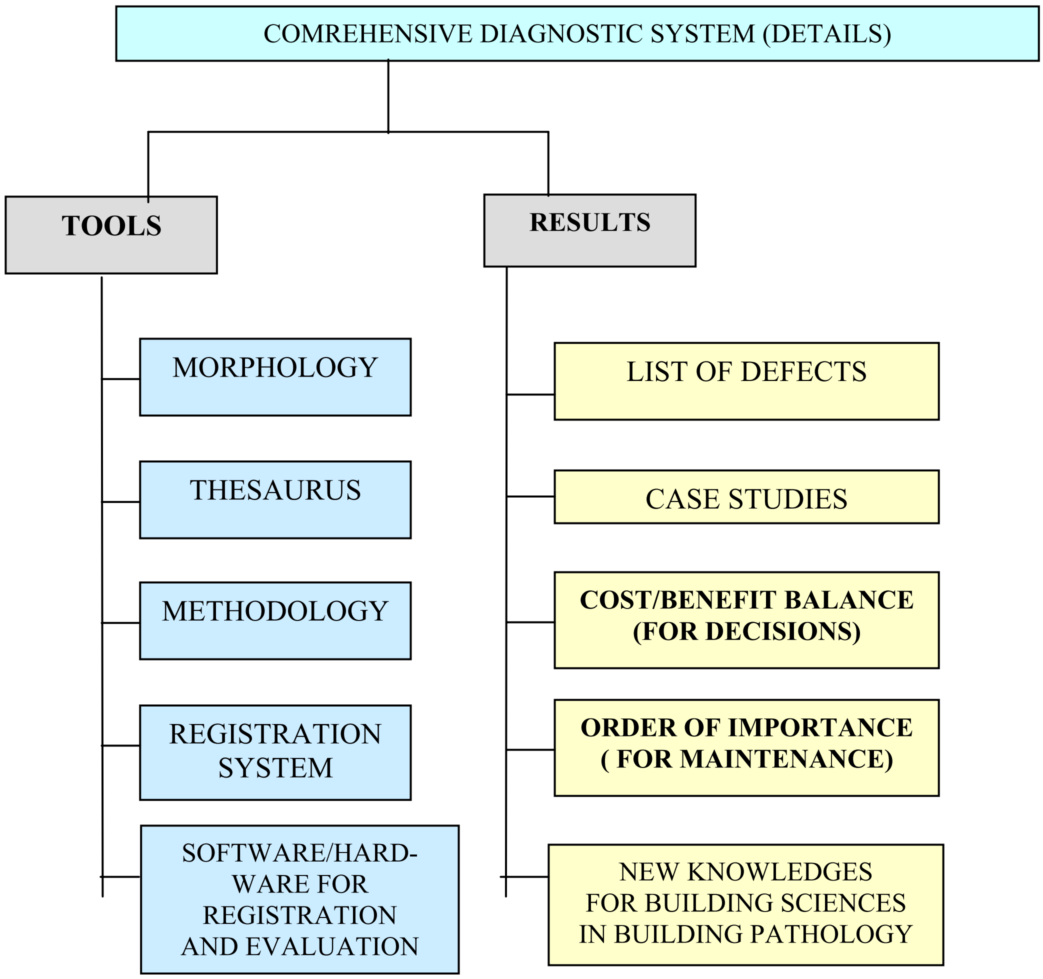

Figure 5.

3. EXPERIENCES OF FLAT ROOFS EXAMINATION

The majority of the defects forming on *the flat roof water-proofing and heat insulation* can be observed on the conventional insulated roof. From the point of view of the protection of substance and providing of the proper application of the building it is very important to explore and repair professionally the failures as soon as possible. Concerning the costs it is very important to explore the failures in the beginning stage because the failures which can be repaired with relatively low costs at the beginning can be repaired only with much more expenditures and together with many other structures after the extension of the damage. It should be noted for example, that the water proofing should be checked by experts at least yearly because several extended and expensive damages can be prevented with this care. The typical failures of the roof can be systematised according to various points of view. The analysis can be performed by the layers of the layer construction of the roof insulation and water-proofing or according to the contribution to the creation of the roof insulation (e.g. material manufacturing, planning, execution, operation), but it can be carried out according the so called weak points, details of the structural nodes.

Before introducing the diagnostic procedures and testing methods applied for the flat roof construction it is reasonable to determine some principles [Koppány & Graf 1985] in connection with the examinations as follows:

- the examination mustn't inhibit the proper use of the building;
- the examination should be quickly performable with easily usable tools;
- during the whole process of the examination the least possible damage can come out in the flat roof water-proofing;
- if the destructing examination is unavoidable it can cover the least possible area and the place of sampling should be immediately repairable (in a waterproof way).

The diagnostic work can consist of several phases - which are important from the end point of view. At first before the examination on the spot it is reasonable to inform on the basic data, structures and building conditions of the building to be examined. In case of old roof it is not always possible since the plans and other documents could get lost and they cannot be often reconstructed. In a significant part of the cases the structural character, layer construction, the used materials and the technologies should be identified during the examination on the spot.
During the visual examination the visual failures should be discovered then the analysis of the operation of the structure can lead to the determination of the more complex causes of the failures. During the visual examination the identification of the place of leak for large discontinuities (damages) on the water proofing of direct layer order has generally no difficulty. In case of quick examination the condition of the roof water-proofing is determined basically with visual examination, completed with a deteriorate free instrumental measurement if necessary (see Fig. 6.).

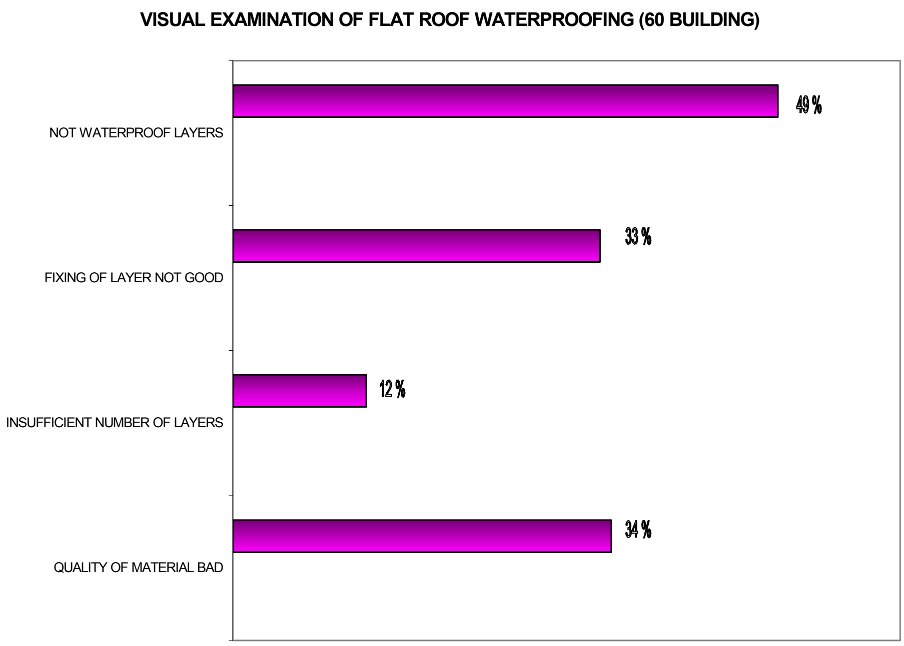

Figure 6.

For comprehensive examinations the procedure covers the all structural and complementary elements of the roof. The condition, load capacity, deformation of the bare floor should be examined, the building physical properties of the floor structure, etc. should be evaluated.

There are a lot of interesting data from the examination of 60 flat roof constructions (see Fig.7.) This figure shows the characteristics of frequency of the typical defects at the examined old flat roofs. The greatest number of defects were in the expansion joints area, nearly fifty percentage. The age of the examinnated roofs was on average 20 years. The roofs of the older industrial building had the worst condition, most of them were in bad repair.

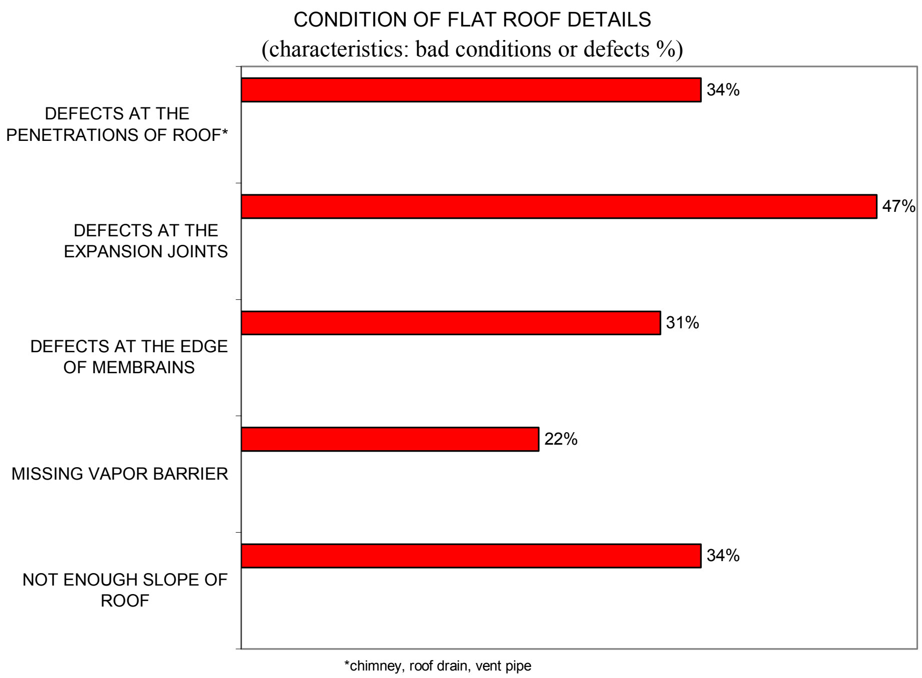

Figure 7.

So in order to maintain the condition, safe and durability of the flat roof water-proofing and insulation all significant factors should be examined even with a deterioration examination if necessary. It can happen that the diagnostics are started with a visual examination performed within the frame of a quick examination and to determine exactly the causes of the abnormalities, defects observed during the examination a complex examination is required.

4. **CONCLUSION**

In the paper is reported the development and structure of a field based survey methodology by the research group at the Széchenyi István University as practical diagnostic decision support tools. The new system was developed for the one of the biggest building holder-operator organisation in Hungary. The diagnostic system firstly based on a visual examination on the spot. At the beginning of the developing work an important requirement was the visual demonstration of the examinations' results. It was expected from the tools of system to be able to show the general conditions of the buildings or the condition of selected constructions. In the process of visual examination the experts have a big amount of data. For the effective handling and using the data the research group of the university created a registration subsystem with tools:

• morphological box for building constructions and
• thesaurus of building construction (connections with the morphological box).

The morphological box as a diagnostic tool wasn't used earlier in Hungary. The first probation of this tool has successfully happened and our client, the building holder-operator has installed this tool in his data registration subsystem.

Using of this tools the experts can survey the connections of the examined constructions and can use also a clear visual survey of the different durability of the existing constructions. Some details from the results of the visual examination of 60 different flat roofs can illustrate the efficiency the new subsystem.

It would be proper if the results of the constructional diagnostics, the experiences of the pathologic analyses make easier the work of experts using our registration subsystem. The practikal diagnostic decision support tool can help choose the appropriate correctiv maintenance procedures, it is once which remedy to the real causes of the failure.

REFERENCES:

Möller, K.(1945) Építési hibák és elkerülésük. (Building defects and prevention of defects), Egyetemi Nyomda, Budapest, pp.32

Molnárka, G.(2000) The methodology in visual examination in building pathology. Hungarian Electronic Journal of Science, 2000. (www.sze.hu) pp. 10

Zwicky, F. (1966) Entdecken, Erfinden, Forschen im Morphologischen Weltbild, Droemer/Knaur, München/Zürich pp. 174

Koppány, A. (1997) Épületszerkezetek morfológiai rendszere.(Morphological System of Building Constructions), Research Working Paper, p.40, SZIF, Győr

Koppány, A. (2002) Building Diagnostics. Construction and City Related Sustainability Indicators (CRISP). EU 5. Framework. CRISP Working Committee Conference, Budapest. ÉMI. p. 8 (summary on: http://crisp.cstb.fr /Database-Hungary)

Koppány, A. and Graf, H. (1985) Mängel an Flachdacheindeckungen und ihreUntersuchungs-methoden, Bauzeitung, Berlin, pp. 135-137

Industrialized Building Typologies in Public Residential Building and Durability of Materials and Components. Research on Some Important Interventions in Middle-South Italy.

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ABSTRACT

The spread of techniques of industrialization in building (great load-bearing panels systems, tunnels and banches-tables with plugging panels, steel structures and mixed steel-in situ concrete) in Italy was limited to high-densely populated urban areas where important interventions of residential building were made thanks to agreement and public aid. These techniques were also used to resolve emergencies such as the necessity of re-building the building estate after the 1980 earthquake in Campania and Basilicata.

This made it easier the application of the diagnostic method worked out by CIB W86 *Building Pathology Board*, because these interventions were realized in few big cities interested also by similar and not usual administrative procedures.

Thanks to a deep analysis of 14 interventions, for an amount of about 4.500 flats, it was noticed a structural lack of quality control processes both on the project and the realization, which is typical of the administrative instruments used for these works, especially the Italian systens of awarding of contract "*concessione di sola costruzione*" and "*appalto concorso*". This seems to be one of the main causes of quick decay and bad durability that characterizes the most part of these interventions, corroborating the negative experience of other European Countries, such as England.

By a specifically technical point of view, all interventions were studied considering aging state, quality performance, functional defects or anomalies, decay situations and more frequent pathologies, trying to find out causes and possible solutions by the analysis of single cases and the diagnosis of decay process.

New failure cases came out, which are worth of attention from specialists, in order to avoid their recurrence. It was also noticed that a correct project and realization can make durable and efficient as for performance also these building systems and neither prejudices and unconditioned enthusiasm are justified. In other words, it is unavoidable a deeper investigation in knowledge and data about the performance in time of this building systems.

KEYWORDS

Techniques of industrialization in building, functional defects or anomalies, decay, failure cases.

1 SURVEY FIELD

The research group coordinated by Prof. Arch. Filiberto Lembo is currently working in the Faculty of Engineering at the University of Basilicata – Potenza, and previously operated in the Faculty of Architecture of Rome – *La Sapienza*; in the last twenty years it has been working on the data collection of the pathologies and of the durability characteristics of the industrialized building systems of residential structures, by means of surveys, which were often repeated in a cyclical manner after some years, on the most relevant interventions of the middle-south of Italy (Lazio, Campania and Basilicata).

As a matter of fact, the following quarters were analyzed:

- Roma, *Corviale Nord e Sud*, Piano di Zona n. 61 - n. 1.080 flats of subsidized buildings - Building Company Ing. O. Manfredi S.p.A. - large bearing panels system model FINTECH-ITALCAMUS - yers1972-74 (planning), years 1976-1985 (carrying out);
- Roma, *Giardinetti*, Piano di Zona n. 27 - n. 252 flats of conventionated buildings - Building Company M. Odorisio - large bearing panels system model COSEDIN S.p.A. (Balency MBM) – years1975-76;
- Roma, *Casal dé Pazzi*, Piano di Zona n l0-11- n. 152 flats of conventionated buildings - Building Company CEVIP S.p.A., large bearing panels system model CEVIP S.p.A. - years 1984-85;
- Pozzuoli (Napoli), *Monterusciello* - Lot n. 6 - n. 216 flats of subsidized buildings after natural calamities (bradyseism of Pozzuoli 1983) - Holder: Association of Building Companies RDB S.p.A., Piacenza - ITACI S.p.A., Salerno - reinforced concrete "pillar-beam" system - prefabricated reinforced concrete pillars and floors made by "prédalles", staircases carrying out with running as wind-brace - stopping up with large prefabricated panels - years 1984-85 about;
- Pozzuoli (Napoli), *Monterusciello* - Lot n. 8 - n. 148 flats of subsidized buildings after natural calamities (bradyseism of Pozzuoli 1983) - Holder: Association of Building Companies Ingg. Carriero/Baldi S.p.A, Napoli - Mambrini S.p.A., Roma - Opere Pubbliche S.p.A., Roma - large bearing panels system model FORAP - years 1984-85 about;
- Pozzuoli (Napoli), *Monterusciello* - Lot n. 9 - n. 156 flats of subsidized buildings after natural calamities (bradyseism of Pozzuoli 1983) - Holder: Association of Building Companies GECOPRE S.p.A., Cagliari - SAFAB S.p.A , Roma - Pier della Francesca S.p.A., Torino - CO.RO.NA. S.r.l., Napoli - large bearing panels system - years 1984-85 about;
- Pozzuoli (Napoli), *Monterusciello* - Lot n. 11 - n. 154 flats of subsidized buildings after natural calamities (bradyseism of Pozzuoli 1983) Holder: Association of Building Companies FEAL S.p.A., Milano - SICOAP S.p.A., Napoli - steel "pillar-beam" system and floors made by "prédalles", staircases carrying out with slipform, running as wind-brace - stopping up with large prefabricated panels - years 1984-85 about;
- Pozzuoli (Napoli), *Monterusciello* -Lot n. 13 - n. 150 flats of subsidized buildings after natural calamities (bradyseism of Pozzuoli 1983) - Holder: Consorzio Impresa lng. Vito Fasano, Taranto - ZANUSSI S.p.A., Spilimbergo (PN) - reinforced concrete spatial cells made by ZANUSSI Edilizia Industrializzata S.p.A. – years 1984-85 about;
- Pomigliano d'Arco (Napoli) – *Comparto n. 7 – Settore residenziale n.1* – n. 128 flats of subsidized buildings after the 1980 earthquake – Holder: Società Italiana per Condotte d'Acqua S.p.A., with Società per il Risanamento di Napoli S.p.A., Impresa Pasquale Corsicato, IM.CO. Impresa Centrale di Costruzione S.p.A. – large bearing panels system model PEIKERT, years 1981 (planning), 1983 (approval of the project), 1986 (end of works);
- Pomigliano d'Arco (Napoli) – *Comparto n. 7 – Settore residenziale n.2, 3 e 4* – n. (127+147+60) = 334 flats of subsidised buildings after the 1980 earthquake – Holder: Società Italiana per Condotte d'Acqua S.p.A., with Società per il Risanamento di Napoli S.p.A., Impresa Pasquale Corsicato, IM.CO. Impresa Centrale di Costruzione S.p.A. – industrialized forms and e lofts made by "prédalles", years 1981 (planning), 1983 (approval of the project), 1986 (end of works);

- Atripalda (Avellino), *Alvanite* District - n. 303 flats of subsidized buildings after the 1980 earthquake - Holder: A.LO.S.A. S.p.A., Rorna - years 1982-91;
- Avellino, *some districts* - n. 1.023 flats of subsidized buildings after the 1980 earthquake - Holder: Association of Building Companies FEAL S.p.A. et al. - VOLANI Edilizia Industrializzata S.p.A. et al. – FEAL/VOLANI steel system, floors made by "prédalles" and staircase carrying out - (or, in same districts) tunnels and precast lining panels system –years 1981-86 until 1991;
- Potenza, Cocuzzo "*Serpentone*" District – n. 138 flats of subsidized buildings - tunnels and precast lining panels - Building Company: Ingg. Lino & Tito Del Favero, Trento - years 1976-80;
- Potenza, Cocuzzo "*Serpentone*" District – n. 138 flats of subsidized buildings - reinforced concrete beam-pillar system laid on site and REP mixed beams steel–reinforced concrete – Building Company: Geom. Giuseppe Padula - years 1975-84;
- Melfi (Potenza), houses for the dependents of the Nuova Officina Grandi Riparazioni of S.Nicola distric of Melfi of the "Ferrovie dello Stato" - n° 125 flats of subsidized buildings - *competitive contract* (i.e. "*appalto concorso*"); reinforced concrete spatial cells made by ZANUSSI Edilizia Industrializzata S.p.A.;
- Francavilla sul Sinni (Potenza), *Rione S. Elania* - n. 27 flats of subsidized buildings - *competitive contract* (i.e. "*appalto concorso*") of n. 180 accommodations into 5 towns - large bearing panel system made model SACEP S.B.S., years 1986-89.

As one can easily notice, these are wide interventions, partly funded by the State in order to solve high tension situations from the standpoint of buildings in the capital city or in other large urban areas, most of the times after natural disasters, the latter being the case of the bradyseism of Pozzuoli or of the earthquake of Campania, Basilicata and, partly, of Puglia in November 1980.

This is a homogeneous and representative sample of the so-called "*industrialized construction*" building technologies, which, apart from these special applications have never been used in Italy so far, especially in the south and in the middle of the Country.

As for the *industrialized constructions*, it is important to consider the fact that the specialized literature states the necessity of not mixing together the industrialization processes belonging to both the economical and cultural aspects of the society with its external display, such as the use of special machines or specific material, all of them being capable to bring significant improvements from the point of view of the production factors.

As a matter of fact, in a highly developed industrialized society, for the different productive occasions, at various levels and with different protagonists and balances of the production factors (amongst them, *time* is fundamental and it is considered as the main aspect for the process, project and product choices), interventions performed at the same time and in the same place by means of various building techniques at different rationalization levels do make sense [i.e. Lembo *et al.* 1971, and the *Introduzione* by F. Lembo 1978, pagg. XIV-XXVI].

In the last years survey methodologies were applied on these interventions: these techniques were arranged by the *W86 Building Pathology Commission* for the detection of the defects causes and for the forecast of the decay process, later defined in the *State of the Art Report* in 1993.
Before specifying the technical conclusions belonging to the investigations carried out, it is fundamental to consider some conclusions on the qualitative consequences deriving from procedural factors.

2 RELATION BETWEEN PROCESS QUALITY AND PRODUCT QUALITY

The survey is mainly on such buildings realized after special tender procedures, as the case of the *only building license* (i.e. "*concessione di sola costruzione*") and the *competitive contract* (i.e. "*appalto concorso*"): these procedures are nowadays anachronistic after the approval of the new Italian regulations on public works. Furthermore, they have currently replaced by the *integrated contract* (i.e. "*appalto integrato*"). For this reason, we believe that there is still the risk of a lack of controls, which are typical of the examples considered above.

When these procedures are utilized (they are generally justified by the specific technical complexity of the building techniques which are to be used, with respect to short lengths of time), the building contractor proposes the executive design and the qualitative features which are to be respected.

In these situation, the builder proposes also the norms, and the judge and the part which is going to be judged are the same person; the control system, always prepared on the hypothesis that both the project and the roles are to be fixed before the constructor choice, being, as a matter of fact, jeopardized and completely inadequate.

In particular, some of these procedures, as for the case of *licenses* (i.e. "*concessioni*"), the executor, apart from arranging the design, nominates the works manager and gives him the money, every sort of objection being in this way cancelled. The extreme consequence of this process is that, in some cases, certain buildings, at the time of their completion, are already "not habitable", with plenty of severe pathologies (Pomigliano d'Arco (Napoli), *Compartment n° 7*).

This is the consequence of a particular trend, also seen in the past years in other European countries, which tended to realized low cost lodgings by means of different kinds of inducements – as it happened in Great Britain -, through industrialized building systems guaranteed by the just created Government structures, in order to step over the classical validation processes [i.e. Lembo 1988].

So that while in Italy the *Societies of Validation* need by designers the deepest attention on less important licenses, the important ones enjoy special conditions which produce a decisive reduction in building quality or in its durability. This can be noticed in the public residential building field, where there have always been more silent and consolidated controls: many good enterprises failed because of the high quality standards required by commitments Institutes on public residential works and because of not profitable prices. It is even worst as for "richer" fields, where controls are much more difficult because of the "nature" of works, such as motorways and railways or the realizing bridge on the Messina Strict.

Rationally speaking, if the Contract Special Specifications are descriptive and are drawn by the contractor, they will not include all the possible technical solutions, while, if compiled by the enterprise, they will describe only the solutions adopted, which might also be incorrect or not good for the sake of the durability of the structure.

If the Contract Special Specifications are performance-based, they will be very demanding from the scientific point of view, for both the formulation and the management: they need laboratory and on site trials which cannot normally be performed by the contractor, at least in Italy, as also the University laboratories have troubles in finding the apparatus useful for the environmental safety trials. The sole attempt from this viewpoint was made by the Residential Office of Emilia Romagna, which produced tons of documents and technical specifications, no matter the boycotting action of the enterprises.

Some examples are going to simplify with the evidence and strength of historical events the importance of the problems related to durability caused by the procedures of licenses that are, as the International Labour Organization experts say**,** the *grader* that adjusts the soil level (that they define "the methods tweaking"), before the filing with the *spoon* (that they define "the time study").

2.1 Roma, *Corviale*

This is the case of a twelve floors building with three parts, the overall length being approximately one kilometer; it contains 1080 lodgings of subsidized construction, shops, garages, cellars and similar ambients.

The very first mistake was made at the time of conception: the scale of intervention was typical of a technical solution with large bearing panels on a laid on site ground floor with industrialized formworks, but the design planning out did not consider longitudinal cross bracing walls, because the structural designer (Riccardo Morandi, famous for his experience on bridges) considered a purpose made solution only based on laid on site walls with slipforms and precast floors, so as to please the architectural designers (coordinated by Mauro Fiorentino) who wanted a variable section for each floor, with a free intermediate floor.

It was a competitive contract, with the consequence that the enterprise was allowed to consider alternative technical solutions for the development of the original project: at the very end the building was realized as it should have been, with large bearing panels on a laid on site ground floor with industrialized formworks, and with elastic cross bracing keys, the longitudinal walls being absent and the openings of the various floors being not lined up.

A large quantity of steel bars was adopted with a very small cover, because of the reduced thickness (16 cm) of the reinforced concrete sheets. As a matter of fact, the damaging effects of carbonation were not considered at all, thanks to the structural Standards which allow the designer to consider very small dimensions for the cover.

And you have to add incredible inattentions: external one layer reinforced concrete stopping up panels, insulated from inside, joined "*head to head*", whose impermeability is due only to the sealing and the only possible periodical maintenance requires a 35 meters impalement on a inclined plane (the garage covering). The same panels on the façade - 6,00 meters length, 1,50 meters high and about 10 centimeters thick - are sustained by two omega shape black steel rod solded to a plate on the floor in the pier for installations and two weldings of black steel plates corresponding to the transversal walls, covered with a 15 millimeters mortar of cement: they won't take long time before moving away. The same panels have a border toward the internal side - under the window sill - to hold the window frame, which seems to be able to produce a remarkable thermal bridge and to drip on the inner lining, realized with a material as much hydrophilic as possible such as gypsum plastering lath bricks and expanded clay. The same problem about thermal bridges is true also for internal loggias, galleries and *cavaediums*. It seems that the building knowledge stopped respectfully on the boundary of the industrialized building technique and that neither literature or rules – even if produced by mutual consent and by foreigners Bureaus - have been able to predict the performance in time of building solutions [i.e. Lembo 1978].

2.2 Pozzuoli, *Monterusciello*

It is a program to realize n.4.000 flats (about 20.000 rooms) with all facilities, after the bradyseism of the 1983, to built a new city organized by the Faculty of Architecture of the University of Naples. The cost of all is 104 Mld in Italian lires (about 52.162.147 euro). The *announcement of competition* has been signed by the Minister of Civil Protection, On. Vincenzo Scotti on the 7 November 1983.

The announcement of competition declares that:
"4) Structure of high – Stopping up – Partitions"
"The Building Company that participates to this competition they will use the proper compatible system of prefabrication related to the horizontal and vertical structures and to the external stopping up and interior partitions. No plasterboard."
"5) Covers"
"The covers, the flat covers, they will be waterproof with sheathe bitumen on the top of an adequate insulation. The floors are boarded with squared tiling of cement. The discharge of the meteoric water

it will be trough roses gully grating and tubs made of heavy pvc. The flat roof will be paved with small squared of cement leaned on support of pvc."
"6) Thermic and acoustic insulations"
"The insulation it will be conformed to the law 30 April 1976, n.373. The partitions between different flats they will be isolated acoustically with rock wool or equivalent materials."

It is quite clear that the constructive knowledgement has been "stopped out of the door" because the Minister based himself on a document most generalized and has given all the concessions of the project, the realization and the direction of the building of the 150 block of flats to the Consortium of Building Companies, that realized buildings free of any restrictions and any controls during the works.

In the major part of the construction with large prefabricated panels, they have been used panel with one layer multitubolar, interiorly isolated with light insulating panel and inner lining in plaster. That produces pathologies of condensation near the thermic dispersions, and vertical spreaded cracks at the joint of the panels, due of the thermal shock summer-winter of whom the structures are subjected.

In the same way the Buildings Companies have modified the project, placing insulations from outside made of thin plaster on isolation, and in this case the pathologies are completely absent.

The bearing panel a "sandwich solidale" they have had a wide spread. The very poor thermal qualities they have been disappeared in many European countries.

Here it shows the wide presence of joints "*head to head*" between the prefabricated panels – about 50% of the built up (all the rest is almost the type "*working for frame*" with few examples of types "*with guides and cams*"), with very low durability, added the scarce attention on the positioning of the panel.

Also has been verified the lack of the reinforcement scarcely covered of the bearing and the stopping up panels, and the prefabricated lofts, as well "staircase wall" carried out.
Little squares of the joints or section of black steel, used by parts of connection, quickly corrode because of differential aeration.
The cuts of capillarity on the ground floor arouses enormous upstarts of capillarity. All that can cause inhabitability of the rooms.

Quite often, the covers most of the time are wrong, like a "*hot roof*", but without vapor barrier under the insulation and with the sheath on the top of the isolation. All that produce condensation and moulds on the intrados of the lofts.

The bad conception and the built up of the waterworks causes frequently losses, which in the prefabricated structures are very important.

That's way, the "liberality" with whom has been carried out the work has had heavy effects on the quality and on the durability of the buildings, that's way the buildings are not kept in good conditions of preservation.

This bad performance is indubitably due to the <u>form of license</u> used, the *only building license* (i.e. "appalto di sola costruzione") and to the lacks in the <u>terms of contracts</u>, which are descriptive and proposed by the holder at the same time that the economical offer. But it is difficult that this could not be due also to the fact that the projects and the realizing processes were not evaluated by independent and competent organisms.

3 CONCLUSIONS

The same talk just done for the Constructive Program of Pozzuoli – *Monterusciello* it can be repeated for the one related to the 20.000 apartments, to be build it up in the area of Naples and his hinterland, to fulfill the requirements of the people followed up the earthquake of 1980, included the attendance of *Pomigliano d'Arco*, and the one foreseen for the boroughs affected by the same earthquake in Campania and in Basilicata, of which belongs the attendance of *Avellino*, which is the most.

Here too the institute of the "*only building license*" has brought out many faults, giving out the planning, the carrying out and the work under the supervision of the holders, and bringing about strong inefficiencies submitted to control. Except a few exceptions, the attendances suffer of the same pathologies and of the same problems of durability, like entire generations of engineers has failed his task.

But such conclusion it could be ungenerous at the same time led astray.

It could be very important to have a deeper university preparation or PhD of engineers about the characteristics and powerful pathologies of the industrialized building: sometimes, reading the documents, the inspection reports, the ratification and approval opinions, it looks like the drawers they haven't got the master's authority of the object, even operating on the high standards; certainly, it could be difficult to expect that *cavaedium* between the floors of 25 x 25 cm, made up by a panel like 'U' made of reinforced concrete of 7 cm of thickness, in which go trough the pipers of the heating plant and waterworks, it could be the place of a such air-drift, to build it up the condensation inside (Roma, *Giardinetti*). Furthermore it could be difficult to expect that in the structure made of steel like "pillar and beam", treated only with anti-rust and synthetic paint, with lofts made of reinforced concrete made by *prédalle*, the parts more corroded after 18 years are not from the attacks of the ground of the pillar HEB of the *pilotis* floor beaten from the rain, but the beams on the intrados of the porches or the loggias, subjected on differential aeration because they are only partially incorporated in loft made of reinforced concrete, or attached by the dripping water from a bathroom (Avellino, *Quartiere Valle*). This factors, attached to the *project* and to the *product* , can turn out like "spoons", in the presence of the factors of different weight and importance, which they work like the "grader" to level the ground.

And this one are the ones linked to the *process*, and particularly to the terms of contract, when the contracts are done by public money.

The history of the industrialization of the building in Italy, delaminated trough the study of the realizations, it will show that has been *etero*-directed, has not been used to gain high levels of productivity or performances, or short times for the realization of buildings, as for many years they have told as.

It has been used to justify the concentration of the contracts and, then, the choice of the high qualified holders out of the local Building Companies.

To gain that, the contracts they must be profitable and desirables, not only of big amount and concentrates, but runned in a very peaceful way and with pleasure of choice.

Now we understand, why so many mistakes of projects, of idea, of carrying out, of verification, they are all concentrated in these realizations.

In a conference on durability of building materials and components is important to call out between the causes of the mistakes and pathologies, also the one not completely technical, when it proves that their importance could be fundamental.

4 REFERENCES

LEMBO, F. *et al.* 1971, *Industrializzazione dell' edilizia - approccio alla struttura sistemica del processo edilizio,* Roma, Istituto di Tecnologia dell' Architettura dell' Università di Roma *"La Sapienza".*

LEMBO, F. (edited by), LUGEZ, J. 1978, *La prefabbricazione a pannelli nell'edilizia residenziale,* con prefazione di C. Blachere, Bologna, Ed. C.E.L.I.

LEMBO, F. 1988, 'Qualità e conservazione nel tempo delle costruzioni industrializzate - l'esperienza inglese', in *Recuperare*, [gennaio-febbraio 1988], pp. 91-99.

LEMBO, F. & MARINO, F.P.R. 2002, *Il comportamento nel tempo degli edifici – Cause di degrado e soluzioni progettuali dei sistemi edilizi 'tradizionali' ed 'industrializzati'. Casi di studio*, EPC Libri, Roma.

Building Typologies of Public Residential Building and Durability of Materials and Components. Research on 12.500 Residential Flats of E.P.E.R. of Potenza

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ABSTRACT

The E.P.E.R. (Provincial Bureau for Residential Building of Province of Potenza) realized, during the last '90s, a 12.500 flat real estate. This forms an homogeneous stock, which was analyzed according to the diagnostic method worked out by CIB W86 Building Pathology Board.

It was noticed five phases in realization, characterized by building typologies differing in dimension, distribution, materials and building systems. For each phase all buildings were studied considering aging state, quality performance, functional defects or anomalies, decay situations and more frequent pathologies, and it were defined causes and possible solutions thanks to the analysis of single cases and diagnosis of the decay process.

In this way it was found out an exact correlation between building typologies and durability, characterizing the recurring defects for each building typology, linking the anomalies to causes of bad functioning and defects in building, to defects in designing and techniques of realization or in the maintenance actions.

This study makes it possible the realization of projects of maintenance and/or validated retrofitting interventions that could avoid the repetition of errors made in the past, setting new and more precise "rules of art".

KEYWORDS

Database on Italian building systems, building typologies and defects, decay situations, pathologies.

1 INTRODUCTION

This study collects and analyzes the whole stock of interventions that the E.P.E.R. (Provincial Bureau for Residential Building of Province of Potenza, i.e. the old Istituto Autonomo per le Case Popolari) realized during the last century in the Province of Potenza, with the aim of analyzing its contents according a typological and technological point of view, considering the using and aging state, the performance quality, functional anomalies or defects, situations of decay and recurrent pathologies.

Within a more complex research that, thanks to an organic analysis of the E.P.E.R. flat real estate, had the aim of realizing a sort of reliable cataloguing of pathologies and appropriate maintenance techniques in specific cases, it has been adopted a procedure appropriate to give support to the decisions made within the cataloguing process, based on the analysis of single cases and the diagnosis of the decay process.

This system is based on the examination of many data and base knowledges that makes it possible to project interventions of maintenance and retaining of building. All this linking the anomalies to causes of bad functioning and defects in building, and to defects in techniques of realization or in the maintenance actions.

The main problem that the E.P.E.R. has to face at the moment is managing the interventions of maintenance and regeneration of a large number of buildings. Moreover, this is complicated by the variety in technologies and typologies of the more than 12.500 flats built in about 90 years in 101 municipalities of the Province of Potenza.

2 THE E.P.E.R. IN THE PROVINCE OF POTENZA

A first step in the research was dedicated to acquire detailed information on the flat real estate object of the analysis, to quantify buildings and flats, to define exactly their location and localization and to identify them according typologies and technologies of building systems.
Moreover, this first phase of the research made it possible to acquire information on the development of typological and distributive schemes of the flats that, during the 90 year life of the Institute, could be classified in the following phases:

<u>First phase: 1906-1915</u>. The Rule that put into effect the Luzzatti Act of 1904, defined specifically the standards that for half a century identified the typology of *"economical popular"* house, fixing minimum standards of cubature, lighting, ventilation, maximum number of floors and building standards for floors, roofs and sanitary.
The distributive characteristics of internal spaces are defined by the application of this Rule at the minimum level as possible, because it was considered especially the economical aspect.
They were considered mainly the following aspects: the entry to the flat was directly from the stairwell in the kitchen and through this to the others rooms; the rooms are quite large because many people could live in the same room; the kitchen is very big too because all the day-time activities take place here, it was also the sole heated room with a big fireplace and it had a "*secchiaio*" (bucket spot) useful to many uses. There were not, or were quite small, a corridor, self-contained room, entries, to increase the useful area of rooms in equal condition of covered areas (i.e. of costs). If there are corridors, the medium habitable area for flat is about 1.40 square metres, the sanitary is outside the flat, sometimes common to more that one flat and has a brickwork chamberpot; the medium useful area of habitable rooms (kitchen included) is about 17.00 square metres, and the utilities one (sanitary, fireplace, *secchiaio*) is about 5.00 square metres.

<u>Second phase: 1919-1934</u>. In this second phase the flat, considered in a first time only according an economical aspect, becomes more rational and comfortable and makes it possible to guarantee better living solutions. It tends to make rooms more independent introducing the entry, also in order to make independent the kitchen; the sanitary, even if smaller, is put inside the flat; the medium useful area of

habitable rooms is about 17.50 square metres, the utilities one (sanitary, fireplace, *secchiaio*) is about 6.50 square metres, the entry and self-contained room are about 6.70 square metres.

Third phase: 1934-1940. The main typologies of this phase consist in so called "*popolari e popolarissime*" flats and represent the moment of withdrawal in the typological evolution of flats, because it comes back the concept of cheapness. The sole new element consists in the fact that both typologies introduce for the first time a "small kitchen" in alcove for the mere function of cooking foods.
This is separated by a partition wall from the living room, where the rooms come back to appear because the entry does not make them independent.
The differences between the two types depends essentially on dimensions:
a) Popolarissime: the useful area tends to the minimum, in fact it has been reduced, thanks to special legislative measures, beyond the limits of the minimum standard considered by the 1908 Act, that prescribes that popular flats have to respect local building rules and, however, the Sanitarian Act of 1907. The useful medium area of rooms is about 12.00 square metres, the utilities one (sanitary and small kitchen) is about 3.25 square metres, and the entry is about 1.90 square metres.
b) Popolari: the useful medium area of rooms is about 16.45 square metres, the utilities one (sanitary and small kitchen) is about 6.80 square metres , the entry is about 3.70 square metres. Another characterizing element of these typologies is the fact that they combined for the first time the sanitary and the small kitchen with a sole technological *cavaedium* for discharges, chimney, etc.

Fourth phase: 1945-1985. In these forty years the building typologies, even if different because of the Acts succeeding in time, have homogeneous characteristics. In fact, in the most part of cases, each flat has an entry; the kitchen, from a big one in the first phase or the small one in alcove, is better dimensioned according to the specific use and the living room acquires more importance; the bedroom zone is separated from the living one; the useful medium area is about 14.00 square metres, utilities (bathroom and kitchen) are about 13.00 square metres, entry and self-contained room are about 7.00 square metres.

Fifth phase: after the 1985. Since 1985 the improvement of habitable and technological quality of flats is combined with the general concept that all flats have to be provided with spaces that, as for number and typologies, can respond to the necessities of families which are destined to.
The new idea is that all flats have to be divided in order to assure a sufficient level of autonomy to adults or older people living with the head of the family couple and, generally speaking, it is showed a new attention for "*special users*" which flats are destined to.
For this reason, it has to be provided in medium (4 beds) and big flats, a space independent from the other parts of the flat with a sanitary dedicated.
The deriving distributive scheme interprets in a modern key the first schemes of 1906.
According to architectonical typology, there are building:
isolated: low residential typologies that lodge few families and are collocated in a town planning scheme with a low building density. The building, whose internal parts depends on a single stairwell, is isolated on each side;
arrayed: linked single residential building typologies, usually low, repeating identically and articulated in order to realize a building continuity. Moreover, this organization makes it possible, with an adequate orientation, an homogeneous exposition to the sun;
in tower shape: the isolated building has had a vertical development maintaining a moderate occupation of soil;
in line: the building is developed according the favourite direction. Among variable elements that characterizes this typology there are the number and the collocation of vertical ways in relation to the number of flats served or existing on each floor;
duplex: this architectonical typology presents a flat located on two levels;
in gallery: a building that presents a structure overhanging from the external wall edged by a baluster or a railing, being an accessible area that turns around the building or a part of it both inside and outside.

The analysis of the building methods used for the structural system makes it possible to find out the following building technologies:

- masonry;
- evolved-traditional: mixed procedure using reinforced concrete and bricks and structures with plan elements (bearing walls and/or large panel and lofts carrying out);
- traditional in reinforced concrete with structures with linear elements (pillars and beams);
- processes based on industrialized techniques (tunnel);
- processes based on the use of prefabricated elements.

The Figure 1 shows the relations between building technologies and building typologies, referring to the period of building.

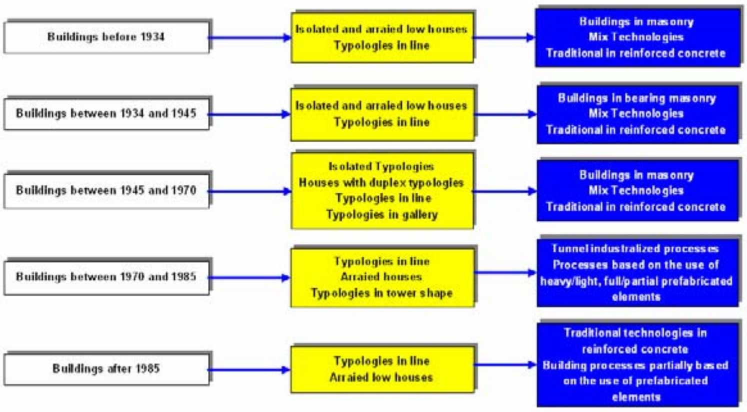

Figure 1. Relations between building technologies and building typologies.

2.1 The building stock in the Province of Potenza

According to a disaggregate analysis of the data about the research on the whole intervention of E.P.E.R. in the Province of Potenza, it can be noticed that the public intervention is massively present in the chief town where represents, with 307 buildings, 5.294 flats and 623 premises, about the 30% of the whole complex of buildings realized. There are eight areas of intervention on the municipal area of Potenza: Centro District, Cocuzzo District, Francioso District, Libertà District, Lucania District, Murate-Montereale District, Risorgimento District, S. Maria District; the 89,58% of the buildings were realized since the second after-war period until nowadays and, in particular, the 72,96% between the 1945 and the 1970.

By an architectonical point of view, the main typology is the classical parallelepiped in line (85,34%), with huge dimension building experiences and an high habitable density, such as for Cocuzzo District's building, in Tirreno Street, called "*Serpentone*", made of four in line blocks of 162, 138, 102, 133 flats. The buildings realized according to other typologies are less numerous: there are 18 buildings in array (5,86%), built between the 1956 and the 1972; there are 13 "tower" buildings (4,24%), among which, as for the building typology, there are the five buildings of Ionio Street called "*Torrette*"; there are 14 isolated buildings (4,56%), concentrated in Lucania District with two interventions, the more recent of which is the Nitti Street one dating to 1968-1974, the older one is the Appia Street one dating to 1929. And these buildings of Appia Street, with other eight built in Crispi Street, in Centro District, between the 1924 and the 1928, represent, also for building technology, the sole experience in the whole Province of totally masonry building. In the whole, if you do not consider the three blocks of "Serpentone", realized with "tunnel" techniques (0,98% of the whole intervention in Potenza), the 71,34% of the 307 buildings is made with reinforced concrete and the 22,8% with the traditional mixed system. There aren't "gallery" or duplex typologies, neither, as for building technique, completely prefabricated buildings. They are worth mentioning some interventions that have the sole façade prefabricated: two of "*Serpentone*" block, a part of the building called "*Serpentino*", and also in Ionio Street in Cocuzzo District, two buildings in Adriatico Street.

The E.P.E.R. intervention consisted in building, maintenance and regeneration in 88 of 101 municipalities of the Province of Potenza. In the municipalities of Castelnuovo Lucano, Fardella and Teana it is realizing interventions for 14 flats each one.
On the whole, the E.P.E.R. realized 917 buildings for a total of 7.268 flats and 1.698 premises. Even if in the municipality of Potenza it is confirmed the tendency to adopt for the building the in line typology (581 buildings corresponding to the 63,36% of the whole built in the Province), there are represented also other recurrent architectonical typologies. In fact, quite in all municipalities it is widely represented the "in array" typology, with a complex of 116 buildings (12,65%) and duplex buildings (127 buildings corresponding to the 13,85% of the whole) in 27 of 88 Municipalities, among which they are worth mentioning for the quantitative consistency of interventions in the specific architectonical type, the Municipality of Avigliano, Bella, Filiano, Oppido, Palazzo, Muro Lucano. There is a quite sufficient number of isolated buildings (84 corresponding to the 9,16%); there are tower type ones (6 buildings corresponding to the 0,65% of the whole) in Rionero and Melfi; in the municipality of Picerno (1 building) and Venosa (2 buildings) there are also the sole examples of a "gallery" building.

As for building technique, the 98,69% of interventions is made with reinforced concrete (53,87%) and with mixed system (44,82%); the sole example of "tunnel" industrialized process is in the municipality of Rapolla, and there are 11 buildings, corresponding to the 1,20% of he whole, realized with completely prefabricated systems, in the municipalities of Chiaromonte, Francavilla, Picerno, Rapolla, Rionero, Lagonegro, Lauria. The results of the analysis of the building data of the 917 buildings are particularly worth of notice: none of them has been built before the second after war period and almost the 40% had been built between the 1970 and nowadays.

As for the aggregate analysis of the data related to the E.P.E.R. flat real estate, we can observe that the public real estate, according to a national trend, is concentrated in the chief town, with an incidence a bit higher than 25%. The whole intervention is noticeable: 1.224 buildings, 12.562 flats (among which 6.365 are rented), 2.321 premises (among which 979 are rented), 1 sports ground (in Lagonegro), 2 social centres given in concession to the Caritas (in Melfi and Lagonegro).

The 68,87% of the buildings are "in line", while the "in flocks", duplex and isolated buildings are about the 10%; in Venosa and Picerno there are the unique three "gallery" buildings; in Potenza, Rionero and Melfi there are 19 "tower" buildings. With the sole exceptions mentioned above of tunnel industrialized building processes (in Potenza and Rapolla), almost the 58,24% of the buildings completely prefabricated realized in some municipalities of the Province of Potenza (11 prefabricated buildings corresponding to the 0,90% of the whole) and 15 masonry buildings in Potenza are made in reinforced concrete, and the other 39,30% are made with a mixed system.

Almost the whole intervention has been realized between the 1945 and the 1985; not much more than the 2% was built between the first and the second after-war period. Finally, the 7,68% of the buildings was built after the 1985 and the 33,33% after the 1970, which partially explains their generally good level in maintenance.

3 RELATIONSHIPS BETWEEN TECHNOLOGICAL SOLUTIONS AND RECURRENT PATHOLOGIES - FREQUENT PROJECTUAL AND BUILDING MISTAKES

The analysis about the typical pathologies of the E.P.E.R. flat real estate of Potenza let us to find out the main building pathologies and we collected forms about "*critical points*" of the buildings.

We referred for the identification of single pathologies and their causes to two sources:
- for the traditional techniques to the French Unified Technical Documents (U.T.D.) about the main categories of buildings;

- for non traditional techniques to the Avis Téchniques, realized by C.S.T.B. special commissions.

Through this analysis we could realize a data bank that is the starting point for the realization of a computerized database comprising all data about the observed defects.

After a disaggregate analysis of pathologies, splitting up each building in sub-systems and classifying the pathologies found out in each part of the building, the defects were grouped mainly in: *a.* leakings of water from coverings; *b.* leakings of water from walls; *c.* separation of the floors; *d.* separation of lining; *e.* lack of impermeabilization; *f.* thermic dispersions (thermal bridges); *g.* acoustic dispersions (acoustic bridges); *h.* moulds and stains; *i.* diffuse pathologies caused by condensation; *l.* pathologies caused by installations and discharges; *m.* pathologies caused by window and door frames; *n.* cracks.

These classes of defects were aggregated according:
1. the subjective importance: from leakings of water from coverings (15,61%) and walls (10,29%), to moulds and stains (14,02%) to thermal bridges not repaired (12,15%), to lack of impermeabilizations (10,94%), to separation of the floors (8,54%) and lining (6,95%), to cracks (6,38%), and finally to defects of window and door frames (4,80%) and of installations (especially heating plant and waterworks) and discharges (5,25%);
2. the cost of regeneration interventions: the more expensive ones deriving from the problems caused by leakings of water from coverings (35,29%), separation of the floors (17,65%), moulds (17,64%), separation of lining (11,76%), leakings of water from walls (10,39%), installations and discharges (8,83%), cracks (2,95%);
3. the relationship between different building techniques, as showed in 'Figure 2'.

Putting aside a series of pathologies strictly due to some more or less serious mistakes in project (such as it is showed in 'Figure 3') and, more frequently, to mistakes in the realization of the different parts of the works and although the frequent attempt of taking particular precautions and devices to avoid condensation, the pathologies due to condensation are particularly serious. The hygrothermal quality rules concern three main aspects: thermic insulation, superficial condensation risks, condensation risks in external walls. Generally speaking, the condensation concerns the rooms whose walls have a superficial temperature lower than dew's one.

The analysis confirms that this kind of pathologies can be brought back once more to mistakes in project and realization ('Figs 4 and 5'), but also to mistakes in conduction of the heating system and the "bad use" of the building.

Among the more frequent mistakes in project there are:

- not repaired thermal bridges;
- mistakes in the appraisal of *useful conductivity* of materials used, insulating and not insulating, that can be even higher in comparison with the theoretical ones resulting from the laboratory tests;
- slighter insulating elements that can't avoid condensation and do not respect what the Act 10/91 prescribes;
- non-use of controlling devices to control relative humidity that, in use, can be really high because of the internal water vapour caused into the air by people, equipments (cooking stoves, gas stoves, etc.) and by doing specific actions (cooking foods, washing up and drying dishes, clothes and floors, having vapour from baths, showers, etc.).

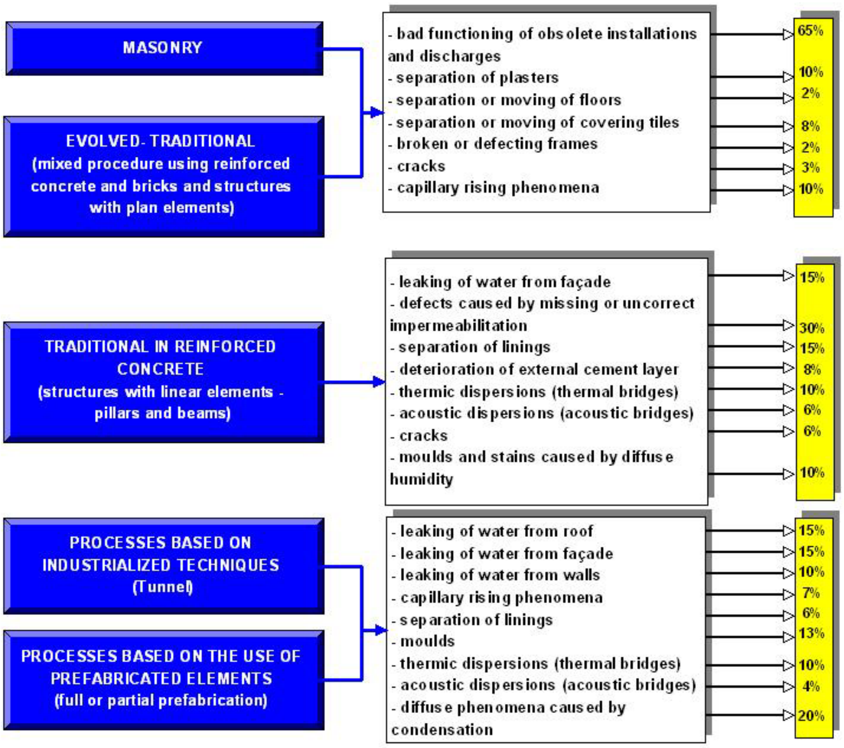

Figure 2. Relationship between building typology and pathologies.

MISTAKES IN PROJECT	
Covering closings	**External Vertical closings**
- wrong evaluation in technical leans to define the way the water flows down; - missing distribution of main collecting spots and wrong placing in areas easy to inspect and missing evaluation of the possibility of self –cleaning of the floor; - wrong evaluation of thermic expansion joints in difficult condition in use caused by seasonal and daily thermic variations and the consequent damaging of sheath; - wrong planning of the perimeter border (sheet) in correspondence of parapets, sky lights, entries, chimneys, that show heights and geometries not adequate to protect from waters and any other weather, causing humidity, internal stains, etc.; - pier to create a lean, laying on the insulating panels without their metal lath of reinforcement without proper solutions to cause an homogeneous of the material, etc.; - wrong planning of the system of conveyance and collection of waters; - wrong planning or realization of the impermeable connection between the parapet and floor, causing damages by leaking of water, evaporation and condensation, etc.; - wrong planning of parapets showing an inopportune geometry and, consequently, inadequate to the dynamic of winds and rain water, which often causes cracks, leaking of water, stains and spots of façade, etc.	**- reduced thickness of reinforced concrete sheets of the external layer of vertical panel, which caused a premature corrosion of reinforcements; - wrong selection of materials or stopping up components related to climatic and environmental characteristics of the site (seasonal temperature, significant thermic variations, wind system, geological situation, polluting sources, etc.) which causes different kind of phenomena such as stains in the walls, condensation, moulds, etc. In other words they didn't chose a complex of materials for the building system considering their effective environmental aggressiveness and the use of the structure; - wrong or missing planning of thermal bridge making them hunchbacked, causing cracks and damages, etc. - wrong geometry of perimeter of external frames that aids the stagnation of water on the sides of the room, which causes the oxidation of the window frame and stains caused by lateral drippings of water and separations of the plaster; - wrong geometries of formal design of the façade, not adequately harmonized finishing or not adequately connected to the support, making visible critical points caused by leaking or stagnation of water, separation of material, internal moulds, etc; - wrong prediction about the performance of the external layer as for hygrometric, climatic, polluting characteristics of the site, causing stains and spots, separation of materials, etc.**

Figure 3. Main mistakes in project of horizontal covering closings and external vertical closings.

Figure 4. An example of the lack of attention in the carrying out and junction of single strata prefabricated panels. Some of them presents breaches and the most part of joints are not adequately sealed. It is inevitable in this way that there are leakings of water and infiltrations of air inside the habitable rooms, causing the creation of thermal bridges.

Figure 5. The wrong project of water piping and collecting system produces leakings of water caused by stagnation, cracks, lack of cohesion of concrete, stains on external surfaces.

Among the most frequent mistakes in realization there are:
- incorrect carrying out of insulating materials (for example without vapour barrier or, even worst, with a vapour barrier put on the insulating "cold face");
- insulating materials thicker than the ones calculated by the planner;
- using of insulating decaying in time;
- using of external plasters that avoid the water vapour coming from inside can exit.

As for mistakes in conduction of heating system and the bad functioning of the building can be mentioned:
- intermittent or too long unused heating system. In fact, the intermittence in heating causes condensation especially in bedrooms where during the night there is a high production of water vapour because of the breath of sleeping people and, simultaneously, a drop in temperature of air and, consequently, in temperature of the walls;
- splitting up of heating by autonomous systems. In fact, the problem of different timing of autonomous systems causes a less efficient heating of the whole building and this can be a concause in the facing rooms.

Finally, as for the bad use of the building, we can notice:
- the abolition, due to aesthetical causes, of the cooker-hood in the kitchen;
- the presence of furniture and pictures at the external walls. This is the case, for example, of built-

in wardrobe set against the facing perimetrical walls of the bedrooms. The fact that there are furniture that usually have a noticeable thermic resistance, causes a drop in temperature of walls where they are set against and the temperature of the corresponding thermal bridges, which causes condensation and mouldy stains.

Schematically, the main causes of humidity into the flats results to be:

- percolations of rain water from roof, cracks in the walls, open joints, holes in the materials, etc.: this kind of problems are quite unusual (with the exception of damaged impermeabilization), which can be easily identified, and all the experts know the recovering techniques;
- leakings of water from foundations: it is valid what explained above even if the recovering techniques are more complex and expensive;
- leakings of water caused by escaping water from canalizations, water-pipes, drains, etc.: usually the regeneration is made a long time before the mould forms;
- excessive hygroscopicity of lining;
- humidity in building;
- superficial condensation.

The standardization of recurrent defects made it possible to express typical solutions for regenerating works, that have the value of "*rules of art*".

4 REFERENCES

ANIACAP, 1986, *Le Leggi per la casa*, Ed. Edilizia Popolare, Milano.
ARCANGELI, S. & AUGELLI, L. 1987, *Le patologie da condensa*, Ed. Edilstampa, Roma.
BELLICINI, L. GRUTTADAURIA G. (a cura di), 1985, *L'edilizia in Italia: orientamenti e tendenze del mercato,* Indagine CRESME, ETAS Periodici, Milano.
BLACHÈRE, G. 1971, *Savoir Bâtir*, Italian version, Hoepli, Milano.
CIPOLLINI, R. 1982, *L'organizzazione del lavoro in edilizia 1945-1980*, FrancoAngeli, Milano.
FERRACUTI, G. 1982, *La casa: mercato e programmazione*, Ed. G. Einaudi, Torino.
IMBRIGHI, G. 1987, *L'edilizia economica e popolare. Tecnologie e progetto*, Editrice NIS, Roma.
LEMBO, F. 1988, 'Qualità e conservazione nel tempo delle costruzioni industrializzate - l'esperienza inglese', in *Recuperare*, [gennaio-febbraio 1988], pp. 90-99.
LEMBO, F. & MARINO, F.P.R. 2002, *Il comportamento nel tempo degli edifici – Cause di degrado e soluzioni progettuali dei sistemi edilizi 'tradizionali' ed 'industrializzati'. Casi di studio*, EPC Libri, Roma.
MACCOLINI, M. 1981, 'Le tecnologie per la residenza tra evoluzione ed involuzione', in *Coscienza della Città*, Edizioni E.A. Fiere di Bologna.
TALANTI, A.M. s.d., *Storia dell'Industrializzazione in Italia 1945-1954*, A.I.P, Volume I.
ZAFFAGNINI, M. 1982, 'Italia dei miraggi ricorrenti' in *Tecnologie per la residenza*, BE-MA Editrice, Milano.
ZAFFAGNINI, M. 1982, 'Le nuove strategie insediative', in *Tecnologie per la residenza*, BE-MA Editrice, Milano.

Renovation of Housing using a glass skin extra space, more comfort, better energy performance and an extended life span

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ABSTRACT

In almost every European country a large stock of houses exists. Many houses are outdated in respect to the space offered to the occupants, the general level of comfort and the energy performance. In the Netherlands some 1,300,000 post war houses built in the 1946 - 1965 period are now outdated and are facing either renovation or demolition.
Row houses consist of some 50% of the total housing stock (fig 1)

Fig. 1 Typical outdated row housing in the Netherlands (1953)

In this research a method of renovation using a greenhouse like skin is presented. The glass skin is placed over an existing house giving space for additions and creating a thermal buffer. It is more or less like fruits and vegetables being preserved in glass. The consequences regarding ventilation, occupant requirements and build ability are discussed. The result of this approach is prevention of demolition of outdated houses and enabling the occupants to stay in their dwellings. The occupants will enjoy more space, more comfort and an improved energy performance.

KEYWORDS

Housing, Durable Renovation, Energy performance

1 INTRODUCTION

After World War II in the Netherlands some 30% of the existing housing stock was damaged or demolished. In the 1946 – 1965 period in the Netherlands an enormous effort has been effected in building over 1.300.000 new homes.
These houses are now (2004) considered small, poor energy performers and of only limited comfort. It must be emphasized that in those days these houses formed the perfect answer to the need of the people and most people were really happy to live in these houses.

A closer look at the typical post war houses shows a two story pitched roof row-house with a small garden in the front and a slightly bigger garden at the back. (Fig 1)
Usually the lot measures some6 x 25 meters, (150 m² or 1612 square feet) the ground floor generally consists of an entrance, a living room, a small kitchen, a toilet and a staircase. The first floor has two bedrooms, a small bedroom and a bathroom. Under the pitched roof there is an attic only for storage due to the low height and often only accessible by a folding stair. The ground floor plan makes clear that space is very limited [fig 2]

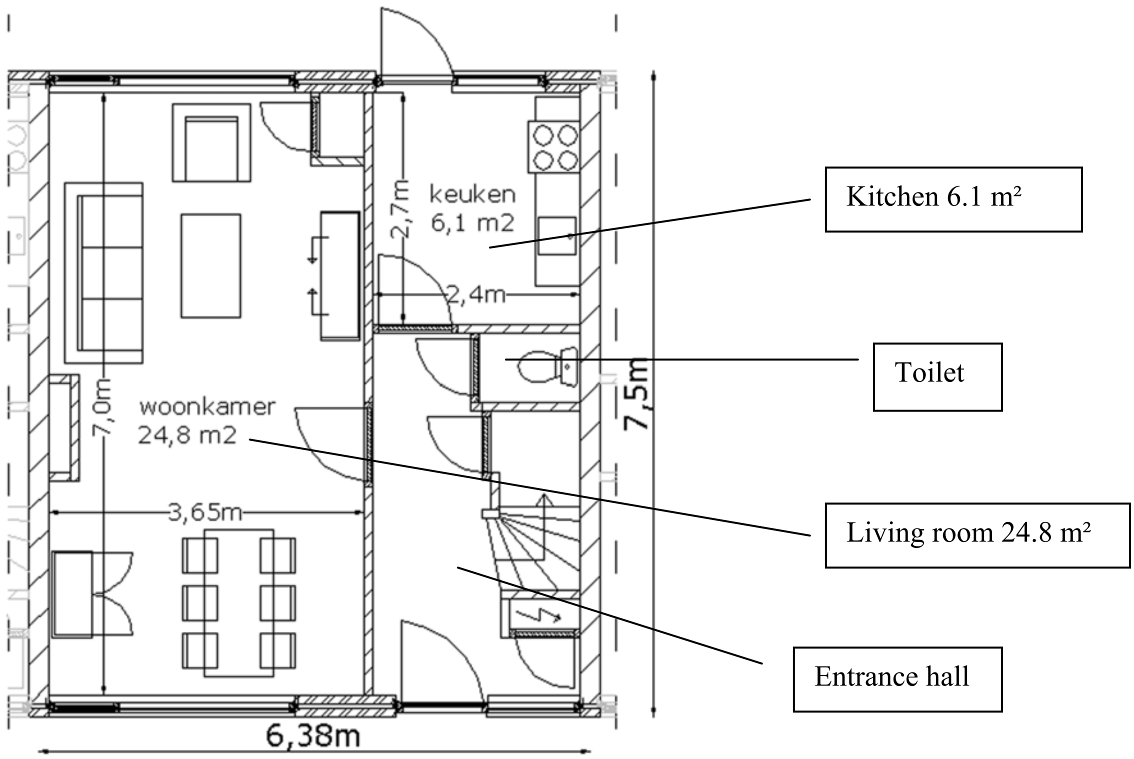

Fig 2 Row house ground floor plan.

Over the past 50 years many changes have taken place in society. Regarding housing the most significant changes are space requirement per person, energy performance and level of comfort.

The space requirement per person has gone up from 21 m² in 1947 to 50 m² in 2000. The number of persons per dwelling has gone down from 4.6 persons in 1947 to 2.3 persons per dwelling in 2000.
[NVTB, 2001]

Not only space requirements and comfort requirements have changed, also energy performance has become an important issue. The old fashioned row houses with old fashioned façade details consume a lot of energy according to today's standards and need improvements in order to stretch the life span. Single pane windows and cavity walls without thermal insulation do not meet today's requirements.

2 APPROACH

This study focuses on the renovation of outdated housing in order to stretch the life span. In this specific case the possibilities of a glass skin over an existing dwelling are analyzed. The dwelling is more or less "preserved" like fruits and vegetables in a glass jar. The glass skin gives the opportunity to add space to the dwelling. Using the results of an occupants survey, [Dogge et al, 1996] the occupants' requirements are determined and the extra space is filled in. The energy performance of the existing house is determined according to the existing Dutch standard.

3 ANALYSES

Postwar row housing in the Netherlands can be characterized as being of an extremely time-related design. After the Second World War there was a lack of houses, material, equipment and skilled labor. With great effort a large number of houses have been built in unprecedented quantities. [table1]

year	*Number of houses built*	*Year*	*Number of houses built*
1946	800	1956	68000
1947	8000	1957	87000
1948	36000	1958	87500
1949	42000	1959	82000
1950	44000	1960	82000
1951	59000	1961	81500
1952	56000	1962	79500
1953	60000	1963	97500
1954	68000	1964	100000
1955	60000	1965	115000

Table 1 Number of houses built in the 1946 – 1965 period in the Netherlands.
[Tellinga, 2004]

In the Netherlands over 600.000 outdated post war row houses exist while only some 60.000 new houses being build per year. The old row houses are considered too small, offering only very limited comfort and the energy performance is low. In fact already many these outdated houses have been demolished in order to create space for new houses. If all 600.000 outdated row houses are demolished this will represent an enormous amount of debris, as on house alone represents some 140 to 150 tons of material. For comparison, the empty weight of a Boeing 747 SP is 147 tons.

New dwellings have been constructed with a greenhouse like skin over the entire house. This has resulted in comfortable houses with a good energy performance [Karsenberg, 2002] fig. 4

Fig 4 New row houses under a glass skin being built in Culemborg, Netherlands (2001)

Combining the greenhouse like skin with the existing houses is a new concept.
The greenhouse skin offers the occupant more space and also creates new possibilities, like flexible space: in the summertime the entire space under the glass skin can be used, in the winter time the space between the existing house and the glass functions as a thermal buffer, improving the energy performance. The useable living area changes with the season.

Analyses has been carried out regarding occupant requirements and offered space. Extra space is required, on the ground floor the hall, living room and kitchen are considered being too small. On the first floor the bath room is considered to small and an extra room on the attic is wanted in order to have a possibility for a home office.

4 DESIGN

The design consists of two elements: The placing of a glass skin over the existing house and the addition of elements in order to fill up the required extra space.
The basic design concept is given in fig 5

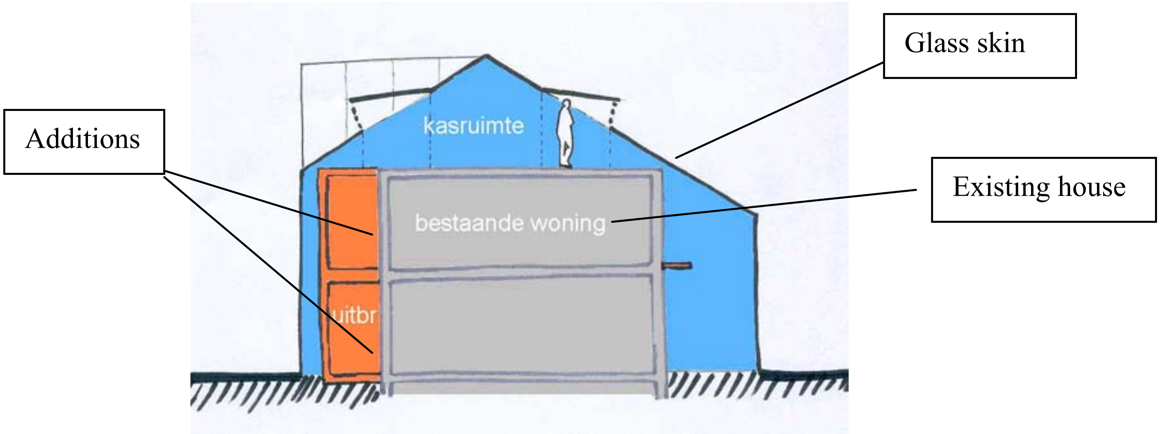

Fig 5 The existing structure, the glass skin and the addition in vertical section.

The advantages of the design can be seen in both the ground floor plan and the first floor plan. With only a small addition in the space provided by the glass skin the kitchen is moved from the back to the front of the house, enlarging the living room. Also the toilet is moved enlarging the hall.(fig 6)

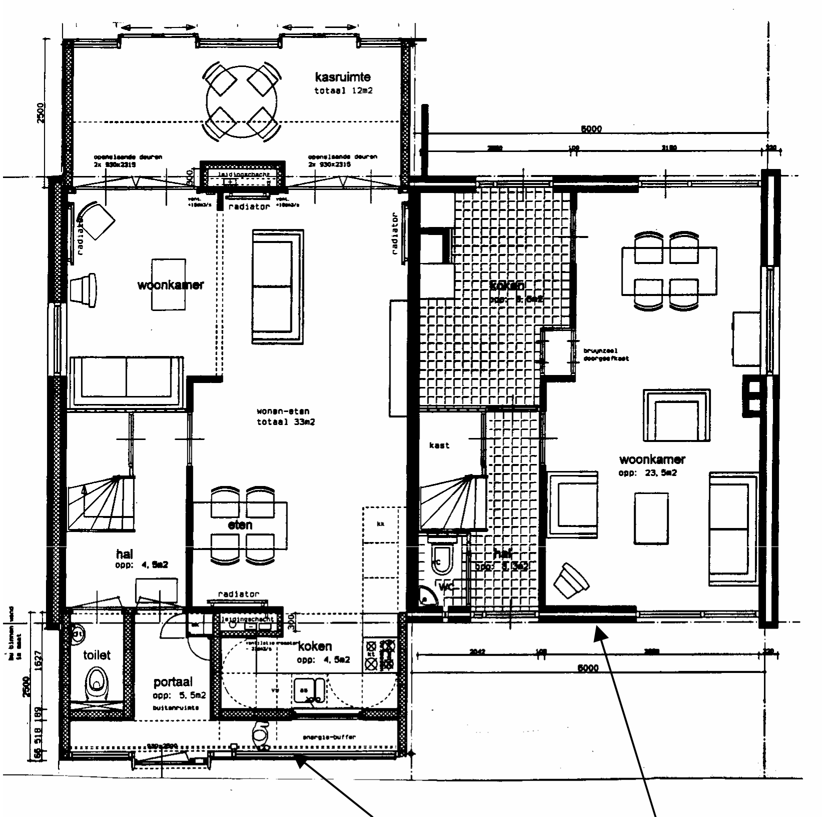

Fig 6 The ground floor plan under the glass skin (left) and in the existing situation (right)

On the first floor only a small extension is placed above the extension on the ground floor, enabling the bath room to be increased in size. The bedrooms all have a balcony covered by the glass skin. At the outside of the front and back façade a services duct is added giving space for ventilation shafts, A central vacuum cleaning system and other services. (Fig 7)

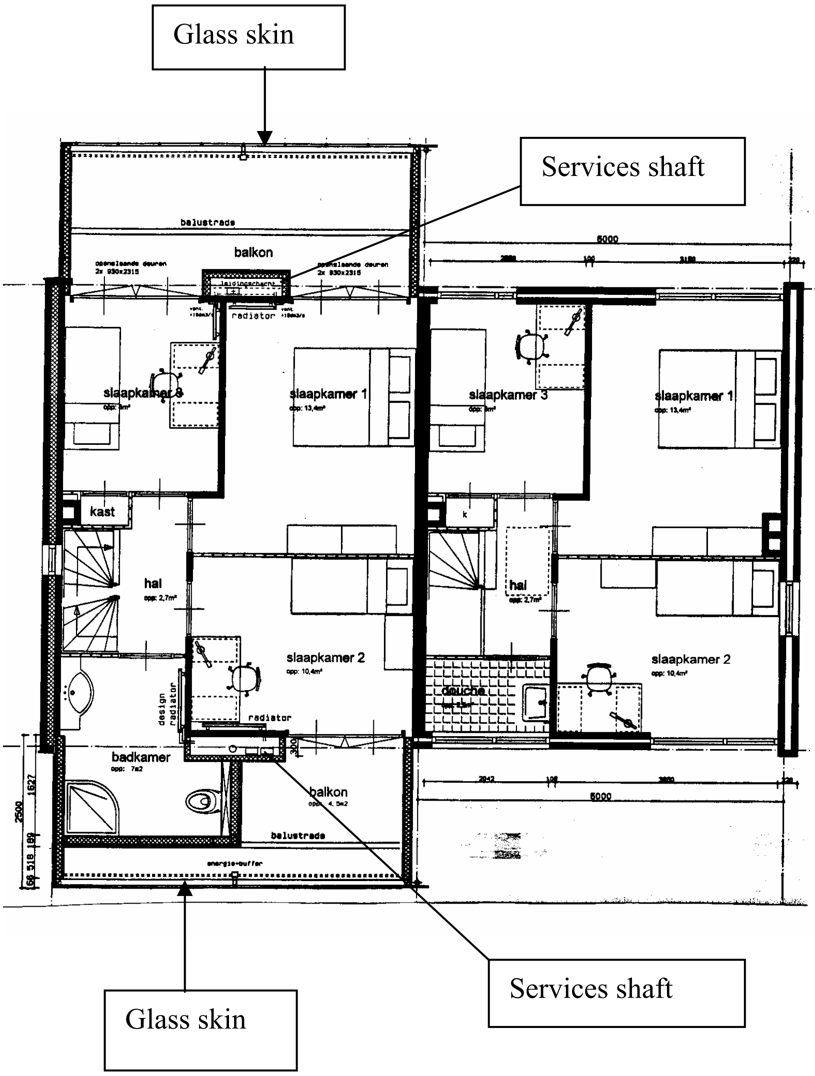

Fig 7 First floor in the new situation (left) and in the existing situation (right).

5 RESULTS

The glass skin over the existing dwelling will create a buffer space between the outside climate and the climate in the house. During winter this phenomena will reduce the energy demand of the house for heating. In the summer period overheating of the greenhouse is to be prevented. Large openings for ventilation are necessary. Using a computer simulation with a ventilation factor of 10 the temperature in the dwelling will rise approximately 2º C above standard has been calculated.
The question is if additional ventilation is needed. If necessary there is space for extra ducts available in the services shafts.
During winter the extra area between the existing house and the glass skin is not to be used and acts only as a temperature buffer. The space should not be heated, as this will result in higher energy losses.

The glass skin building structure is an existing product in the agricultural greenhouse building sector. The structure is able to carry snow loads according to the Dutch standards. Also sufficient detailing is provided to cope with condensation and ventilation.

The services shaft mounted at both front and back façade enables placing of large piping for a forced heat exchange ventilation system and also enables accessibility for maintenance of this system.
This maintenance is extremely important as all ventilation is provided by the heat exchanger system and "sick building syndrome" symptoms are to be avoided.

The occupant will receive in this concept extra space and extra comfort, the operating cost will be reduced significantly.

Using and adding to an existing structure will reduce demolition cost and disposal of waste.
The energy incorporated in the material will not be lost, only limited material is needed for the proposed retrofit again reducing the energy needed to create and transport this material.

The average yearly consumption of natural gas for a Dutch dwelling is 2087 m^3, representing an emission of some 3715 kg CO_2. With the glass skin the natural gas consumption is reduced to 625 m^3 natural gas, representing an emission of 1112.5 kg CO_2 The reduction of CO_2 emission is 2600 kg.
The Dutch aim of CO_2 reduction is some 2.0 Mton for the year 2010.
If it would be possible to realize the 2.6 ton CO_2 reduction over 600,000 houses a CO_2 reduction of 1.56 Mton CO_2 would be achieved, a quite remarkable conclusion.

In respect to the CO_2 emissions it should be mentioned that the emissions due to heating purposes still are very limited. The family car is producing much more emissions, at an average 16,330 km/year and a consumption of 1 liter to 10 km the family car consumes 1.633 liters of petrol.
The CO_2 emission of petrol is 2.2 kg/liter, thus the CO_2 emission of only the family car is 3600 kg.

6 DISCUSSION

The final question is will the occupant accept this concept or not. The extra space in summer will be appreciated, but will the occupant also accept the smaller area in winter?
Will the occupant accept the heat recovery system as main source of ventilation, being used to the so called natural ventilation concept, which is nothing but simply opening a window.
In this design the glass skin covers the entire dwelling. The question is the use of the skin at the north side (in the drawings the entrance), as the buffer here will almost not be heated by the sun.
Another aspect to be dealt with is cleaning. A glass cover will allow dust to build up. Not all places are easily accessible or can be cleaned easily.

7 ACKNOWLEDGMENTS

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8 REFERENCES

NVTB, 2001, Consument gericht toeleveren in de 21 st eeuw.
Dogge, Pott, Smeets, 1996,Dynamics, Housing preferences and appreciation.
ISBN 90-6814-064-7 (in Dutch only)
Tellinga, 2004, De grote verbouwing. Verandering van naoorlogse wijken
Karsenberg, Wienberg, 1999-2002, Housing Lanxmeer, Culemborg. (interview)

Reduced service life due to common building failures in Denmark

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ABSTRACT

Durability is more than wear and tear in the sense that many problems occur that terminate the service life of building materials and components far too early. These problems are usually associated with big expenses for repairs or exchange of the involved materials and components. Moreover, additional expenses are very often needed for necessary repairs of adjacent materials/components and/or correction of the constructions in order to avoid that the problems reoccur.
Some of the most common failures in Denmark are associated with roofs including flat roofs and sloped roofs, bathrooms including watertight membranes and floor gullies, wooden floors and decay of buildings due to dry rot. In recent years big problems have also been encountered in constructions with high humidity levels associated with health problems and resulting in mould growth. Mould problems have lead to considerable expenses, e.g. due to lack of maintenance and/or incorrect refurbishment.
This paper gives an overview of a number of the most frequently occurring building failures and their reasons and gives examples of how they are dealt with. Finally the paper discusses how the different reasons for building failures might be attacked in order to reduce or even avoid failures in the future

KEYWORDS

Building surveys, building failures, service life, roofs, bathrooms, floors, mould.

1 INTRODUCTION

Building failures have a significant effect on the service life of building materials and components and may in severe cases reduce the service life to almost nothing. Problems with materials and constructions can never be totally avoided. However, considering the nature of the failures seen in practice and not least the huge number of failures that occurs, it should be possible to gain great benefits just by reducing in the number of failures. It is believed that the situation can be improved by attacking the problems systematically, even though this should not be considered an easy task, as much effort has been put into this area for many years - apparently without great success.

The reasons for the failures are many but 3 of the most common are the following:

Many of the failures occur due to the fact that common knowledge is not used. For example the well-known fact that a vapour barrier should not only be made of a vapour retarding material, but it should also be mounted so that it is airtight in order to avoid humid indoor air from penetrating to colder parts of the construction. Nevertheless, in many cases the vapour barrier is not mounted correctly with the consequence that problems are encountered soon after the building is taken into use.

Another big problem is the use of well-known materials and constructions in new ways. For example the design of many floor constructions has been changed due to energy saving requirements in the building regulations (or energy saving measures made by the occupants). These changes may at first appear to be minor, but unfortunately the results could be major problems, e.g. with mould growth in the constructions.

The last problem to be mentioned is the introduction of new materials. The use of new materials always calls for precaution as no experience exists. Quite often it will be necessary to perform some sort of experiments – or at least an analysis – in order to assess the use potential.
A number of the new products come without documentation for the most necessary performance properties, making it difficult – if not impossible – to use them correctly. Even worse, the test results for some products are misinterpreted by the supplier so that the users are mislead, e.g. claiming that a reflective foil has the same effect as a thick layer of insulation.

In the following a number of examples of common building failures will be given followed by a discussion about the reasons for these failures. Finally there is a discussion of how the problem of building failures might be reduced.

2 PROBLEMS WITH ROOFS

During the last 30 years many changes have occurred as regards roofs in Denmark. Some of these are due to ever-stricter energy saving measures now calling for the use of about 300 mm insulation in a roof construction. Besides, new building components and materials have been introduced in flat roofs as well as in sloped roofs resulting in a number of problems.

2.1 Sloped roofs

For sloped roofs 2 major problems are:

- Change of constructions with little or no insulation to energy efficient constructions have lead to failures due to insufficient ventilation after the change and/or failures in the vapour barrier.
- Change of constructions with roof tile underlay from ventilated to unventilated type.

The first issue is an example of a technical problem with an easy and well-known solution, namely the introduction of a decent vapour barrier, if necessary, and the use of existing directions on how to add insulation to an existing construction without jeopardizing the ventilation and without introducing risks of condensation in colder parts of the construction.

The second issue is more difficult. Some 10 years ago a number of new thin membranes were introduced on the Danish market as alternatives to the existing roof tile underlays. The advantage of the new products was that not only were they watertight, but they were also vapour permeable. Consequently, the traditional ventilation between insulation and roof tile underlay could be omitted and the roof construction did not need to be as thick as before. Unfortunately the new products did not perform without problems, as some of them were only watertight for a short period of time. In such cases an investigation of the new products was launched especially to identify the necessary performance properties including the degradation mechanisms acting under the in-use conditions. One of the problems identified was the so called “tent effect”, i.e. water penetrated some of the products when these were lying directly on a substrate for example plywood – the effect of water penetration was similar to water running through an old fashioned tent when it was touched on the inside. A test method was elaborated due to these investigations and a number of products have been tested resulting in the withdrawal of quite a few products from the market. Another problem was the workmanship as there was a considerable lack of qualified information on how to make this new type of constructions. This problem was solved when new directions were sent out. Besides, in a new voluntary association all suppliers have to document how a number of commonly used details are made (by building them in a mock-up). However, there are still problems especially with products that are watertight only for a short period of time, but so far further reasons/degradation mechanisms have not been identified.

Figure 1. Deterioration of plywood in a sloped roof made with wood-based elements assembled in situ. Due to leaks in the joints between elements in the corners, humid air has penetrated into the construction, where it condensates on the cold surfaces above the insulation.

2.2 Flat roofs

For flat roofs 2 major problems are:

- Change of constructions from solutions made in situ to prefabricated solutions assembled on the building site.
- Change of constructions from ventilated to unventilated have led to a significant number of failures due to insufficient information/knowledge about a new type of vapour barrier.

The first subject is especially connected with leaks in the vapour barrier over the joints and lack of securing the joints between elements from penetration of precipitation immediately after assembling. The result of both is a high humidity level in the elements resulting in mould growth and in extreme cases in deterioration of the wood. Leaks in the vapour barrier are mainly due to the failure of recognising the importance of securing the joints from beneath and - to a much lesser extent - due to problems with the tolerances of the joints, which the present solution is unable to accommodate.

The second subject is connected with the large proportion of flat roofs that are made as prefabricated wooden elements with a special vapour barrier – Hygrodiode – that not only acts as a common vapour barrier but also allows humidity from the roof construction to penetrate back to the interior of the building in summer time. The major part of roofs of this type performs without problems but unfortunately the use is restricted as the performance is dependent on the sun to warm up the surface in order to force the humidity out of the construction, i.e. the construction is not functioning properly on parts of the roofs (and facades) lying in shadow. This has resulted in a fair number of failures as the restrictions in the field of use have only been common knowledge for a short period of time.

Figure 2. Leaks in the vapour barrier (as seen to the left) due to poor workmanship allow humid air to penetrate to this school roof, where it condensates and causes mould attack and dry rot.

3 BATHROOMS

Bathrooms have for a number of years been #1 as regards failures. In fact failures occur in all kinds of bathrooms whether traditional or more innovative.

3.1 Traditional bathrooms

Traditional bathrooms are made from concrete, masonry or lightweight concrete. All things considered, the performance is satisfying, as solutions of this type are rather tolerant to minor mistakes.

The most common failure in traditional bathrooms is old floors that are renovated by removing existing terrazzo and installing a new screed with ceramic tiles but without changing the floor gully to an adequate type or securing the joint between floor and walls, see Fig. 3. Failures show as water penetration between floor and floor gully (water often penetrates between the old and the new floor and from here to the room below) or through the joint between floor and walls. Failures of this type are examples of problems that occur as a result of not following legal requirements and existing knowledge.

3.2 Lightweight bathrooms

For the past 30+ years so-called lightweight bathrooms have been used quite extensively, because they are cheap, easy to install and without excess water. The major part of these constructions is made from board materials, e.g. gypsum boards, calcium silicate boards or plywood.

Figure 3. Penetration of water through a floor constructed with a solid steel profile with concrete. The penetration is caused by water entering cracks between an old and a new floor, because the floor gully was not replaced. The steel corrodes and expands causing the cracks to grow bigger, thereby accelerating the deterioration.

Unfortunately the use of these lightweight bathrooms has lead to a rather large number of failures with water penetrating though walls and/or floors. Common failures are use of unsuited materials e.g. board materials with insufficient properties, watertight membrane missing or too thin and failures around details especially floor gullies, see Fig. 4. The failures have a number of different reasons but most of all it has proved to be difficult to teach the skilled labourer and the user the importance of strictly following the directions for use.

Figure 4. Deterioration of plywood wall and entire floor construction due to ceramic tiles applied directly on plywood - without watertight membrane. Besides the missing membrane the construction was too weak to resist dimensional changes in the plywood.

4 FLOORS

4.1 Crawl spaces

Crawl spaces have been used extensively in Denmark for many years and have been considered to be very safe constructions. However, new requirements facilitating the access of disabled persons to buildings have made the use of crawl space constructions very difficult not to say impossible. At the same time problems with old crawl spaces are increasingly due to mould growth etc.

Old crawl spaces are usually not well insulated and the users therefore like to apply additional insulation in order to save energy and to increase the thermal comfort. Additional insulation will quite often block – totally or partly – the ventilation gaps to the crawl space leading to accumulation of moisture in the construction. As many crawl spaces rely on a delicate equilibrium between humidity supplied from the environment and humidity removed by ventilation, even small changes might affect the equilibrium in an unfavourable direction. Increased humidity leads almost inevitably to mould growth and in severe cases to deterioration of the wood. The failures are most often due to difficulties in analysing the complex changes in temperature and humidity that occur when a crawl space is insulated and/or the ventilation gaps are partly blocked. In a minority of cases the problems are simply due to not making changes in accordance with common knowledge.

4.2 Wooden floors

Even though wooden floors have been used for centuries, they still account for quite a few failures – even in old well-known constructions.

For traditional floors with wooden boards nailed or screwed to joists, the main failure mode is too big variations in the widths of the joints between boards and/or creaking.

For floating wooden floors a number of failures are seen. There are some examples of uneven floors – due to the substrates being uneven and this can not be taken up by the boards. Another major problem is big variations in the widths of the joints due to restrictions to the free movement of the assembled floor. Free movement is crucial for floating floors, as dimensional changes will inevitably occur in the floor due to changes of the relative humidity in the environment.

Also quite a number of floors experience swelling due to boards mounted too close (not in accordance with the "10-board measure" as given by the manufacturer). When such floors are exposed to the humid environment of the late summer, the dimensional changes in the boards result in swelling of the individual boards and when there is no room for further expansion the entire floor swells i.e. lifts up, see Fig. 5.

Figure 5. Swelling of wooden floor due to boards mounted without possibility for expansion in the humid season.

Lately problems have occurred with delamination of laminated boards typically consisting of 3 layers. When selling the boards, the board supplier usually put restrictions regarding their use normally restricting the humidity in the environment to 30-65 % RH. This in itself is hard to keep in Denmark as relative humidity in late summer often exceeds 65 % and in winter often goes below 30 % even in ordinary dwellings. Besides, the same boards are often claimed to be fit for use in constructions with floor heating, where the humidity under winter conditions are far below the restrictions set by the supplier. Anyway dimensional changes (shrinkage) under very dry conditions may result in breakage of the glue bonds between the layers in the laminated boards.

The major part of the problems may be attributed to not having made the floors in accordance with common knowledge but a fair part can also be attributed to the materials (or rather the use of materials in places where they are not suitable).

5 MOULD GROWTH

In recent years mould growth has become a serious problem as it has been proven to have a significant effect on health. Problems are seen in many different constructions e.g. roofs, crawl spaces, bathrooms, walls with additional interior insulation, basements etc.

Mould growth is facilitated by humidity in the environment and is consequently often found where constructions are affected by problems with water penetration, rising damp, condensation etc.

A number of severe cases in Denmark have been caused by lack of maintenance especially in roofs. The consequences have often been very costly repairs e.g. renewing an entire roof construction.

Figure 6. Façade with mould growth on wind barrier made with gypsum boards. Often mould growth in facades is due to water penetrating behind the cladding without being drained out (fast enough).

In other cases problems have occurred during the building period due to missing protection of vulnerable parts from precipitation. Also these damages have been seen to be very costly. Other causes of mould growth are constructions that are changed/insulated without paying attention to the risks of jeopardizing the ventilation or to condensation in the constructions. Finally mould growth is seen after accidents, e.g. pipe breakage or fire, where constructions are soaked and, if not dried very fast, are vulnerable to attack.

6 HOW CAN FAILURES BE MINIMIZED?

The causes for failures are almost unending. The question to answer is how can this be changed and the number of failures reduced?

As can be seen from the above, the causes for failures fall in different categories including not least the 3 mentioned in the introduction.

Quite a few of the failures are caused by not using existing knowledge. Great benefits should be easy to gain, as there are no technical problems. The key is more relevant information addressed not only to specialists but to the persons performing the work on the building site. It sounds like an easy task, but actually it is assessed to be very difficult. In Denmark for example a fair number of publications have been issued regarding good practice. This includes 2-page information sheets about common failures, why they occur, how they can be avoided and how failures can be fixed. Unfortunately, even these short publications are mostly read by architects or engineers and not by the persons who need it most. The reason is believed to be that these persons are neither used to look for information nor used to read a lot and especially not in a very technical language. To overcome this problem it has been discussed in Denmark to write publications in everyday language and with many illustrative figures addressed directly to workers.

Another big problem is the use of well-known materials and constructions in new ways. Many changes that at first appear to be minor turns out to lead to quite different working conditions for the materials/constructions e.g. with reduced ventilation increasing the risk of humidification, or reduced temperature increasing the risk of condensation in the construction. In both cases the result might be mould growth or even deterioration of organic materials in the constructions. Changes in the use of materials should consequently always be assessed/analysed in order to verify that the working conditions are not affected in a negative way.

Finally the introduction of new materials is quite often associated with failures. Not because the products are of poor quality, but because their potential is overestimated – by marketing persons especially – or they are used in a wrong way. This is actually a very different matter as only one small mistake can lead to considerable failures. A prerequisite for the introduction of new materials, without experience of their use, is that the performance properties are documented and that the properties fit the performance requirements for their use. Ideally such information should be given by the manufacturer/supplier, but unfortunately information is often not available. It is proposed that manufacturers should not only give guidance on what the products can be used for but should also provide information on limitations in their use. Besides the supplier should provide the information necessary to assess whether a product is suited for the intended purpose or not. Finally, the users should be encouraged to ask for documentation – from independent institutes/bodies – and to make their own assessments prior to the use of unknown materials.

It appears obvious to provide information about materials, constructions and their proper use via the Internet. However, this should be done with care in order to avoid incorrect information to be spread and taking into account differences in building traditions between countries. Currently a lot of the information available on the Internet is unfortunately not correct, so precautions should be taken at use.

The tasks mentioned above are so big and difficult that they are best dealt with in cooperation between countries. Preferably such work should be financially supported – especially when taking the huge amounts of money used on building failures into account. If this is not possible, at least sharing knowledge on a voluntary basis would be worthwhile.

Residencial Buildings Maintenance: A Theoretical Practical Approach

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ABSTRACT

A building undergoes several sorts of waste, whether by a bad weather or its use. In this manner, interest by buildings maintenance is based on the fact that yearly, residential condominiums are compelled to spend elevated amounts for accomplishing this type of activity. Most problems that demand maintenance activities could be avoided if during the project stage existed a higher care on choosing supplies and over architectonic for expenses and procedures.
This paper has intended to identify some kinds of repairs achieved on residential buildings from Cruz Alta City – RS in South of Brazil. The record's gathering was accomplished by visitations to buildings, where a questionnaire containing matters about maintenance, pathological demonstrations, project and expenses characteristics was applied.
The main conclusion indicated buildings preservation, on the inspected condominiums is practically inexistent. Through the data analysis it is possible to confirm that periodical maintenance is accomplished only on elevators and in some cases, on water containers, electronic gates at building and garage entrances. Other styles of maintenance such as paintings, electric charge, water and drain tabulations are only considered when some problems that will have impact directly to the condominium expenses or will bring some discomfort to the tenants, due to interruption on services or condominium's area employment comes out.

KEYWORDS

Maintenance, pathological manifestation, residential condominiums.

1 INTRODUCTION

A building when in activity suffers several types of deterioration or caused by bad weather or use. The deterioration results in the need to accomplish maintenance activities along its useful life to maintain the durability and functionality of the construction.

In general, the durability and the functionality of the construction are strongly linked with the durability of the materials and elements that compose the structural part of the built atmosphere. In that case, the durability is affected by some factors that cause the degradation, known as degradation factors. Many times, those factors, when considered separately have no effect or very little affect in the durability. However, together with other, they can cause significant damages to the built atmosphere and, therefore, they cannot be inconsiderate [John 1987]. Based on those factors, the maintenance occupies fundamental role to maintain the construction with a satisfactory acting prolonging its useful life.

This way, the interest for the maintenance in constructions is related to the fact that annually the tenants are forced to spend high amounts to carry out that kind of activity. Many of those problems that demand maintenance activities can be avoided, during the project stage; if there were a larger care in the choice of the materials and in the architectural details.

2 MAINTENANCE

The absence of performance requisites along the decision processes of the undertaking associated to the lack of indicators to evaluate the products conformity and processes with the specified quality, along the production phase and use of the constructions, potentate the appearance of pathological manifestations and they contribute to the consolidation of inadequate techniques and unproductive [Calmon & Griloall 2000].

In that sense, the maintenance of the buildings includes all of the activities that take place in its equipments, elements, components or facilities, with the purpose of assuring them suitable conditions of safety, livable, efficiency and other, for the execution of the functions for which were made or built [Perez 1988].

NBR 5674 [Brazilian Association of Technical Norms 1999] justifies the maintenance as a form of “damages, decadence, loss and other risks, by the attentive verification of the use and conditions of permanence of the technical and functional characteristics and of their facilities and equipments ".

NBR 5674 describes the purposes of the maintenance through the following characteristics: functionality, safety, hygiene and comfort.

The current trend is the population to inhabit condominiums as a form of safety. Bezerra [2000] considers that residential condominiums have been trying to provide, in the metropolises, the existent life conditions as in small formerly cities, trying to isolate the external world violence. In that way, the legitimate desires to obtain comfort and life quality, it has been trying to adopt devices and equipments that should work whenever requested, without the interruption, and, most of the time, in smaller and smaller costs. To reach those objectives, in fact, the administration of all those activities should be efficient and for this it constitutes a challenge.

In general, the constructions are a physical support for the direct or indirect accomplishment of a whole activity type, and own, therefore, a fundamental social value. However, the constructions present characteristics that vary from other products. Usually, the constructions are built to assist their users for many years and, along that time, the services should present conditions adequate to the use of which they are destined, resisting the environmental agents and use that alters their initials technical attributes [Brazilian Association of Technical Norms 1999].

According to Perez [1988], the maintenance begins in the moment that the builder gives the building; however, the process has origin in the planner's drawing board. Vasconcelos [1998] considers the change of mentality in the relation builder/client in the delivery moment of the property it is a fundamental point.

When is talked about maintenance, it is verified that since the beginning of the XX century, this activity stopped being something that only costs money to assume a prominence position [Rodrigues 2003]. The tendency is that the reality that already existents in the manufacturer industry extends for the residential area. Independently of the difference between the property and industrial maintenance, it can be observed that the maintenance activities started to receive special attention in the last years due to growing concern and the understanding of maintenance importance of the properties. Persike [2002] in his paper he proposes a methodology that seeks to insert a planning maintenance, since the best moment of the interference, even the team destined for its execution and the registration for future obtaining of an updated dossier, in conditions of being improved with new techniques and products that will appear in the market.

In front the presented context, it is verified that the property maintenance possesses a wide development possibility in what plays the means to reach patterns already practiced by the maintenance used in other industries. The cultural factor many times is the main barrier for the development of a preventive maintenance program. The access to the information allows apparent the relations between supplier and consumer, that is, right and duties of both parts. An appropriate orientation followed by an operation manual of use and a wide and detailed maintenance can propitiate to the users warranty of the good construction performance [Santos 2003]. These information should be correct, precise and adequate about the goods and services [Grinover apud Santos, 2003]. The user, many times lay in relation of how correctly use, execution and maintenance of its good can for lack of information, be one of the main damages causes to the construction [Santos 2003].

It is indispensable some change of mentality about constructions maintenance. The users need to be aware that a property with appropriate maintenance provides larger satisfaction and larger valorization.

2.1 Maintenance Costs

The loss of performance in a construction or of one of its parts results in recovery activities that unavoidably result in costs for their users. In spite of this is hardly possible to observe some kinds of concern in relation to the maintenance planning of those constructions. The problem becomes more serious while the damages are not repaired. It is still necessary being known the quality of the materials, executive procedures adopted in the construction and of the level of administration of the preventive maintenance, implanted in the use of the edification.

The costs involved in the maintenance suffer influence of several variables that for its turn are distributed in the several stages of the constructive process. A clear definition of the types of involved costs facilitates the understanding and future analysis. This provides a larger visibility of the expenses in the different stages that a construction goes through. In that way, the constructive process can be divided in three large stages: conception, execution and use [Lichtenstein 1985]. For each of these stages differentiated activities are associated.

Meira [2002] lectures on several types of costs involved in a construction among the maintenance costs. Besides, it is possible to verify in his literature that there is an over position of costs. Many times a certain type of cost appears in more than a category. As this author, the costs can be divided in:

- global (along the useful life): included in this phase the construction: costs, operation, maintenance, modernization or adaptation and of demolition or sale;
- maintenance and operation: are included in this phase the current costs of: cleaning, illumination, operation of equipments, consumption of water and light and gas, taxes and security;
- common area charge: wages, administrative costs, purchase of cleaning products and maintenance, office material (papers, photocopy among other).

Regarding the constructive process should be pointed out that when the same is accomplished with integration among the stages (planning, project and execution) a larger possibility exists to provide a product (construction) better quality. In this case, it is noticed that the users have a wrong idea, once built the edifications it will remain in the same good conditions for a long time.

One of the points that cannot be forgotten is the socioeconomic factors that interfere in the use of the constructions. Residents of low incomes prioritize other needs which can present homes with a larger degree of deterioration or for the quality of the construction, or for the lack of maintenance.

2.3 Maintenance and the user's satisfaction

It is verified that in the extent of the constructions, maintenance is faced as cost and actually it should be faced with investment in the property. That investment has as their users' consequence larger immediate satisfaction and a larger valorization of the property.

The concept that maintenance adds value to the property is not many times practiced by the users of the same ones by lack of a larger explanation of the advantages of executing maintenance. It is observed that many times the users have the wrong idea that once built the construction she will stay in acceptable conditions for a quite long period.

The users' satisfaction is another factor that should be considered once it will influence in the relation user and construction. Oliveira's paper [1998] tries to identify the factors that interfere in the users' satisfaction associating the quality of the home. Reis [1998] identifies the relation of the maintenance levels, cleaning, inner and external appearance with the residents' satisfaction.

3 ANALYSES OF THE RESULTS AND CONSIDERATIONS

This paper was accomplished in the city of Cruz Alta located in Rio Grande do Sul south of Brazil. The studied city is considered of small size (inhabitants) and the sample consists a total of 14 constructions basically residential constructions and only two (correspond to 14,28% of the sample) had small commercial establishments in the first floor and the remaining 85,72% are of constructions totally residential.

The paper basically had the following stages:

- elaboration of the questionnaire to be answered by the person in charged of the construction;
- accomplishment of a previous contact with the responsible for the construction;
- application of the questionnaire and visits to the buildings;

- tabulation and analysis of the data: accomplished with the aid of the Sphinx program.

There were several points suggested regarding the architecture of those constructions. Items as number of units for pavement, number of pavements, besides the existence of elevators and garages. In the general, about 35% of the studied constructions possess 4 units for pavement. It is worth to point out that 57,15% of the condominiums of the total sample possess elevators. It is observed, still, that about 14,28% of the condominiums possess more than 10 pavements. Other 28,57% possess from 8 to 10 pavements and 21,43% possess from 5 to 7 pavements and the remaining constructions possess less than 5 pavements.

The questionnaire approached subjects regarding incidence of pathological manifestations. In visual exam it was possible to verify that all of the constructions possess some mainly type of pathological manifestation related to the facade. In many cases was possible to verify that the lack of maintenance in the constructions besides problems for their users they also cause, a terrible impression. Figure 1 shows that disregard with the construction.

Figure 1. Facade of a construction with painting problems and detach of the plaster.

Due to the typology of the roofs which are covered with cement fibro tiles in most of the cases (see fig. 2) and *plate-bande* use, it is verified a considerable percentile of coming infiltrations of the roof. One of the verified causes of those infiltrations was the incorrect measurement of the gutters joined to the *plate-bande*, gutters with obstructions and with imperfection (rusty and holed). Table 1 presents the incidence of the covering types. That covering type provokes pathological manifestations that risk the appearance of the facades and the comfort of the construction users.

Type of covering	Incidence (%)
Fibro cement or similar with *plate-bande*	85,72
Fibro cement or similar without *plate-bande*	0,0
Visible roofs ceramic	14,28

Table 1 Roofs Incidence in the constructions

Figure 2. Roof covered with fibro cement with *plate-bande* presenting infiltration stains.

Table 2 shows the incidence of pathological manifestations like infiltration coming from the roof.

Occurrence	Frequency (%)
Exist	14,28
Existed	28,57
Don´t Exist	57,15

Table 2 Occurrence of coming infiltration

Figure 3 shows clearly the endangerment of the construction's facades. In some cases, in spite of answering that there never was infiltration was possible to verify moisture signs in the units just below the roof.

Figure 3 Moisture originated from the roof endangering the home of the last pavement.

Referring to maintenance, it was possible to verify in the several items approached in the questionnaire that basically is done in the elevators and in the water tank. And in the water tanks in spite of more than 80% of the constructions affirm to maintain a periodic maintenance two constructions didn't know to inform the frequency of that maintenance.

The other items approached in the questionnaire showed other several problems with the constructions and the largest problems are related to the water, that is, some leak type, the infiltration getting close to 57,14% of the sample with moisture problems in the units, following by painting problems in facades and infiltration in the under-ground, both with 42,85% and subsequently, fissures in the facades with 21,42% of occurrence.

Regarding the costs involved in the composition of the condominiums the research had difficulty in establishing percentile that demonstrated in a specific way the expenses of the condominium due to lack of documentation that proved the costs month to month of at least three years. In spite of the little documentation obtained they were made some considerations related to the composition of the costs.

The costs involved with the maintenance of the constructions possess two groups that are: the group one that corresponds to the highest costs and that it is related to the employees' payment (cleaning, gardening, among other) electricity and water according to table 3. The group 2 that includes the taxes, security of the construction and extra expenses. the last group doesn't exceed 10% of the total value of the condominium.

Kind of Cost	Incidence (%)
Equipmetns, taxes, insurance	7,15
Eletricity	14,28
Employees	21,43
Water	57,14

Table 3 Maintenance Costs involved in the condominium

The main consideration of the research consists in the fact that construction maintenance is practically inexistent; maintenance can be verified only for elevators because exists the compulsory nature in its execution, but even having the obligation 50% of the interviewees' didn't know to inform which was the periodicity of it. The same happens with the neatness of water tanks, considered a basic maintenance and of easy execution it was not possible knowing the periodicity of its execution. Many items approached are only submitted the maintenance (repair) when they present some problem that stops its usage. An example is the items facade painting, electric charge, piping of gas, water and sewer.

In the visits it was possible to verify painting problems in facade, detach of the plaster, broken blinds and detach of tablets are quite frequent, but they are not a residents´ concern, it is considered just as " detail " being gotten better hereafter. Those residents prefer to wait so the constructive element always deteriorates a "little more" for only then to make possible some type of maintenance activity. Those problems when appear don't configure as an isolated pathological manifestation but they can, with the time, result in very larger problems as, for instance, infiltrations. It is necessary to point out that constructions with problems, cause a terrible aspect unassuming the depreciation of the property in the market

There is urgency in modifying the mentality that preventive maintenance is a cost for the condominium. The activities of periodic maintenance can prevent future problems and soften possible damages to the construction prolonging the useful life of the property and maintaining the same valued in the market. That will only be possible if the condominium has a registration of all their expenses month to month so it can be identified possible extra cause of expenses and causes of reappearance of problems.

In this direction, the planning and control of the activities correlated with the needs and the users' desires are fundamental for the objective being reached and the maintenance is accomplished in the foreseen time.

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5 REFERENCES

BEZERRA, J, E, A. *Um estudo da manutenção predial na gestão terceirizada dos condomínios com base na TPM*. Dissertação (mestrado em Engenharia de produção). Universidade de Santa Catarina. Florianópolis: UFSC. 2000.

BRAZILIAN ASSOCIATION OF TECHNICAL NORMS. NBR 5674. *Manutenção de Edificações - procedimento*. Rio de Janeiro. 1999.

CALMON, J,L., GRILO. Falhas externas em edificações multifamiliares segundo a percepção dos usuários. In¶ *Encontro Nacional de Tecnologia do Ambiente Construído.* Salvador, Bahia. 2000.

JOHN, V.M. *Avaliação da durabilidade dos materiais, componentes edificação – emprego do índice de degradação*. Dissertação (mestrado em Construção). Universidade Federal do Rio Grande do Sul. Porto Alegre, RS: 1987.

LICHTENSTEIN, N.B. *Patologia das Construções: procedimentos para formulação do diagnóstico de falhas e definição de conduta adequada à recuperação de edificações*. Dissertação (mestrado). Escola Politécnica da Universidade de São Paulo. São Paulo: EPUSP. 1985.

MEIRA, A .R. *Estudo das variáveis associadas ao estado de manutenção e satisfação dos moradores de condomínios residenciais*. Tese (doutorado em Produção). Universidade de Santa Catarina: Florianópolis, SC: 2002.

OLIVEIRA, M.C.G. Os *fatores determinantes da satisfação pós-ocupação de usuários de ambientes residenciais*. Florianópolis, SC, 1998. 182 pp. Dissertação de Mestrado. Universidade Federal de Santa Catarina.

PERSIKE, I,H. *Metodologia para manutenção do patrimônio edificado partindo do diagnostico de causas prováveis e indicando sua terapia*. Disponível em http://www.ecv.ufsc.br . Acessado em 07/09/2003

PEREZ, A .R. *Manutenção dos edifícios*. In: tecnologia das Edificações. São Paulo: Ed. Pini - IPT 1988. pp. 611-614

REIS, A . Relação entre níveis de manutenção, limpeza, personalização, aparência, satisfação e principais alterações realizadas nas habitações. *In: Encontro Nacional de tecnologia do Ambiente Construído*, 7., 1998 Florianópolis: UFSC, 1998. pp. 597-604.

RODRIGUES, M. *Manutenção industrial em Curitiba e cidades circunvizinhas: um diagnóstico atual*. Curitiba, PR, 2003. 150 p. Dissertação de Mestrado. Centro Federal de Educação Tecnológica do Paraná.

SANTOS, A .O . *Manual de operação das edificações residenciais: coleta de exemplares e avaliação de seu conteúdo frente às diretrizes da NBR 14.037/1998 e segundo a perspectiva dos usuários*. Dissertação (mestrado em Construção). Universidade Federal do Rio Grande do Sul: Porto Alegre, RS. 2003.

VASCONCELLOS, L.A .C. *Planejamento da manutenção predial visando à pós entrega*. Disponível em http://www.infohab.org.br. Acessado em 07/09/2003

Integrated diagnostics using advanced in situ measuring technology

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ABSTRACT

This work presents a diagnosis protocol about building materials and their decay patterns, incorporating quality control characteristics. This protocol, which is demonstrated in practice on the historic building of the National Archaeological Museum in Athens, Greece, can be useful for scientists and agencies such as a Ministry of Culture. In situ NDT&E approach for pathology studies like Infra-Red Thermography (IR-Th), Fiber Optics Microscopy (FOM) etc, along with the application of instrumental techniques in lab like Mercury Intrusion Porosimetry (MIP), Fourier Transform Infrared Spectroscopy etc, are proved to be advanced diagnostic tools regarding building materials characterization, environmental impact assessment, as well as previous conservation interventions assessment. Furthermore, the collected materials and decay data can be recorded on the building scale using CAD and/or GIS, attempting the development of a planning methodology for monitoring the decay and the durability of building materials and finally buildings life cycle control and management.

KEYWORDS

Building materials, decay, diagnosis protocol, NDT & E

1 INTRODUCTION

In previous works, the necessity of compiling a diagnosis protocol as part of an integrated methodology for the conservation management of historic buildings is indicated [Togkalidou *et al.* 2003]. It is also demonstrated that the planning and programming of the diagnostic campaign under specific standards, which are also enhanced by the principles of quality management, can lead to the increase of buildings lifetime. It is obvious that there is a growing need for the determination of the durability and the service life data of historic buildings. A diagnosis protocol will give to the scientific community common based criteria for the steps and methods required about materials characterization and decay diagnosis, as well as for the evaluation process of former conservation materials and interventions [Moropoulou *et al.* 2004]. In parallel, the level of the offered services by public and/or private agencies that undertake conservation management of historic buildings will be better safeguarded under a well organized integrated scheme, which will develop the contemporary socio-economic web of conservation/restoration works [Terje N., 2002].

In this work, a diagnosis protocol about building materials and their decay patterns, which incorporates the above mentioned characteristics, is presented. This protocol can be applied by agencies that perform maintenance management of numerous buildings of historic importance such as a Ministry of Culture. Moreover, the suggested protocol is demonstrated in practice on the historic building of the National Archaeological Museum in Athens, Greece.

2 RESULTS & DISCUSSION

The determination of building materials service life and durability is a long-lasting problem that has been addressed, for many years now, to scientific community and agencies that perform buildings maintenance management. Unfortunately, in cases of historic buildings where the authentic materials and many times older conservation materials are not industrially reproducible (like contemporary ones) such a determination is even more complicated. Therefore, there is a high necessity that the collected data about the serviceability and the critical performance characteristics of historic building materials to be comparable. This can be achieved only when the same methodology procedure is followed.

A diagnosis protocol is presented in figure 1, attempting to incorporate all the necessary steps. In the first required step, where the input data is compiled, archive material regarding historical, architectural, structural data, as well as environmental conditions data like temperature, relative humidity changes, wind frequency and direction, rain frequency, frost frequency, sun radiation, presence and concentration of pollutants and suspended particles, must be collected. Second step is the building inspection, where manuals and check lists are filled in by the experts taking under consideration the building conservation state and the necessity of conservation interventions application [Moropoulou *et al.* 2003]. When the building inspection is completed, a report is compiled, where except from the check lists, macroscopic observations accompanied by extended photographic documentation are delivered along with a decision about the necessity of materials characterization and decay diagnosis. If the materials diagnostic campaign is necessary, researchers will proceed to the selection of the most representative investigation areas of the building for the application of non destructive testing and evaluation techniques, as well as for the sampling to occur. In addition, recording of the characteristics of the investigation areas like: surface type (masonry, stone, plaster, smooth, relief, and decorative plaster), dimensions, location on the building, previous interventions etc is vital. Details concerning the NDT&E techniques and the equipment that will be used, as well as the corresponding standards (whenever they exist) should also be reported. Sampling should be performed at various heights and depths in order to investigate all the materials in use, as well as the presented types of deterioration. Routine NDT&E techniques that could be used are: Fiber optics microscopy (FOM) for the investigation of materials surface morphology, texture,

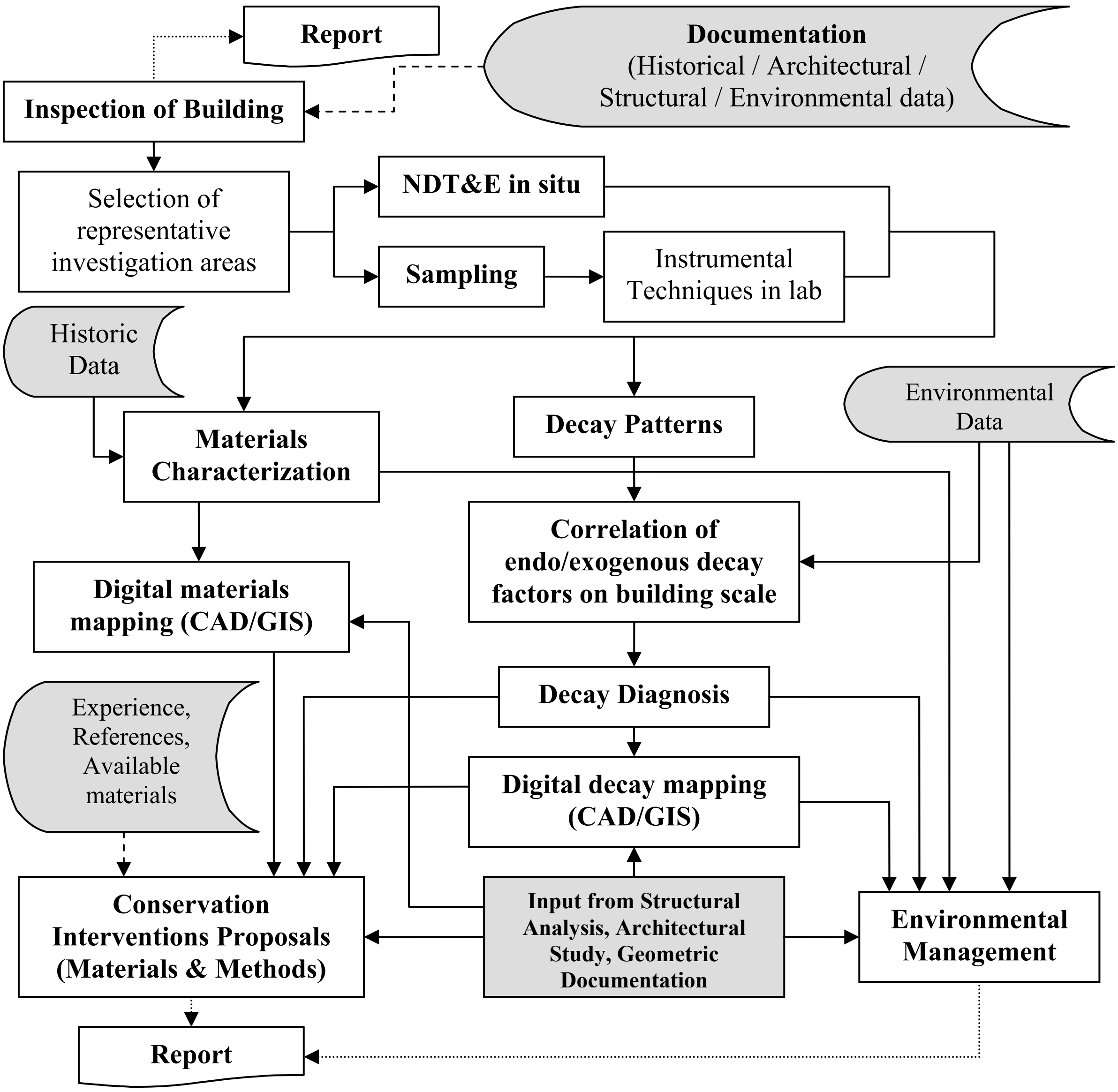

Figure 1. Flow sheet representing the proposed Diagnosis Protocol

microstructure, classification of superficial decay patterns etc. Ultrasonic tests (US) for the weathering determination of the material(s) – structure(s) (cracks, discontinuities etc.), using the direct or the indirect method, depending on the case. Infrared thermography (IR-Th) measures the thermal radiation emitted by the material – structure being examined and renders the image of the surface area in colors, in relation to a temperature scale in order to obtain information about the materials compatibility, moisture phenomena and deterioration of materials. Ground penetrating radar (GPR) for the determination of the layers thickness in a structure and their interfaces location. Colorimetry is used to assess the aesthetical modifications that the materials undergo due to environmental impact and/or conservation interventions.

Supplementary use of several instrumental techniques in lab on samples completes the determination of materials and decay patterns regarding mineralogical and physico-chemical characteristics. Some of the most often applied techniques are: X-Ray Diffraction (XRD) for the identification of the crystalline compounds. Fourier Transform Infrared Spectroscopy (FTIR) for the qualitative identification of the compounds presented. Thermal Analysis (TG/DTA) for the qualitative and quantitative determination of the presented compounds. Mercury Intrusion Porosimetry (MIP) for the determination of materials micro-structural characteristics. Scanning Electron Microscopy with Energy dispersion by X-ray analysis (SEM-EDX), for mineralogical investigation, and semi-quantitative element analysis. The elaboration of the results of all the above mentioned techniques leads to two of the most important outputs of the diagnosis protocol, the materials and decay patterns characterization.

Moreover, the combination of materials and decay patterns characterization with the recorded by the first step environmental data, can lead to the correlation of the endogenous and exogenous decay factors on the building scale; a very important step as far as the integration of the decay diagnosis is concerned, in terms of materials durability and serviceability. Therefore, decay diagnosis is another major output of the diagnosis protocol, since it incorporates the materials data, the decay patterns data and the environmental data. Another important issue concerning the recording of materials characterization and decay diagnosis on the building scale is the digital mapping of materials and decay on computer aided drawings (CAD). The architectural drawings of building façades in CAD can function as the base map, where each material/decay pattern is represented by a different layer and displayed by a different color, depending on the acquired NDT and instrumental techniques results. Therefore, the materials/decay patterns of the façades can be displayed on screen classified according to their surface macro/micro-morphology and physico-chemical characteristics [Moropoulou, A., Delegou E.T., 2003]. Furthermore, the digital mapping can be accomplished using a geographical information system (GIS) which presents more advantages in relation to CAD. Firstly, the calculation of each layer area (m^2) is by far easier using a GIS (where also each layer can represent a particular material and/or a decay pattern). This results in the environmental impact assessment in terms of quantification, since the comparison of each decay pattern extent with the orientation of the building facades is possible. Secondly, another advantage of GIS comparing to CAD is the potentiality of building relational databases; that is the potentiality of managing a multidisciplinary database, which will include environmental, functional, material and structural data.

Additionally, an integrated environmental planning can be accomplished using a GIS by the analysis of all the relevant data with the objective of eliminating the negative impact of the natural and artificial (man-made) environment on historic buildings and complexes. Integrated environmental planning, as one more output of the diagnosis protocol, serve also the purpose of preventing additional damage and preserving the desired conservation state of the historic site; while this site is revitalized in the course of further treatments–uses and its cultural role is reinforced [Moropoulou, A., Delegou E.T., 2003]. It is worth mentioning though, that the processes that embody CAD and GIS are a multidisciplinary task and require the collaboration of architects, surveyor engineers, chemical engineers etc, meaning that the input of their analysis is necessary.

In conclusion, an integrated conservation interventions scheme can be proposed as final output, taking into account though the scientists' experience, bibliography and conservation materials availability. Furthermore, the collection of all the above mentioned data compiles the final report.

Case study: National Archaeological Museum

The historic building of National Archaeological Museum is located in Athens centre, a city of polluted urban atmosphere. Its construction started in 1866 based on the drawings of the German architect Ludwig Lange, and ended in 1889 under the supervision of another German architect, Ernest Ziller, who made major modifications on the Lange's initial drawings. National Archaeological Museum western façade is consisted of several types of plasters and pentelic marble surfaces (figure 2a). It was a historic building with a high necessity for materials characterization and decay diagnosis, as well as of high priority for conservation interventions application, because of the Athens Olympic Games last summer. The following results report the characterization and the decay diagnosis of the purple masonry plaster according to the suggested protocol. The presented material is located on the masonries of propylaem, as well as on north and south galleries of western façade. Purple masonry historic plaster underwent in 1994 a partial reconstruction, where no materials data recording took place for the archive of the Greek Ministry of Culture. A new plaster was constructed on the lower

Figure 2. (a) Photograph of the main entrance (west façade) of National Archaeological Museum; (b) photograph of a plaster investigation area

parts of the masonry and a new purple paint was also applied. Furthermore, in 1998 a protective coating was applied on both types of the plaster, a material of organic composition specialized in wood protection. During the building inspection major discolorations were observed on both new and historic plaster due to the bad conservation state of the protective coating (fig. 2b). Three different types of the coating state of conservation were observed macroscopically and verified using FOM: (a) cohesive interface of protective coating to substrate (fig.3a); (b) loose interface of protective coating to substrate due to fracturing and detachment, presentation of a yellowish hue (fig.3b); (c) friable protective coating which presented a whitish hue. Seven different investigation areas selected where both new and historic plaster areas were included.

Figure 3. Fiber optics microscopy (FOM) images displaying the different conservation states of the protective coating [magnification: (a) x50, (b) x100, (c) x100]

FTIR results demonstrated that the initial protective coating was a poly (vinyl butyral) polymer (PVB), a terpolymer of vinyl acetal, vinyl alcohol and vinyl acetate monomers. During its degradation process (PVB) hydrolyzes into PVAL poly (vinyl alcohol) [Horie, 1987]. FTIR results also showed that the friable whitish coating had been completely hydrolyzed from PVB to PVAL, whereas the yellowish one displayed a similar spectrum with the original material [Moropoulou *et al.*, 2002]. The purple color on the historic masonry plaster (dated back to the 1889), as FTIR and SEM-EDX results indicated, is consisted of the inorganic pigment of Fe_2O_3 (natural earth), whereas the filler is consisted of calcium carbonate. The purple color of the new masonry plaster presented similar chemical composition to the authentic one except from the presence of cadmium (Cd) and sulfur (S) proving that is a cotemporary industrial product, since CdS gives a bright red hue and it is widely used by paint industry. It is worth mentioning though, that a colour difference was macroscopically observed between new and historic color whereat cohesive protective coating presented. Colorimetry measurements on both investigated areas verified this observation, since a relatively high total color difference between them was pointed out (ΔE=6.2 based on the CIELab Uniform Colour Space) [Moropoulou *et al.*, 2002].

Infrared Thermography discloses temperature readings and their distribution on the examined architectural surfaces by the rendering of different colors. The investigation area of the historic plaster (figure 4) revealed that the areas of fractured, detached and loose coating rendered by yellow, red and mauve colors respectively presented higher temperatures (temperature readings of 25°C to 24.2°C) than the areas of cohesive coating which were depictured by blue and black colors (temperature readings of 22.53°C to 21.73°C). This is attributed to moisture withholding deriving from the protective coating application (an organic material specialized in wood protection) that prevents plastered surfaces' "breath" constraining their vapor permeability. Therefore, protective coating presented total physicochemical incompatibility to the substrate, apart from the already mentioned aesthetical problems [Moropoulou et al., 2002].

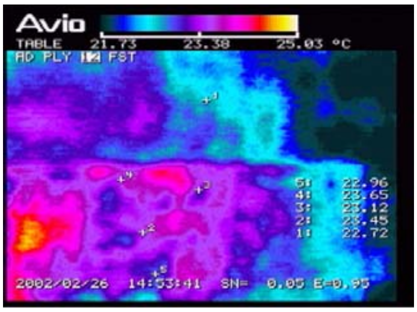

Figure 4. Infra-red thermography image of historic plaster investigation area

In depth sampling and instrumental techniques were performed for the identification of the plasters materials. In particular, both historic and new plaster finishes were consisted of lime binder with calcareous aggregates, except for a small amount of cement presence in new plaster binder. Historic plaster second layer was lime based with calcareous aggregates which contained aluminum-silicate admixtures. The presented binder aggregates ratio was 1:2 as size distribution analysis demonstrated. New plaster second layer was lime-cement based with calcareous aggregates, while binder aggregates ratio was also 1:2. Representative results of MIP and Tg/DTA of both materials are displayed on Tables 1 and 2 respectively [Moropoulou *et al.*, 2002].

Sample	*Cum. Vol. (mm^3/g)*	*Bulk Dens. (g/cm^3)*	*Total Poros. (%)*	*Av. Pore Rad. (μm)*	*Sp. Surf. Area (m^2/g)*
new plaster (2nd layer)	112.3	2.10	23.52	0.65	1.78
historic plaster (2nd layer)	145.2	1.98	28.73	1.15	1.25

Table 1. Representative MIP results of both new and historic plaster

Sample	*Weight loss (%) per Temperature interval (°C)*			
	<120	*120-200*	*200-600*	*>600*
new plaster (finish)	0.21	0.17	1.68	31.64
new plaster (2nd layer)	0.45	0.35	2.37	36.22
historic plaster (finish)	0.1	0.09	0.95	41.15
historic plaster (2nd layer)	0.09	0.02	0.65	42.09

Table 2. Representative Tg/DTA results of both new and historic plaster

Further analysis showed that soluble salts concentration values were under the critical thresholds on both examined materials, besides the presence of gypsum in the most examined samples of historic plaster finish [Moropoulou *et al.*, 2002].

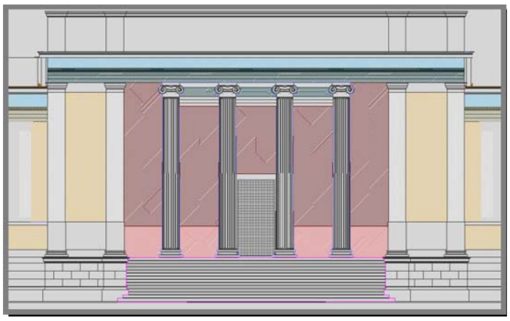

Figure 5. Materials Mapping in CAD

As already explained (see diagnosis protocol description), digital materials/decay mapping performed using CAD (figure 5) and GIS (figures 6 & 7) based on the acquired data by NDT and instrumental techniques. In figures 5 and 6, where materials digital mapping is presented by CAD and GIS respectively, the historic masonry plaster is depicted by purple color, while the new one is rendered by pink color. The area extend of each investigated material was calculated by the means of GIS. Historic plaster area was 248.56m^2, whereas new plaster area was 13.63m^2.

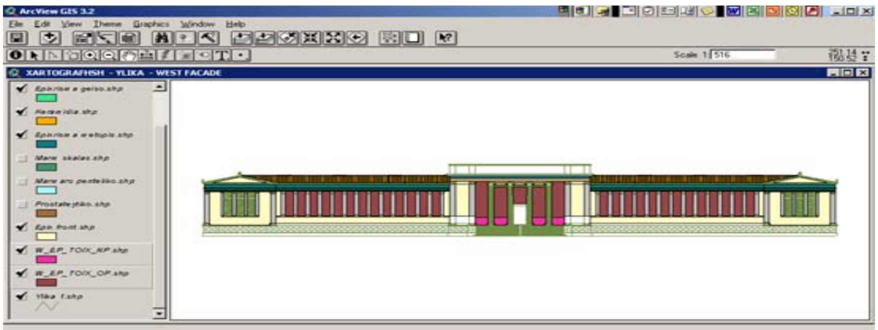

Figure 6. Materials Mapping in GIS

In figure 7, the decay mapping of the examined surfaces is presented by GIS. Brown color depicts areas of coating total detachment and intense fracturing (total area on west façade: 25.56m^2), whilst the blue color represents the areas of coating loose interface to the substrate (total area on west façade: 219.72m^2).

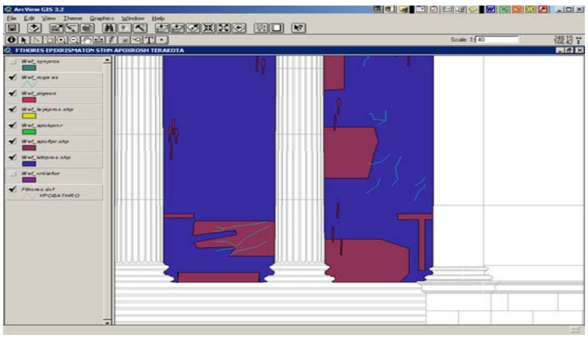

Figure 7. Decay Mapping in GIS

Finally, the suggested conservation interventions about the investigated areas can be summarized as: the protective coating, besides its incompatibility to the substrate, had totally lost its critical performance characteristics and serviceability; thus it should be removed. After its removal, experts will see if there are any discolorations on the historic and new color (colorimetry measurements could be proved very helpful on that matter) and they should decide if a new protective coating or a new color must be applied. In the case of no further discoloration being present, a siloxane based protection coating could be applied, which is physico-chemical compatible with the underlying materials. If the investigated colored surfaces resemble patchwork both new and historic colors must be removed and a brand-new can be applied using the marmorino technique [Moropoulou *et al.*, 2002].

3 CONCLUSIONS

The introduction of an integrated methodology for materials characterization and decay diagnosis, through the proposed diagnosis protocol is accomplished, although further analysis and more criteria may be required. This protocol can be applied by agencies that perform maintenance management of numerous buildings of historic importance such as the Ministry of Culture. It could be proved helpful for the scientific community as well in the attempt of establishing common evaluation criteria about building materials durability and serviceability. In situ NDT&E approach for pathology studies, along with the application of instrumental techniques in lab, are proved to be advanced diagnostic tools regarding building materials characterization, environmental impact assessment, as well as previous conservation interventions assessment. Furthermore, the collected materials and decay data can be recorded on the building scale by incorporating CAD and/or GIS. Finally, the presented CAD/GIS mapping can develop a planning methodology for monitoring the decay and the durability of building materials, attaining management and control of buildings life cycle.

4 REFERENCES

Togkalidou, T, Moropoulou, A., Karoglou, M., Padouvas, E. & Chandakas, B. 2003, ‘System for conservation management of Historic Buildings incorporating quality control principles’, Proc. Innovative Technologies and Materials for the Protection of Cultural Heritage. Industry, Research, Education: European Acts and Perspectives, Technical Chamber of Greece, Athens, Greece, 16-17 December, in press.

Moropoulou, A., Bakolas, A., Togkalidou, T., Karoglou, M., Kaouri, P. & Chandakas, B. 2004, ‘Performances and durability of materials for monitoring and quality control (the case of Municipal market of Pyrgos)’, Proc. Scienza e Beni Culturali XX, ed. G. Biscontin, G. Driussi, Publ. Arcadia Ricerche, Bressanone, Italy, 14 17 July 2004, pp. 765-774.

Terje N., 2002, ‘The economic and management challenges of present maintenance policies of the European built environment-The need for improved management systems’, 7th Cultural Heritage Network Hypermedia, Managing Monuments and Archaelogical Sites, Milan, Italy.

Moropoulou A., Chandakas B., Togkalidou T., Karoglou M., Padouvas E. 2003, ‘A new methodology for quality control and monitoring of historic buildings: A tool for lifetime engineering’, Proc. 2nd International Symposium, ILCDES 2003, Integrated Life-time Engineering of Buildings and Civil Infrastractures, Kuopio, Finland, December 1-3 2003, pp.269-274.

Moropoulou, A., Delegou E.T., 2003, ‘Innovative technologies and strategic planning methodology for assessing and decision making concerning preservation and management of historic cities’, Proc. 7th International Symposium of the Organization of World Heritage Cities, eds. A. Moropoulou, E. Kollias, G. Papatheodorou, Rhodes, Greece, 6 10 September 2003, CD-Rom.

Horie, C.V., 1987, *Materials for Conservation, Organic consolidants, adhesives and coatings,* Butterworths, Kent, England.

Moropoulou, A., Koui, M., Delegou E.T., Bakolas, A., Karoglou, M., Rapti, E., Kouris, S., Mauridis, Th., Ypsilanti, E., Avdelidis, N.P., Ntae, D., 2002, *Conservation Interventions Planning for the main façade of the National Archaeological Museum*, Lab of Materials Science and Engineering, Final Research Working Paper, National Technical Uni. of Athens, August.

Reliability of nondestructive tests for on site concrete strength assessment

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ABSTRACT

Concrete is an inhomogeneous materials and even if a uniform distribution of its component is assumed it is very difficult to develop a model to correctly evaluate its on site mechanical behavior. Also compressive tests of concrete cores gives results affected by uncertainty and strongly dependent on reference standard used.

Several nondestructive testing methods have been developed in the past for on site concrete strength assessment. Among them rebound hammer and ultrasonic pulse velocity (UPV) tests are the most commonly used in practice though their reliability and usefulness is quite controversial. A good calibration of the methods it is only possible if a good knowledge of the concrete properties is already achieved, i.e. it is necessary to use some destructive tests to obtain such information. When assessing wide inhomogeneous structure such limit could be crucial from an economical and practical point of view.

An improvement of the reliability of nondestructive tests could be obtained by their combination as well as in the SONEB method. Basing on a wide number of experimental determinations under laboratory condition different regression models have been here proposed. A poor correlation between UPV and concrete strength was evidenced. Better results was obtained for rebound hammer test. It was evidenced that a preliminary knowledge of concrete characteristics it is of great importance to optimize regression model. A coefficient of determination higher than 0.9 was obtained by using a combined method.

KEYWORDS

Concrete strength, nondestructive tests, ultrasound, rebound hammer, cores

1 INTRODUCTION

The increasing age of reinforced concrete structures over all the world has led to a growing demand for reliable tools for concrete degradation assessment. The wide extension of some of these structures requires such assessment tools to be economical and easy to use. Concrete is an inhomogeneous materials and even if an uniform distribution of its component is assumed it is very difficult to develop reliable predictive models to correctly evaluate its on site mechanical behavior. Several efforts in such sense have been however done by researchers and some models are available on literature [Bazant & Becq-Giraundon, 2002; Bazant *et al.* 2002; Ince *et al.* 2003; Yang & Huang 1996]. Several nondestructive testing methods have been developed in the past for on site concrete strength assessment, a comprehensive literature survey can be found in ACI Committee 228 [1988], Malhotra & Carino [1991], Bungey & Soutos [2001], Tay & Tam [1996]. Among them the rebound hammer and the ultrasonic pulse velocity (UPV) tests are the most commonly used in practice. Even if these methods are standardized their reliability, also taking into account all limits and suggestions given in the standards, is quite low. A good calibration of the testing methods it is only possible if a good knowledge of the concrete properties is already available, i.e. it is necessary to use some additional destructive tests to obtain such information. When assessing wide inhomogeneous structure such limit could be crucial from an economical and practical point of view.
Results from compression tests of concrete cores could be used as a reference for the above mentioned nondestructive tests. However it is well known that compression tests themselves are affected by a number of factors which result in some uncertainty in the concrete strength determination. Besides, measured concrete compression strength is strongly dependent on testing standards adopted and in some case far from the intrinsic strength of concrete, if an intrinsic strength of concrete could be defined [Neville, 1996].
Aim of the present work was to evaluate the reliability of rebound hammer test and UPV test on concrete of different composition and strength class. Basing on a wide number of experimental determinations different regression models of general purpose are proposed by the authors. Coefficient of correlations were however observed to be influenced by concrete samples used (cubes and core with different height to diameter ratio) as well as on concrete information already available.

2 CONCRETE TESTING

2.1 Compressive strength

Compressive strength test is described in several standards. European standards EN 12390-1/3 allows the use of cubic, cylindrical and prismatic specimens, ASTM C39 and C192 standards allows the use of cylindrical specimens. A great importance is given to geometrical size and length to diameter ratio, especially when non standard specimens, as well as concrete cores, are used. The influence of length to diameter ratio is well documented and related to the friction between concrete surface and press steel plate as a consequence of the difference Poisson's ratio of the two materials. The effect of end restrain is conventional assumed to be negligible for a specimen with a length to diameter ratio (*l/d*) of 2. For other cases correction factor are recommended. Correction factors however slightly differ from standard to standard. ASTM C42-90 commends correction factor not linearly ranging from 0.87 to 1 for a *l/d* ratio from 1 to 2 respectively, whilst BS 1881 basing on Concrete Society suggestion [1987], recommends correction factor starting from 0,8. A comprehensive analysis of mathematical models to represent the *l/d* strength correction factor is given by Barlett et MacGregor [1994]. The dimension of the specimen may also influence the measured strength and variability. The problem is of particular importance for high strength concrete [Patnaik & Patanaikuni, 2002; Tokyay & Ozdemir, 1997] due to the need to reduce the size of test cylinders in order to account the performance of existing testing facilities. A survey of the most common correction factors used is given in the Concrete Society Technical Report 11/87 [Concrete Society, 1987].

2.1.1 Estimation of cube strength

The estimation from a concrete core of standard cube strength, i.e. the strength of a cube made with fresh concrete, vibrated and cured under standard condition, must account two main factors:

1) the estimation of cylinder strength, i.e. the strength of a standard cylinder taking into account the effect of the length to diameter ratio, the effect of voids and improper compaction as well as the damages induced by drilling.
2) the conversion to an equivalent cube strength using an empiric correction factor.

The Concrete Society [1987] recommends the use of the previously mentioned *l/d* factors plus a further 6% reduction to take into account the effect of a lateral surface obtained by drilling compared to the smooth surface obtained from the mould. A strength reduction of 15% is also included to allow for a weaker top surface zone of a corresponding cast cylinder. The cylinder to cube conversion is obtained multiplying by a factor 1.25, a further 8% difference between vertical and horizontal drilling is also taken into account thus resulting in the following formulas

$f_{is,cube} = \frac{2.5 f_\lambda}{1.5 + 1/\lambda}$ (1) or $f_{is,cube} = \frac{2.3 f_\lambda}{1.5 + 1/\lambda}$ (2) respectively where $f_{is,cube}$ is the estimated in situ cube strength, f_λ is the compressive strength of the core with a $\lambda = l/d$ ratio. It is interesting to note that in this case for λ = 1 the estimated in situ cube strength $f_{is,cube}$ is equal to f_λ. Following the Concrete Society the 'potential' strength (f_{pot}) of a properly cured cube standard specimen is however 30% higher than the actual in situ cube strength, the recommended formulas will change therefore as follows:

$f_{pot,cube} = \frac{3.25 f_\lambda}{1.5 + 1/\lambda}$ (3) or $f_{pot,cube} = \frac{3.0 f_\lambda}{1.5 + 1/\lambda}$ (4) for horizontally or vertically drilled cores respectively.

In the European standard prEN 13791:2003 correction factor for standard (potential) strength to in situ (actual) strength is fixed to 0.85, testing cores with equal length and nominal diameter from 100 up to 150 mm are considered to give a strength values equivalent to the strength value of a 150 mm cube manufactured and cured under the same conditions. Testing cores with a nominal diameter at least 100 mm and not larger than 150 mm with a length to diameter equal to 2 are considered to give a strength value equivalent to the strength value of a 150x300 cylinder manufactured under the same conditions. A two step method for converting a concrete core compression test result to the in situ strength of the corresponding volume of concrete core and a survey of the different correction factors available on literature is reported by Barlett & MacGregor [1997].

2.2 Rebound hammer

The test is described in ASTM C805 and EN 12504-2:2001 and is classified as a hardness test. The simplicity and speed of the test contrast with several drawbacks which can lead to misleading or useless results. The results of rebound hammer are significantly influenced by several factors [Malothra & Carino, 1991; Bungey & Millard, 1996] such as: smoothness of test surface; size, shape, and rigidity of the specimens; age of the specimen; surface and internal moisture conditions of the concrete; type of coarse aggregate; type of cement; type of mould; carbonation of concrete surface. The influences of the above mentioned factors are so great that it is very unlikely that a general calibration curve relating rebound hammer to strength, as provided by the equipment manufacturers, will be of any practical values.

A calibration curve for each concrete under testing have to be performed, taking into account the specimen condition, as well as a frequent check on a standard steel mass to verify spring performance or friction problem between the impacting mass and the plunger. EN 12504-2 standard recommends that calibration have to be performed on samples properly clamped between the plate of a testing equipment under a compressive load of about 15% of concrete strength and in any case higher than 7 N/mm^2. According to prEN 13791:2003 standard rebound hammer test with calibration by means of cores test may be used for assessment of in situ strength. In situ strength can be estimated using a basic relationship and a determined factor for shifting the basic relationship curve to take into account of the specific concrete and production procedure. Calibration for a specific area of the concrete structure under evaluation have to be performed on a region large enough for at least 9 test location for rebound test and for taking out cores to be tested for in situ compressive strength (f_{is}).

2.3 Ultrasound pulse velocity

The velocity of sound in a solid material is a square root function of its dynamic modulus of elasticity and its density. Compressive P wave propagation is characterized by the following expression:

$$V_p = \sqrt{\frac{E_d}{\rho} \frac{1-\nu_d}{(1+\nu_d)(1-2\nu_d)}} \qquad (5)$$

where E_d is the dynamic modulus of elasticity, ρ is the mass density, ν_d is the dynamic Poisson's ratio. Knowing the modulus of elasticity of the concrete, other mechanical properties can be estimated from empirical correlation with it, that is the basic idea of the ultrasound pulse velocity (UPV) test. The methods is based on the determination of the time required for a pulse of vibration at an ultrasonic velocity and generated by a transducer on the concrete surface to travel through the concrete. Since wavelength (λ), which is related to sound velocity by the relationship $\lambda = {}^{V_p}\!/\!_{\phi}$ has a great influence on wave scattering and attenuation in a very heterogeneous medium such as concrete, a great importance have to be done to the choice of ultrasound frequency (ϕ) used. Transducers with natural frequency between 20 kHz and 150 kHz are the most suitable for use with concrete. The test is described in ASTM C597, BS 1881-203:1986 and EN 12504-4:2004, a critical comparison of several standards from different countries is given in a review paper by Komlos *et al.* [1996] who showed that, despite the common basis of the measurement of ultrasonic longitudinal wave velocity, there are differences among the procedures as recommended by different nations, furthermore the inherent uncertainty in the various assessments is so high that, according to him, the assessments are not suitable for many practical purposes. The UPV in concrete is in fact influenced by many variables [Lin *et al.*, 2003, Malothra & Carino, 1991] including mixture proportion, aggregate type and size, age of concrete, moisture content, etc.., furthermore some factors significantly affecting UPV might have little influence on concrete strength as well as, for a constant w/c ratio, the aggregate content. The most simple and generally accepted relationship between concrete strength and ultrasound velocity has the following form [Bungey & Millard, 1996]:

$$f_c = Ae^{BV} \quad (6)$$

where A and B are constants, f_c is the concrete cube strength and V the ultrasound velocity. A proper calibration of the basic curve on the concrete under evaluation have to be done in order to use UPV method for concrete strength assessment. According to prEN 13791:2003 UPV method with calibration by means of cores test may be used for assessment of in situ strength. In situ strength can be estimated using a basic relationship and a determined factor for shifting the basic relationship curve to take into account of the specific concrete and production procedure. Calibration for a specific area of the concrete structure under evaluation have to be performed on a region large enough for at least 9 test location and for taking out cores to be tested for in situ compressive strength (f_{is}).

2.4 Combined method

The reduction of the influence of several factors affecting rebound hammer test and UPV method could be partially achieved by using both methods together. A classical example of this application is the SONREB method, developed mostly by the effort of RILEM Technical Committees 7 NDT and 43 CND [RILEM, 1994] and widely adopted in Romania [Facaoaru, 1970]. The relationship between UPV, rebound hammer and concrete compressive strength are there given in the form of a nomogram. The improvement of the accuracy of the strength prediction according to Facaoaru [1970] is achieved by the use of correction factors taking into account the influence of cement type, cement content, petrologic aggregate type, fine aggregate fraction, aggregate maximum size. The accuracy of the

estimated strength is considered however to range between 10 and 14 % for a concrete of known strength and between 15 and 20 % when only composition is known.
Several liner and nonlinear multiple correlation equations have been developed and available in literature [Tanigawa *et al.*, 1984; Malothra & Carino, 1991; Qasrawi, 2000; Arioglu *et al.*, 2001]. When non linearity of rebound number R and pulse velocity V is taken into account in the form of a power product (i.e. $f_c = AR^B V^c$ (7) , where A, B and C are constants) the best prediction could be obtained [Arioglu *et al.*, 2001].

3 EXPERIMENTAL

3.1 Materials

Intentionally different concrete samples (cylindrical cores and standard cubes) with different compressive strengths were used for this experimentation. Cores were drilled from the wall of a road tunnel: 39 cores with a nominal diameter of 150 mm (21 cores with a length to diameter ratio of 2 and 18 cores with a length to diameter ratio of 1), 11 micro cores with a length to diameter ratio of 2. Concrete mixture was unknown, aggregate was obtained from scistous rocks, that are generally very laminated and under concrete compression test failed usually along flaking planes. Design concrete strength class was C25, and maximum aggregate size was 20 mm.
Standard cubes were control samples from a precasting factory. Cubes (150 mm side) were cured in water a tested at the age of 28 days. Portland cement type I 52.5 R (in accordance to EN 197/1) and basaltic aggregate with a maximum size of 19 mm was used for concrete mixture. Different cement content allowed to obtain concretes of different strength class, more specifically 17 cube of C55 strength class, 17 cubes of C45 strength class and 5 cubes of C 30 strength class were used for this experimentation. All samples type are summarized in Table 1.
Cores were reduced to standard sizes (l/d=1 or 2) by diamond saw, to reduce the influence of capping all cores load bearing surfaces were diamond ground up to obtain a surface planarity error lower than 20 µm (EN 12390-1: 2002 requires a planarity error lower than 60 µm) . In order to reduce the influence of unknown water content in the concrete, cores were water saturated by dipping into a controlled temperature bath (20 ± 5° C) until constant weight according to EN 12390-7. All samples, cubes included, were tested in saturated surface-dry condition. Concrete mass density was determined according to EN 12390-7.

Sample type	*Sample name*	*Number of specimens*	*Specimen dimension (mm)*	*Cement type*	*Aggregate type*	*Strength class (cubic)*
Cylinder 1:1	C1:1	18	150x150	Unknown	Scistous rock	C25
Cylinder 2:1	C2:1	21	150x300	Unknown	Scistous rock	C25
Cylinder 2:1	MC	11	50x100	Unknown	Scistous rock	C25
Cube	Ku55	17	150x150	Portland I 52.5 R	Basaltic rock	C55
Cube	Ku45	17	150x150	Portland I 52.5 R	Basaltic rock	C45
Cube	Ku30	5	150x150	Portland I 52.5 R	Basaltic rock	C30

Table 1. Summary of the sample used for the experimentation

3.2 Methods

Compression tests have been carried out by means of a 3000kN hydraulic press, precision class 1, with a load rate of 0.5 N·s/mm^2 according to EN 12390-3. In order to evaluate the influence of steel loading plate on concrete lateral expansion restrain, 6 specimens Ku55 and 6 specimens Ku45 have been tested by inserting between plates and concrete surfaces a 4 mm tetrafluoroethylene foil.
Ultrasonic Pulse Velocity test has been carried out on concrete specimens soon after their extraction from a thermostatic bath in a saturated surface-dry condition, using a vertical direct transmission

configuration with 52 kHz transducers. In order to improve transducer to concrete surface contact vaseline grease was used. Tests have been performed according to BS 1881-203:1986.
In order to reduce and to control surface water content specimens, before being tested at the rebound hammer test, have been stored in a thermostatic room at 20 ± 5° C and 90% R.U. Tests have been performed by means of both an analogical and a digital N type rebound hammer. According to EN 12504-2 standard a regular check on a standardized steel anvil have been performed, during testing concrete samples were properly clamped between the plates of a testing equipment under a compressive load of about 15% of their compressive strength. Determinations have been carried out on 12 point, 6 for each of two opposite faces. The mean values of the determination was the rebound number reported in the results.

4 RESULTS AND DISCUSSION

Potential compressive strength of concrete can be directly obtained by testing cubic standard specimens, by using cylindrical cores different correction factors as previously described have to be used. Correction factors related to length to diameter ratio (2 in our cases) and correction factors related to different curing condition of concrete. Mean compressive strength and standard deviation for C1:1 and C2:1 samples were $f_{\lambda 1:1}$= 20.14 MPa, $\sigma_{\lambda 1:1}$=5.81 MPa and $f_{\lambda 2:1}$= 17.42 MPa, $\sigma_{\lambda 2:1}$= 4.35 MPa respectively. The ratio between $f_{\lambda 2:1}$ and $f_{\lambda 1:1}$ was considered as the geometrical correction factor indicated in different standards. In our case such factor was equal to 0.86, that is lower than that one indicated by ASTMC42-90. The actual cubic compressive strength was calculated by means of equation (1) for C1:1 and C2:1 samples, considering a coring direction perpendicular to concrete casting direction, obtaining the following mean values: $f_{is,cube,1:1}$ = 20.14 MPa and $f_{is,cube,2:1}$ = 21.77 MPa respectively. Also potential compressive strength values was calculated in a similar manner by means of equation (3): $f_{pot,cube,1:1}$ = 26.18 MPa and $f_{pot,cube,2:1}$ = 28,30 MPa. For cubic specimens compressive strength f_c was considered of course equivalent to actual strength and to potential strength. If it is taken into concern that restrain effect during compressive tests have a great influence on the result, it could be possible to consider, as mentioned above, that results obtained by testing a concrete cylinder with a length to diameter ratio equal to 2 should be considered as the “true” (under pure uniaxial compression condition) concrete strength. So it was calculated the “true” compressive strength for 1:1 length to diameter ratio cores by multiplying f_y for the above mentioned coefficient 0.86. According to this concept it was also assumed that cubic specimen tested with tetrafluoroethylene foils could give the “true” concrete strength for a cubic geometry (columnar cracks morphology on tested samples supported this assumption). The so defined ratio between true and potential strength was however evaluated to be sensitive to strength class: i.e. 0.65 for C55 concrete and 0.72 for C45 concrete. Non linear regression models based on power products were used to correlated strength concrete to UVP and rebound hammer number (RHN), in order to optimize correlation the variables mass density (MD) and class strength (CS) were also used. Regression models had the general form $y = a + bx_1^c \cdot x_2^d \cdot x_3^e \cdot x_4^f$ where y was the concrete strength (true or potential) and $x_1, x_2, x_3, x_4,$ the quantitative variables UPV, RHN, MD and CS, when present, and a, b, c, d, e, f the model parameters.
Results are summarized in Table 2. The correlation between true or potential compressive strength and UPV was quite poor, with a determination coefficient (R^2) below 0.5 in both cases, the sum of square residuals (SSR) was double for potential compressive strength than for true compressive strength. By taking into account the mass density no improvement of the regression model was observed (a reduction of the R^2 was instead observed). A good result was on the other side obtained by taking into account the concrete class strength (a parameter however frequently unknown) obtaining an R^2 values of 0.97. In this case a sensible increase of the sum of square residuals was observed when potential compressive strength was considered than the true one. An almost negligible improvement of the regression model was achieved including in both cases the mass density variable. Rebound hammer numbers showed a better correlation with concrete strength resulting in a determination coefficient as high as than 0.8. Once again mass density variables had no influence. A very good value for the determination coefficient was obtained taking into account the concrete strength class (R^2= 0.95, SSR=

1040.9). The best regression models were obtained by considering, like SONREB method, both UPV and RHN (R^2= 0.86, SSR= 2946.2) optimized with inclusion of concrete strength class variables (R^2= 0.97, SSR= 635.0). Once again correlation with potential concrete strength was characterized by a worse regression model (R^2= 0.81, SSR= 8424.3 and R^2= 0.94, SSR= 2518.6 for models not including and including class strength variables respectively).

Dependent variables (compressive strength)		Quantitative variables				Model parameters						R^2	Sum of square residuals
True	*Potential*	*UPV*	*RHN*	*MD*	*CL*	*a*	*b*	*c*	*d*	*e*	*f*		
X		X				-224.27	1.67	0.62				0.46	11626.8
	X	X				-703.31	62.05	0.30				0.47	24267.1
X		X		X		-75.82	0.00	1.81	-0.43			0.41	12748.9
	X	X		X		92.48	-770.07	-2.82	2.61			0.17	37947.0
X		X			X	-194.23	13.45	0.20	0.21			0.97	749.7
	X	X			X	-240.15	9.22	0.26	0.23			0.96	1768.8
X		X		X	X	-74.51	0.00	0.34	0.67	0.39		0.97	737.9
	X	X		X	X	-141.15	0.64	0.40	0.06	0.34		0.96	1687.0
X			X			-54.72	1.87					0.82	3809.5
	X		X			-77.12	2.70					0.80	8949.5
X			X	X		-23.96	61.14	1.68	-0.84			0.81	4080.7
X			X	X	X	-52.03	2.74	0.27	-0.06	0.49		0.95	1040.9
X		X	X			-76.30	0.17	0.46	0.70			0.86	2946.2
X		X	X	X		-103.41	0.61	0.42	0.54	-0.01		0.86	3022.6
X		X	X	X	X	-59.50	0.39	0.50	0.22	-0.24	0.41	0.97	635.3
X		X	X		X	-61.14	0.04	0.58	0.13	0.41		0.97	635.0
	X	X	X			933.84	-3197.52	-0.11	-0.11			0.81	8424.3
	X	X	X		X	-36.65	1.47	0.07	0.19	0.78		0.94	2518.6

Table 3. Results of nonlinear regression analysis

The use of NDT for on site evaluation of concrete strength have to be considered very carefully. Also the evaluation on laboratory condition on properly controlled specimens gave not very good results especially when UPV method was used. A good improvement of the correlation between NDT result and concrete strength can be obtained taking into consideration a preliminary knowledge of concrete properties, e.g. class strength or concrete mix design. Unfortunately during on site inspection a limited knowledge of the materials used for the structures is in most cases available. A calibration of the NDT method chosen with compressive tests on cores taken from the structure is therefore imperative. When that is economically possible a great care have to be taken in evaluating compressive concrete strength. Results obtained by NDT methods are strongly dependent on the physical properties and condition of the concrete whilst "potential" compressive strength as evaluated in accordance to standards is no more than a conventional notion adopted for purpose of evaluation and it is quite hard to define an intrinsic, "true", concrete strength that is on the other side more directly correlated with the physical properties of the concrete itself. Results obtained in this experimentation seem to confirm such consideration.

5 CONCLUSION

A laboratory testing program was undertaken to evaluate the reliability of UPV and rebound hammer tests for evaluation of compressive strength of concrete. A nonlinear regression model based on power products was used to correlate experimental results. A poor correlation between UPV and concrete strength was evidenced notwithstanding the well controlled laboratory conditions. Better results was obtained for rebound hammer test. It was evidenced that a preliminary knowledge of concrete characteristics it is of great importance to optimize regression model. The influence of the definition of concrete strength was also evaluated: the use of the "true" concrete strength, considered as the value determined under non restrained condition testing, resulted in a better regression model than when considering the potential compressive strength as defined in the standards.

6 REFERENCES

TT8-227, Reliability of nondestructive tests, E. Proverbio and V. Venturi

ACI Committee 228, 1988, 'In-Place methods for deterioration of strength of concrete, *ACI Materials Journal*, **85[**5], 446-471.

Arioglu, E., Arioglu, N., Girgin, C., 2001, 'A Discussion of the paper "Concrete strength by combined nondestructive methods simply and reliably predicted" by H.Y. Qasrawi', *Cement and Concrete Research*, **31,** 1239-1240.

Barlett, F.M. & MacGregor, J.G., 1994, 'Effect of core length-to-diameter ratio on concrete strengths', *ACI Materials Journal* **91**[4], 339-348.

Bartlett, F.M., 1997, 'Precision of in-place concrete strengths predicted using core strength correction factors obtained by weighted regression analysis', *Structural Safety*, **19**[4], 397-410.

Bazant, Z.P. & Becq-Giraundon, E. 2002, 'Statistical prediction of fracture parameters of concrete and implications for choice of testing standard', *Cement and Concrete Research*, **32** , 529-556.

Bazant, Z.P., Yu, Q. Zi, G., 2002, 'Choice of standard fracture test for concrete and its statistical evaluation', *International Journal of Fracture* **118**, 303-337.

Bungey, J.H. & Millard, S.G., 1996, *Testing of Concrete in Structures*, 3rd edn.Blackie Academic & Professional, an imprint of Chapman & Hall.

Bungey, J.H. & Soutos, M.N., 2001, 'Reliability of partially-destructive tests to assess the strength of concrete on site', *Construction and Building Materials*, **15** 81-92.

Concrete Society, 1987, *Concrete core testing for strength*, Concrete Society Technical Report n: 11 including Addendum, London,.

Facaoaru, I., 1970, 'Non-destructive testing of concrete in Romania, Symposium on NDT of concrete and timber, London: Institute of Civil Engineers, pp. 39-49.

Ince, R., Arslan, A., Karihaloo, B.L. 2003, 'Lattice modelling of size effect in concrete strength', *Engineering Fracture Mechanics*, **70**, 2307-2320.

Komlos, K., Popovics, S., Nurnbergerova, T., Babal, B., Popovics, J.S., 1996, 'Ultrasonic Pulse Velocity test of concrete properties as specified in various standard', *Cement and Concrete Composites*, **18,** 357-364.

Lin, Y., Lai, C., Yen, T., 2003, 'Prediction of Ultrasonic Pulse Velocity (UPV) in Concrete', *ACI Materials Journal*, **100[**1], 21- 28.

Malhotra, V.M. & Carino N.J. (Eds.), 1991, *CRC Handbook on Nondestructive Testing of Concrete*, CRC Press, 1991.

Neville, A.M., 1996, *Properties of concrete*, Wiley, John & Sons,.

Patnaik, A.K. & Patnaikuni, I., 2002, 'Correlation of strength of 75 mm diameter and 100 mm diameter cylinders for high strength concrete', *Cement and Concrete Research,* **32** 607-613.

Qasrawi, H.Y., 2000, 'Concrete strength by combined nondestructive methods simply and reliably predicted', *Cement and Concrete Research*, **30**, 739-746.

RILEM, 1994, *RILEM technical recommendations for testing and use of concrete construction materials*, E & FN Spon.

Tanigawa, Y., Baba, K., Mori, H., 1984, 'Estimation of concrete strength by combined nondestructive testing method', *ACI SP-82*, **1**, 57– 65.

Tay, D.C.K., & Tam, C.T., 1996, 'In situ investigation of strength of deteriorated concrete', *Construction and Building Materials*, **10**[1], 17-26.

Tokyay, M. & Ozdemir, M., 1997, 'Specimen shape and size effect on compressive strength oh higher strength concrete', *Cement and Concrete Research,* **27** [8], 1281-1289.

Yang, C.C., Huang, R., 1996, 'A two-phase model for predicting the compressive strength of concrete', *Cement and Concrete Research* **26** [10], 1567-1577.

Impact on the Design Life of Buildings in a Tropical Hot Wet Environment

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ABSTRACT

The degradation of the built environment is an enormous economic and environmental problem. Knowledge about the exposure environment and its relationship with the degradation of various building materials is very much needed, particularly in the tropical belt as a basis for proper maintenance and service life planning. The rate of degradation is directly affected by the macro, meso and micro environments and these are specific to geographic location. This paper considers the impact of the hot-wet tropical environment in Malaysia on life cycle degradation factors affecting buildings. Particular reference is made to an in depth investigation of a 50 year old hospital undertaken in 1988 in the southern town of Johor Bahru. The hospital had suffered structural and durability failure arising from a lack of timely maintenance intervention. A remedial approach developed based on assumed future usage patterns taking into consideration monitoring and maintenance requirements is highlighted. The need for a better characterization of key environmental degradation factors and the development of a more rational approach to the design are also considered.

KEYWORDS

Hot-wet Tropical Conditions, Servicability – Life Cycle Degradation , Durability Assurance, Concrete Failures

1 INTRODUCTION

There has been an increasing recognition for the need to forecast and control the cost of building and infrastructure ownership, because a high proportion of the life cycle costs of a facility may be set at the time of completion. Service life planning aims to reduce the cost of ownership. An assessment of how long each part of a structure will last; helps to decide the appropriate specification and detailing. When the service life of a structure and its parts are estimated, maintenance planning and value engineering techniques can be applied. Reliability and flexibility of use can be increased and the likelihood of obsolescence reduced.

To safeguard the built environment; action is urgently needed. In principle, there are two possibilities. Firstly, society should try to improve the environment surrounding the materials and secondly, better products, processes, methods and standards should be developed. The first action is being pursued by the environmental research area particularly in the developed economies but not exclusively. In Malaysia for instance there have already been moves in the last 5 years to take a more proactive approach to environmental pollutants. An international standard (ISO 15686-2001 (1) in design life of buildings is currently being developed under ISO/TC59/SC14 and this forms a basis for a more rational approach to service life planning.

A critical requirement for a proper approach to design planning is an understanding of the key environmental degradation factors which affects durability. This coupled with a planned maintenance approach should be the basis for future management of the built environment. There is only a limited consideration of these factors in the tropical belt countries and this paper attempts to put together some real life data for further consideration.

2 CHARACTERIZATION OF KEY ENVIRONMENTAL DEGRADATION FACTORS

The main sources of information used to classify the environment were obtained from the meteorological services and presented in full elsewhere [Gurusamy, K., 2004].

The characteristic features of the climate of Peninsular Malaysia are uniform temperature, high humidity and copious rainfall and they arise mainly from the maritime exposure of the country (see Fig. 1). Climatological records show that the humidity in the Klang Valley and its surroundings remain almost constant throughout the year with annual mean of 83.4% which is 6% higher than in UK and Hong Kong. However, the mean values hide the fact that the humidity is in the range 50-70% at between 7-8 hours daily which is the optimum range of values for carbonation. The annual mean maximum is 98.0% and the annual mean minimum is 55.7%. While there are local variations throughout the country this is not considered significant enough for the assessment of durability.

The mean annual rainfall and solar radiation shown in Fig. 1 are typical for Peninsular Malaysia. The Air Pollutant Index (API) has been regular recorded since 1996. A comparison between the API for Kuala Lumpur (Inland Urban), Johor Bahru (South Coast Urban), Seberang Prai (Industrial) and Kuantan (East Coast Urban) is also given in Fig. 1. Kuala Lumpur the capital city with its high volume of traffic is generally the more polluted environment compared for instance to the Industrial district of Seberang Prai. This is related to the considerably higher levels of vehicular emissions in the Klang Valley. The measure of SO_2 confirms that the worst affected areas are where there is a concentration of industries (ie) in Johor in the South (32 $\mu g/m^3$) and Seberang Perai in North. (23 $\mu g/m^3$). Actual data on Carbon Dioxide concentrations is not presently available however an examination of the carbon monoxide measurements indicates that the urban concentration was 2.5 to 3 times the background concentration during the period 2000 – 2003. The equivalent figures for the

Industrial Environments is 2.0 to 2.5. This does suggest that carbonation damage is likely to be accelerated in the Urban and Industrial locations in Malaysia. This requires further consideration based on field data.

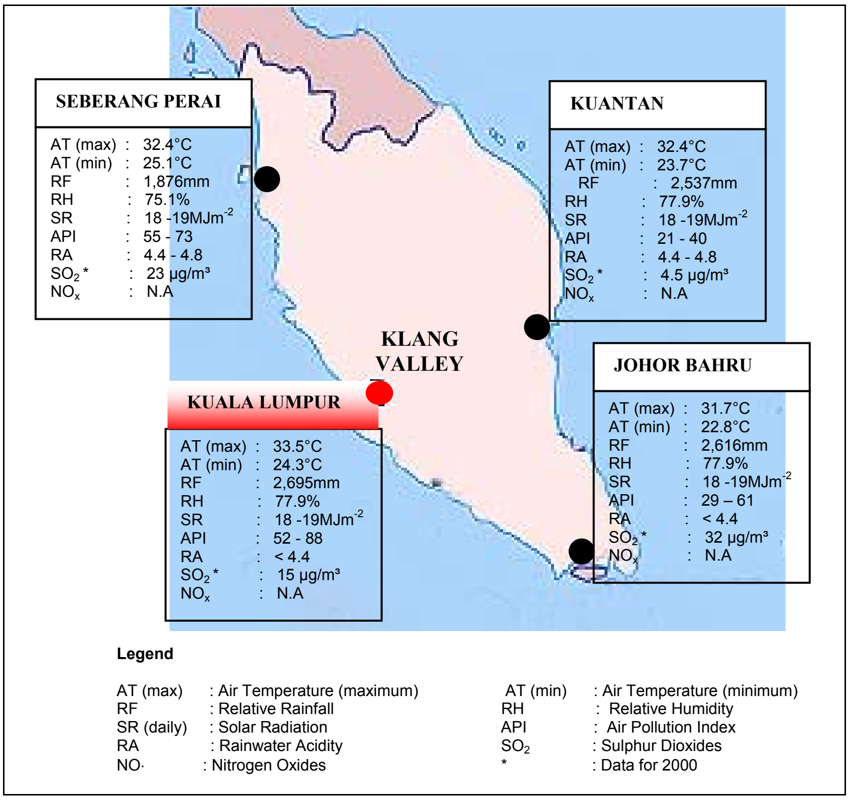

Figure 1: Environmental Characteristics for selected locations, Peninsular Malaysia, 2002.

Peninsular Malaysia is surrounded by oceans (i.e.) the Malacca Straits (West), the Johor Straits (South) and the South China Sea (East). Sea spray may be regarded as a form of particulate pollution which can travel under the influence of wind for several kilometers inland, from coastal areas but rapidly decreases with distance from the coast. This is also the case in Peninsular Malaysia with the most significant effects in the North East and East of the Peninsular as the South and West are much more sheltered by adjacent land masses (i.e.) Singapore and Sumatra respectively.

The general Malaysian environmental characteristics are compared to relevant southern European data [ISO 15686 2004] in Table 1. Generally this indicates that more severe macro environmental conditions for building component deterioration exist locally. The choice of sealants, external cladding materials and architectural finishes in buildings are therefore likely to have reduced service life and requires further systematic documentation and examination.

1	**Temperature**			**ISO 15688-7 from Annex A For Europe**		**Malaysian Climatic Condition**	
	Category 3	Hot	a) Ave. Temperature b) Max. Temperature c) Min. Temperature	> 35ºC - -		NR 33ºC 24ºC	
2	**Rainfall Humidity**						
	Category 4	Very humid	a) Rainfall (mm/year) b) Humidity (average yearly 9 am RH)	>1300mm/year > 80%		1800 -3000 mm/year 75.1% - 84.7%	
3	**UV Radiation**						
				Moderate	Severe	Moderate	Severe
			a) Annual radiation on horizontal surfaces	< 5 GJ/m^2	≥ 5 GJ/m^2	NR	5.8 – 6.9 GJ/m^2
			b) Average temperature of the warmest month of the year	< 22°C	≥ 22°C	NR	≥ 32°C

NOTE: NR - Not Relevant.

Table 1: Comparative Analysis of macro/meso environmental conditions in Malaysia with reference to Annex A Guiding Supplement in ISO 15688-7 given for Europe.

3 CASE STUDY – INVESTIGATION OF A GENERAL HOSPITAL IN JOHOR BAHRU

3.1 General Background

The present author [Robery, P. C., et al., 1988] was closely involved with the Consultancy services for the remedial and rehabilitation works for the main block of the Hospital Sultanah Aminah (HSA), Johor Bahru, which commenced on 1st December 1988. The hospital is located in Johor Bahru, approximately ¼ kilometer north of the Straits of Johor (see Fig. 2).

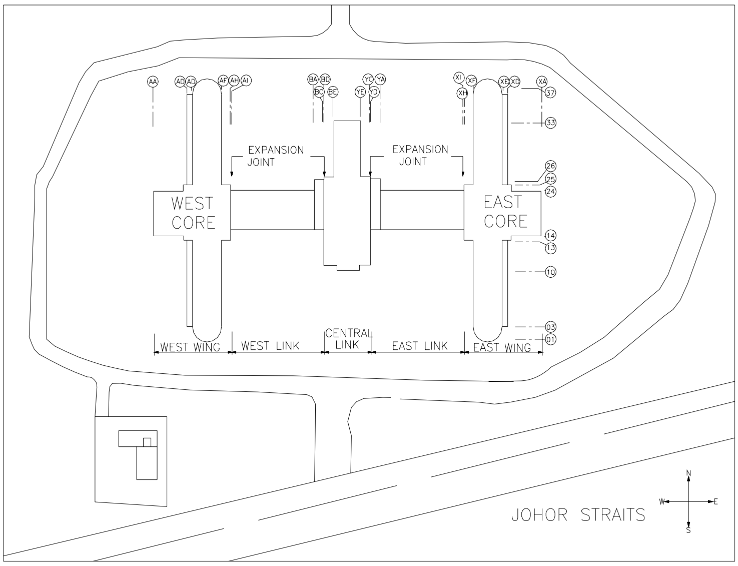

Figure 2: Hospital Sultanah Aminah, Johor Bahru – Location plan at the time of investigation (1988)

The structure is a 6 storey reinforced concrete frame with reinforced concrete pile foundation and a brickwork facade. Construction started in 1938 and was completed in 1941. The ground floor is a reinforced concrete suspended slab below which is a basement. The total cost of the building when the structure was completed in 1941 was estimated to be RM 2,057,075 (USD 541,382). This is equivalent to RM 15,811,900 Million (USD 6,176,520) taking into consideration inflation and the value of money in 1993.

3.2 Initial Overview

The structural investigation involved three distinct areas of interest (ie) the reinforced concrete frame, external brickwork facade and flat roof areas and the soils and external works.

The reinforced concrete columns and beam/slab floors showed some signs of deterioration in the visual form of cracks and spalls. The worst deterioration was in the ground floor soffit beams. It had been reported [PWD, 1987] that cracking in the beams supporting the ground floor was first observed in the 1950's and as a result, a layer of gunite was applied to the beams and the soffit of the suspended ground floor slabs.

The soil condition was considered to have an obvious impact on the stability of the building. The unknown foundation condition, close proximity to the Straits of Johor and original swampy land necessitated a thorough soil investigation. The groundwater drainage system was a significant conduit for sea water ingress in basement areas during high tides resulting in the durability failure of reinforced concrete.

The structural integrity of the building required checking as a result of the reinforced concrete deterioration, changing use/loadings within the building and to account for the differences in design criteria between the 1930's and 1980's. The full details of the structural investigation are given in a previous paper [Gurusamy, K. 2004].

A general view of the building during the remedial works contract is given in Photo 1.

3.3 Durability Considerations

3.3.1 Introduction

A full assessment of material durability is based on a visual examination of selected parts of the HSA with further testing and analysis undertaken to characterize the long term deterioration of different elements of the structure. Only carbonation of structural elements is discussed below. All other elements and chloride ingress mechanisms are discussed elsewhere.

3.3.2 Depth of Cover and Carbonation

As part of the breakout operations and the taking of core samples, the depth of carbonation of the concrete was measured using phenolphthalein ph indicator. These data were compared with the depths of cover at the corresponding locations. The results are summarized below.

Depth of Carbonation

Carbonation of the reinforced concrete proceeded at two rates:

- in the damp, gunite covered basement elements, carbonation depths were negligibly low in well compacted concrete,
- in the dry superstructure, carbonation depths were high and variable.

The exception to the above, was in honeycombed areas of concrete in the ground floor beam soffits. It was noted that in approximately 94% of areas where corrosion had reduced the cross-section of the reinforcing bar, the concrete was severely honeycombed. The bar would consequently have been unprotected from the basement environment since the time of construction. Similar results were found in cores from ground floor slabs, suggesting 80% of the soffit was honeycombed.

Carbonation and Cover in the Superstructure

Average depths of cover measurements were made for the main steel around the superstructure. The data is summarized in Table 2 below.

Element	*Ave. Max Depth of Carbonation*	*Ave. Depth of Cover*
Beam	40 mm	40 mm
Column	40 mm	45 mm
Slab/Canopy	50 mm	19 mm

Table 2: Maximum Depth of Carbonation Vs Average Depth of Cover.

The results showed a significantly lower cover was used in the slabs. Consequently, based on average depths of carbonation, the majority of the slabs had carbonated past the depth of the bar. As these are average values, there are localized areas of corrosion, initiated by greater than average (or less than average) cover.

Carbonation and Strength

Good correlation was found between carbonation of the superstructure and estimated insitu cube strength. For the typical superstructure strength range of 20 – 29 Mpa, a carbonation coefficient was calculated from which the average carbonation rate for that strength range could be calculated. This gave the average depth of carbonation as 51mm after 50 years. In a further 50 years, assuming conditions in HSA do not change (ie, paint types used, change of use/ventilation of rooms), the average depth of carbonation was estimated to be 72mm.

For the beams and slabs at ground floor level, the low depths of carbonation were due to the combined affects of higher compressive strengths (average of 34 Mpa and 40 Mpa respectively), the gunite protective layer and the high humidity which reduced carbonation rates.

Carbonation and Corrosion - Corrosion of the superstructure was not as widespread as would be expected for the high depths of carbonation measured around HSA. This is due to the lack of moisture at the depths of the bar which had restricted the corrosion process. The continued integrity of the structural elements depended upon the concrete being kept free from both water and high relative humidity. This was confirmed by the electrochemical potential data, which showed that areas of damp concrete adjacent to areas of leakage are active and liable to corrode.

Only in the basement environment in areas of honeycombed concrete were widespread areas of corrosion found.

3.3.3 Reinforcement loss of Section – Ground Floor Beams

From the visual survey it was identified that extensive spalling of basement beams had occurred due to reinforcement corrosion. This was due to carbonation of concrete to full depth of steel at the cover zones which was extensively honeycombed. A total of 54 ground floor beams had the bottom main reinforcement exposed to determine the loss of reinforcement section by steel corrosion. Every fourth beam was selected plus other beams which visually exhibited corrosion related defects. The results are shown in Table 3. A total of 42% of the main reinforcement had lost 10% or more of the cross-sectional area.

Loss of Cross Sectional Area	Number of Bars Measured	% of Total
< 10%	53	58 %
10-20%	21	23 %
>30%	15	17 %
	2	2 %
Total	91	100 %

Table 3: Reinforcement loss of section – ground floor beams

3.3.4 Future Usage – Environmental Considerations and Maintenance Approach

The Malaysian environment is characterized by high temperatures (Average 27 °C), and humidity (84% RH) which are fairly constant throughout the year with distinct wet and dry seasons. While the macro environmental factors are important it is the micro-climates specific to the structure which have a significant impact on the durability and service life of the building.

The effects of moisture content, greater than 4%, on reinforcement corrosion was confirmed during the investigation. In the basement east link beams, for example, where serious corrosion was found (up to 25% loss of steel section), the moisture contents were measured at values greater than 10%. This was attributed to the effects of condensation, as the ground floor area was air conditioned and kept at a much lower temperature than the basement.

The effect of leaking services on visual deterioration was evident throughout. These included water stains, rust stains, algae and mould growth, plant growth, brick swelling and cracking, so on. The problem was particularly acute where plumbing was built in and covered by brickwork.

The overall aim, therefore, for future usage of the hospital was to eliminate all forms of wetting of concrete surfaces. This included moisture penetration from whatever source, such as defective brickwork, leaking service pipes, failed roof waterproofing or washing down of porous walls and floors. Structural elements enclosing air conditioned areas would be subject to potential condensation, the critical zone being reinforced concrete members with air conditioning only on one side leading to a temperature gradient sufficient to produce condensation.

By taking a strategic approach to maintenance planning (ie) special attention to control and monitoring at wet areas, it was possible to focus the remedial works on removing potential sources of moisture and water penetration and limiting the extent of structural remedial works to elements which had visible corrosion damage. All other elements which were carbonated beyond cover provisions and potentially in danger of corrosion damage were left untouched with a provision that regular monitoring and maintenance is undertaken. In this way the full cost of the remedial works was capped.

4 COMPARISON OF MALAYSIAN CARBONATION DATA TO EUROPEAN RESEARCH

Considerable data had been gathered as part of a European funded research programme undertaken by Taywood Engineering (M) where a comprehensive review of carbonation data had been undertaken resulting in design life prediction curves. These results for carbonation penetration are summarised in Table 4 and compared to local Malaysian data. The carbonation data discussed from Johor Bahru fits well into the European data range for 30 and 40 MPa concrete for interior exposure. Due to the higher temperatures in Malaysia, approximately 10°C higher on average compared to UK, a doubling in penetration rate would have been expected based on the Arhenuis law. This appears not to be the case most likely due to the higher relative humidities typical of Malaysia and the fact that concrete will not have sufficient time to dry out in the 7-8 hours where the RH is between 50-70% over a 24-hour period. The Malaysian results are however on the higher side of the predicted curve (see Fig. 3). It should also be noted that the rate of carbonation will depend on variability in the local materials used for the concrete production and the efficiency of concrete curing.

Source of Data	**'k' mm(year) ½**		**Time for Carbonation Font to reach steel (years)**		
		Cover Depth	**25 mm**	**35 mm**	**40 mm**
HSA (Grade 30 - 39	5.8[1]		19	36	48
HSA (Grade 40+)	4.9[2]		26	51	67
European data (Grade 50)	2.9[3]		74	145	190
Malaysian data (Grade 50)	4.0[4]		39	76	100

Note: (1) Johor Bahru Data, Hospital Sultanah Aminah
(2) Johor Bahru Data, Hospital Sultanah Aminah
(3) 'k' value based on design curve for European conditions (see Fig. 3)
(4) 'k' value corrected for Malaysian conditions (see Fig. 3)

Table 4 : Depth of carbonation based on diffusion coefficients (k) obtained from insitu testing in Johor Bahru compared to European data .

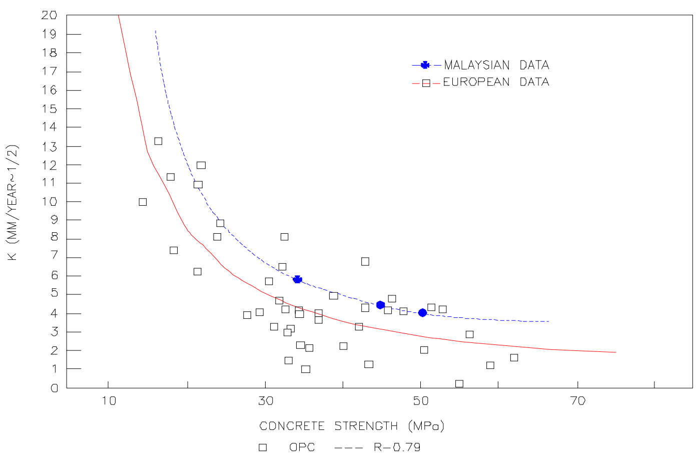

Figure 3: 'k' vs 28 days concrete strength – Interior Environment for European and Malaysian conditions.

5 OVERVIEW OF LIFE CYCLE DETERMINATION

The case study effectively illustrates the Environmental Impact on a 50 year old structure in a coastal hot humid tropical setting. The data gathered confirms the well known effects of carbonation and chloride penetration on reinforcement corrosion. However despite the generally more onerous environmental conditions compared to equivalent data from the United Kingdom the time to damage while accelerated has not increased as expected based on Arhenius Law. In relation to carbonation penetration this appears to be probably due to the higher relative humidities typical of the tropical belt countries and the insufficient time for concrete to dry out. The paper also highlights the importance of timely maintenance intervention to achieve service life expectations. A timely intervention to deal with expected component failure for example of the flat roof areas or leaking down pipes or the application of coatings to protect against carbonation and chloride ingress related corrosion and brickwork damage could have reduced substantially the RM 25 million (USD 6,579,508) remedial and upgrading programme which became necessary in the early 1990’s.

It was also confirmed that Airbourne chlorides were not a problem in this southern coastal region probably because of the sheltered nature of the Johor Straits which has Singapore as a protective land mass to the south. Carbonation of concrete was well beyond cover provisions and this requires consideration during design for unsheltered concrete elements where the availability of moisture will lead to corrosion. Guidance in this regard involves the use of minimum concrete grades and minimum cover provisions based on design life requirements and can be readily be modelled using the data presented herein for the climatic conditions typical of a hot wet tropical setting.

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REFERENCES

BS 8110, British Standards Institution – 1985 "*Structural Use of Concrete*"

CP114 British Standards Institution - 1948 "*British Standard Code of Practice for the Structural use of Normal Reinforced Concrete in Buildings*",

Gurusamy, K. and Browne, R. D. February 1994, "*Life Cycle Cost Design for Buildings & Structures : A Strategic Initiative*", Presented at MIGHT forum on the Construction Industry, Kuala Lumpur, Malaysia

Gurusamy, K. June 1994, "*KL-LRT Precast Viaduct Elements, Long Term Durability For 100 Years Design Life" Report 1303/94/6851*

Gurusamy, K. July 1999, "*Concrete Durability In Malaysia – A Much Needed Reassessment*", Presented at the Cosett'99, Sixth International Conference on Concrete Engineering and Technology Seminar, Kuala Lumpur, Malaysia ,

Gurusamy, K. Sept 2004, "*Environmental Impact On The Design Life Of Building In Malaysia*", Presented at the International CIOB Construction Conference, Kuala Lumpur, Malaysia,

Robery, P. C. , Currie, R. , Gurusamy, K. and Sinclair, T. June 1989, Hospital Sultanah Aminah Johor Bahru, Interim Report No 2, Report 014H/89/4019,

Haagenrud, S.E. and Henriksen, J.F. 19-23 May 1996, *Degradation of built environment – review of cost assessment model and dose-response functions,* Proceedings of the Seventh International Conference on Durability of Building Materials and Components, 7DBMC, Stockholm, Sweden, – Volume 1.

ISO 15686 Buildings and Constructed Assets – Service life planning Parts 1-8, at various stages of publication.

PWD 1987, Construction Unit, Design and Research Branch P.W.D. Headquarters, Kuala Lumpur Preliminary Report on the Defects at Sultanah Tun Aminah Hospital, Johor Bahru (Translated copy), Internal Publication

The Institution of Structural Engineers, July 1980, "*Appraisal of Existing Structures*".

A bridge between old and new.
Thermal window refurbishment of the dwellings in Romania

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ABSTRACT

From the climatic point of view, Romania is a central - eastern European country, with an excessive continental temperate climate, characterized by very cold, snowy winters (with temperatures of -12 ÷ -21^{0}C) and hot, dry summers (temperatures over +30^{0}C). Therefore, the problems of thermal protection, in winter as well as in summer, are taken into consideration, by the authorities as well as by the end-users.

The built heritage of Romania is dominated, at the beginning of the XXIst Century, by the existence of an important percentage of dwellings, with initial low thermal performances, way below the current national and international standards.
Romania has a stock of over 8 million dwellings. Most of them are located in blocks of flats (over 83,799 blocks). 52.5% of the dwellings are built in urban areas. Among them 15.3% were built before 1944, about 76% between 1944 and 1990, and the remaining of 8.7% after 1990[1].

The higher established exigencies, due to the need to reduce the use of traditional fuels, to diminish polluting emissivities and global changes, to increase and diversify comfort requests, as well as to the necessity of ranging to the international norms, lead to the need of upgrading the thermal performance of the envelope of the building. While most preoccupations focus on the thermal improvement of the opaque component of the envelope, it is known that the glazed parts of the vertical envelope are responsible for the most important heat loss. Therefore, the hygro-thermal (and implicitly the acoustic) behavior of the glazed component of the envelope of the buildings (new or existent) is an object of study of acute necessity.

The paper aims to make an analysis of the types of joineries that have been used in the dwellings in Romania, and to present solutions, currently or possibly to be used for the existing buildings.
The measures that can be taken when refurbishing a window, aim to decrease the thermal loss through the glazed surfaces of the envelope. Briefly they consist in:
- the improvement of the performances of the existing joineries
- the replacement of the joineries, with new, performant ones, should it prove to be necessary.

KEYWORDS

dwellings, windows, thermal and acoustic rehabilitation.

1 INTRODUCTION

The construction of these dwellings can be separated into distinct periods, according to the building system that was adopted: at the end of the nineteenth century, Romania had mainly low raise buildings, usually family houses with one or two levels. Either the mansions of the nobility or the merchants' houses with commercial ground floor and dwellings on the upper floor, they have moderate sized windows that are generally equipped with rolling shutters. Built between the two World Wars, the high raise apartment blocks of Bucharest have elegance and style and can be included with their distinct features in the European Modernism. Their generous spaces, large windows and balconies that mark the horizontal line of the storeys, their columns and decorations, the accuracy of the details still make these expensive apartments (Figure 1) interesting and appreciated, although structurally they are not in a correspondent state any more. Most of these apartments have been equipped with shading devices.

Figure 1. "Magheru" Boulevard, an example of Modernist architecture in Bucharest

Changes begin to appear after the Second World War, when cheap mass buildings are more and more visible and invade first the suburbs and then the center of the cities, replacing the old and picturesque traditional housing systems. Between 1950 and 1960 (approximatively) the design was influenced by projects with similar architectural programs carried out in the USSR (buildings of about 4 storeys, with moderate dimension windows, without balconies, grouped around inner courtyards).

The blocks of apartments of the sixties and of the first half of the seventies are built with reinforced concrete structure and brick walls. The inner courtyards have disappeared and their volumetry is simple and rather prismatic. Either tall buildings, with eight to ten storeys or low raise blocks, with three or four levels, they are provided with large windows and generous balconies or loggias.
Around the middle of the seventies, precast concrete systems have been developed and used in mass dwelling buildings. The apartments have smaller rooms than the ones built in the previous period and, due to the structural system, the glazed surfaces decreased considerably.

The earthquake in 1977 proved that these precast concrete buildings had a very good structural behavior and therefore the system was widely used in the next decade. The facades were rather plain and even today there are districts that look absolutely alike, although they are situated in different parts of the city (Figure 2).

Figure 2. Images of blocks of flats of the eighties, in Bucharest.

However, the buildings that "invaded" the center of the cities (like the ones on the former "Boulevard of the victory of socialism") are based on individual designs. Their architectural style follows the line of the epoch; the structure is not as rigid as the one imposed by the precast large panel system, the rooms and windows are generous and in some (very rare) cases, shading devices have been provided.

2 SYSTEMS OF DWELLINGS – SYSTEMS OF WINDOWS

The different joineries are classified according to the material that they are made of (wood, plastic, etc) as well as according to the number of sashes (simple, double, triple) and the type of the opening system (“exterior opening”, “interior opening”, “customary opening”, “sliding opening”, etc) Traditionally, in Romania double windows were used for dwellings, with wooden joinery. There were some exceptions, in the seventies, when steel joinery was designed and provided in mass dwellings.

2.1 Dwellings built before the First World War and in the inter-war period

There are two most common opening systems for the joineries of this period: the “customary” system, with one sash that opens towards the interior and the other towards the exterior and the “interior” opening system, with both sashes that open towards the interior.
The old buildings (dating before the First World War) were equipped mostly with windows provided with “customary opening”, as shown in Figure 3.

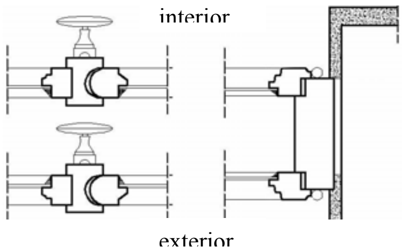

Figure 3. Traditional windows with “customary opening”.

Hygro-thermally these windows have an excellent behavior: when the wind blows on a façade, the exterior sash is pressed on the frame, thus increasing the air tightness of the window assembly.
Although, in our rough winter climate, this characteristic makes such windows very interesting, they have disadvantages that led to their elimination from the contemporary buildings, due to the sash that opens towards the exterior, which:
- represents a potential danger for the by-passers (of being hit) in the case of the windows on the ground floor,
- is difficult (if not sometimes impossible) to clean
- is difficult to maintain, as the frame of the sash and all the metallic accessories are exposed and unprotected to the environmental agents

Windows with “interior” opening system (Figure 4) were mainly used on ground floors (for the safety of the by-passers) and when shutters were provided

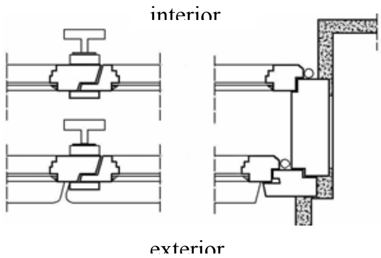

Figure 4. Traditional windows with interior opening

The wooden shutters that could be usually found were either the so called “a la persienne” or the “rolling shutters”. In either case, the closed shutter created an extra air layer, which, along with the shutter, improved the thermal resistance of the window in the cold winter nights. In daytime, the shutters kept a cool indoor temperature or permitted the natural ventilation of the room, while keeping the sun heat out. Before the communist era the shading devices were considered a normal, common equipment for the windows, but they were eliminated from the facades of the dwellings that were constructed after the Second World War, being looked upon as too great a luxury for the mass.

While rolled steel profiles entered the market, sometime in the inter-war period and began to be used in public buildings, the dwelling remained traditional: wooden windows with double sashes, mostly with interior opening and with sun protection devices (especially rolled shutters).

2.2 Dwellings built after The Second World War

In the sixties the same types of wooden joineries were used, but while in the previous period the dimension of the profiles was different (usually the bottom traverse was 10 mm larger than the other elements, ensuring better structural, geometric, life span performances) now all the wooden profiles have the same geometry, from economic reasons; also the carved decoration of the profiles disappeared. Obviously the shutters disappeared, with very few exceptions of blocks of flats built in the center of the cities.
In the seventies, profiles made of steel sheet were produced on pressing machines and metallic joinery windows were provided on apartments.

During this period and until the end of the ninth decade windows were double (two sashes); the most common opening system is "interior opening, coupled" (the sashes work together; when cleaning is needed; a hinge between the sashes allows them to open one against the other, as shown in Figure 5)

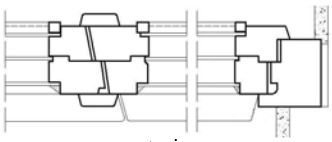

Figure 5. The two sashes are supported on a more robust frame. Placed between the two sashes, a hinge allows them to open one against the other and clean the two sheets of glass

3. IMPROVING THERMAL COMFORT

3.1 Improving thermal comfort by one selves

The effects of the energy crisis of the early seventies were dramatically felt in Romania at the beginning of the eighties, both in the changes of the technical regulations (by diminishing the values of the performance levels of the appropriate technical parameters), as well as in drastic economies in heating the apartments (no apartment heating stations existed and the block heating station of the inter-war buildings worked with gas), from the district steam generating station.
Therefore, individuals had to adapt themselves and their houses to survive in the given situation.
The first measure of improving the thermal behavior of the glazed part of a dwelling was to diminish the air circulation between the interior and the exterior; gaps between the frame and the wall, as well as between the frames and the sashes had to be sealed. Experimentally, and according to the financial means of the individuals, some people improved the thermal comfort of their rooms, by adding sealants between the frames and the sashes (rubber, foam rubber, or even textile made rolls), by adding extra panes of glass to the existing ones, by adding an extra sash (if the frame was strong enough), by providing roller blinds between the sashes when possible, by removing the existing windows altogether (frames and sashes), if the dweller could afford it, or even by adding light weight, compact, insulation panels (made from expanded polystyrene on wooden frames), propped against the window during the night and removed in day time. Adding an extra glass sheet on the sash and fixing it with wooden or metallic profiles is improving the heat loss; on the other hand, this hand-made insulated glass panel is subject of condense and dust problems. In all cases, these measures can be reduced to two principles: sealing against the air infiltrations and / or providing an extra layer of air, with thermal insulating qualities.

Another frequent example of personal approach is represented by the case of the balconies and loggias that were closed by the owners, according to each ones possibilities and means; therefore, each has another paneling, another joinery system, uses other materials for the joinery (metal, wood, plastic), other glass types (clear, ornamental, mass colored, wired).
Ultimately, these *are* systems that an existing window can be thermally refurbished with, but the approach should be made on the whole building, with the appropriate designs, technologies and materials in order to preserve a unitary image of the building.

3.2 Improving the thermal comfort of the windows in the nineties

During the last decade individuals began to improve the thermal comfort of their apartments by improving the thermal performances of the windows. Many wooden windows were replaced with PVC or aluminum windows. This simplistic approach led to problems of condensation on walls, due mainly to the changing of the inner hygro-thermal environment, as the new windows were very much more air-tight and the increasing humidity remained inside the rooms.
Considering the traditional way of life (that implies heavy cooking using blaugas in the kitchen and laundry washing in the bathroom), the quantity of water vapors that accumulates indoors is important.
It may be considered that the interventions that took place in the nineties did not help neither the occupants, nor the buildings.
As the financial possibilities increased, more individuals could afford to install rolling stutters when replacing the joinery, thus accomplishing a better thermal comfort in summer as well as in winter, but with the disadvantage of diminishing the glazed part and with the disadvantages presented above.

3.3 Provisions regarding the thermal refurbishment of the windows

Romanian technical regulations include the performance levels that windows should accomplish. Knowing that old buildings don't have the same performance as the new ones, the main degradations were listed and a correspondent coefficient for the correction of the specific thermal resistance, based on measures and experience, was proposed (Table 1).

Type of joinery	**Type of defect / degradation** (qualitative appreciation, made only to establish a correction of the thermal resistance of the window)	**Correction coefficient of the specific thermal resistance**
Double wooden joinery	- joint between: frame and sash; mullion and sash; sashes ▪ 5 mm < joint width ≥ 3 mm	**10%**
	▪ joint width ≥ 5 mm	**20%**
Coupled wooden joinery	- joint between: frame and the coupled sash assembly; mullion and coupled sash assembly; coupled sash assemblies ▪ 5 mm < joint width ≥ 3 mm	**15%**
	▪ joint width ≥ 5 mm	**25%**
	- joint between the sashes that constitute the coupled sash assembly ▪ 5 mm < joint width ≥ 3 mm	**5%**
	▪ joint width ≥ 5 mm	**10%**
Metallic and PVC joinery	- joint between: frame and sash; mullion and sash; sashes ▪ 3 mm < joint width ≥ 1 mm	**15%**
	▪ joint width ≥ 3 mm	**25%**
	- joint between the groove and the sash; between the groove and the glass ▪ 3 mm < joint width ≥ 1 mm	**5%**
	▪ joint width ≥ 3 mm	**10%**
All joineries	- untightness between the joinery elements and the opening in the building element	**20%**

Table 1.The correction coefficients for the specific thermal resistance of the windows [2]

The correction coefficients of the specific thermal resistance that are provided in Table 1 do not take into consideration the degradations that are due to the inappropriate current maintenance work (repairing of the metallic accessories, replacement of putty, etc).
In all the technical regulations, all thermal calculations are made on new windows. In reality the thermal performance of an old window cannot be the same with the one of a new window. In consequence, Table 1 is an instrument of evaluation of the physical state of the window. The width of the joint dimension is a qualitative mark, expressing a degree of deterioration of the frame – sash assembly and accordingly a decrease of the thermal performance. No connection is made between the thermal resistance and the air-tightness of the window.
As an example, in the case of a double, wooden window, with a measured joint between the frame and the sash of 4 mm width, the corrected thermal resistance could be calculated by taking into account the influence of the appropriate correction coefficient for the specific thermal resistance:

$$\mathbf{R_{corrected} = R - 10\%R}$$

When completing the windows with shutters, the thermal resistance of the window will increase, according to the calculation prescriptions provided by SR EN ISO 10077 – 1 [3]

It should be noted that, in the case of window refurbishment, each existing window is unique (due to the initial thermal and functional qualities, way and quality of maintenance, of use, age, etc).
A checklist, for the main steps that should be made for the rehabilitation of a window, can be the following:

1. Before approaching the refurbishment design, a technical expertise should be made, as detailed as it can be in what joinery is concerned (and aiming to identify the state of the joinery and of the joint between the window and the adjacent wall)
2. On the basis of the results of the expertise, the revision or repairing of the window should be made (with replacing the elements that are more degraded than the rest of the window elements; the measures that shall be taken in order to improve the thermal performances of the window depend on the state of the window. If the expertise shows that the window is seriously deteriorated, the necessity of replacing it with a new one may appear.
3. The air-tightness between the joinery and the wall should be checked and, in case it proves to be deficient, it has to be remade, with injected foam.
4. The re-accomplishing of the air-tightness of the window assembly must be made with care, because a hasty intervention may lead to dis-functionalities of the wall-window assembly (condensation or mold problems) or of the joinery itself (breaking the hinges, preventing the sashes to close, etc). It is recommended to provide gaskets in order to accomplish air-tightness, with to avoid breaking the hinge or the closing devices. In order to protect the gaskets against the UV radiation, they should not be placed towards the exterior.

In what the hygro-thermal environment is concerned, an excessive air-tightness proves to be dangerous, as the elimination of the excess of water vapors from the indoor air is decreased. This may lead unwanted phenomenon, which were not specific before the rehabilitation either for the inner surfaces of the room (condensation on the joinery elements, condensation on the adjacent walls), or for the occupants (oppressive heat, head aches, general discomfort). The excessive air-tightness may lead to an insufficient freshening of the indoor air.

3.4 Possibilities of intervention over the existing window; solutions in principle

It should be noted that, where windows are concerned, the problems due to the of the action of the environmental agents, even if different in weight, have the same consequences over the given subassembly, in many different parts of the world. Studies of repairing, refurbishing and improving the performances of the windows are carried out by research institutes as well as by professional corporations [4], [5], [6], [7], [8].
In the case of the wooden joineries, the approach of improving the thermal performances can be a step – by – step one, each phase being distinct.

By checking the geometric features of the joinery, it can be established if the joinery should be refurbished or if the replacement is imposed.
If the deteriorations exceed 25%, the replacement of the window is necessary.

The sashes should be checked (especially the fixed joining). If the constitutive elements of the frame are in good shape, it is recommended to provide metallic angle profiles, to reinforce the corner.
It is known that the lack of the necessary air-tightness leads to lower indoor air temperature. Therefore, the next step is to ensure the necessary air and wind tightness.
The two aspects of the problem view the diminishing of the air infiltrations between the window and the adjacent wall (by sealing the gaps), as well as between the elements that form the window (the way their geometry permits superposing or by adding gaskets).

Finally the last step can be made, to improve the thermal resistance of the window.
This goal can be achieved, in principle, by changing the type of the glazing (with a contemporary, performant one) or by adding an extra insulating air layer.
The extra layer of air can be accomplished if one – or more – of the following measures are taken:
- providing a supplementary glass sheet, from changing a single glass sheet with an insulated glazing panel, to adding an extra sash to the existing window assembly.
- providing supplementary sun protection devices, that have a role in the improvement of the thermal insulation (shutters, blinds).
- replacing the window altogether, with a more performant one.

The possible condense that may appear on face 2 of the exterior glass is expected to dry, as the air-tightness between the external sash and the frame is reduced.
Figure 6 presents a scheme of interventions over an existing window, in terms of actions and phases.

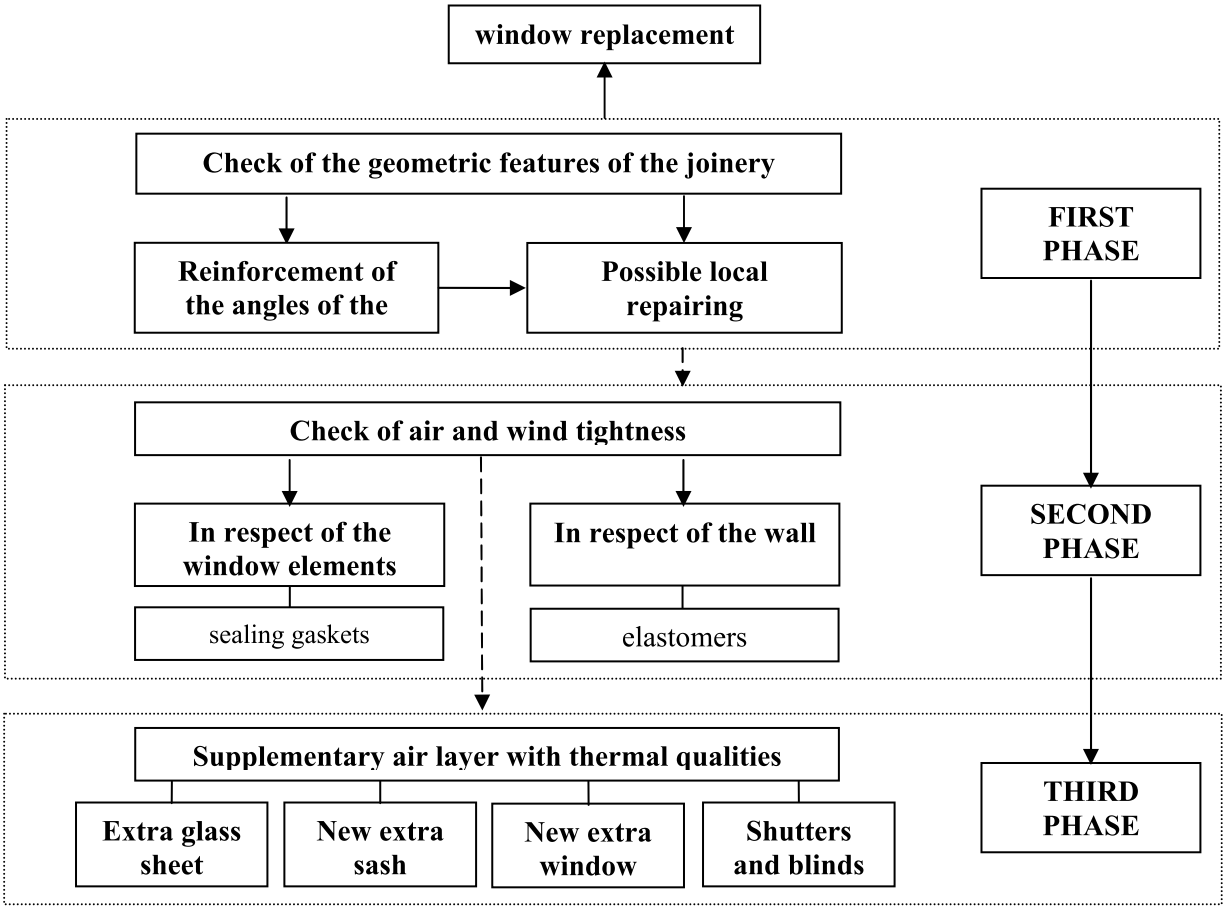

Figure 6. The phases of the intervention process, over an existing window

While improving the thermal performances of the window, implicitly the acoustic performances are improved as well, due to the:

- provision of the third sheet of glass (preferably thicker than the ones existing on the window and with coatings, according to the cardinal points).
- provision of the sealing gaskets
- provision of the sun protection devices
- changing of the window

A supplementary measure for the acoustic improvement is to add sound absorbent materials on the frame, between the sashes.

While a wooden window has chances of being repaired, aluminum or plastic joineries can hardly bear any intervention, the only possible one (like adding a new window in front of the existing one and fix it independently on the wall) being too aggressive. Therefore, in these cases, changing the glazing unit (where possible) is the only possible improvement.
Steel joineries have been improved by adding an extra wooden or plastic sash, or by adding altogether a new window towards the interior of the room.

CONCLUSIONS

Solving the problem of increasing thermal comfort by diminishing heat loss through the glazed part of the envelope of the existing mass dwelling buildings is a goal for any refurbishing approach.
Due to the fact that the immense majority of buildings that should be refurbished consists in blocks of flats, the improvement of the windows can hardly be made by each owner (according to his/her financial means and according to ones' own imagination), as this approach has proven to lead to strange, heterogeneous, images of facades, with too many different kinds of posts, panels, joineries, glazing. The problem should be taken over altogether, across the whole building.
However, for the time being, as there are no possibilities to include the problem of refurbishment of the buildings in nationally or internationally supported financial programs, the next fair solution is to take advantage of the works of general repairing and to consider the thermal refurbishment as a part of these general repairing works.
The fact that these type of work implies the investing of substantial funds makes them inaccessible for the majority of the people who live in mass buildings and, at least for the time being, shall improve their life according to their own private budget.

REFERENCES

[1] National Institute for Statistics, 2002, *General results of the Census of the population and dwellings*, www.insse.ro
[2] Dabija, A-M, Stan A., 2002 *Technical Guide for thermal refurbishing of joineries in existing buildings*, Minstry of Transportation, Constructions and Tourism
[3] SR EN ISO 10077–1:2000, *Thermal performance of windows, doors and shutters – Calculation of thermal transmittance – Part 1: Simplified method*
[4] Steven Winters Ass. Inc. Building Systems Consultants Norwalk CT, 1999, 'Windows & Doors' in *The Rehab Guide*, US Department of Housing and Urban Development, Office of Policy Development and Research
[5] *Repairing timber windows: investigating defects and dealing with water leakage* Building Research Establishment, Good Repair Guide 10, Part 1, 1997,
[6] http://www.cr.nps.gov/hps/tps/standguide/rehab/rehab_energyeff.htm
[7] http://www.state.nj.us/dep/hpo/4sustain/windowsave.pdf
[8] http://irc.nrc-cnrc.gc.ca

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Non Destructive Waterproofing Remedial Treatment

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ABSTRACT

Excess dampness caused by water ingress is the most widespread and damaging cause of deterioration affecting building components. The effects of high levels of moisture on buildings can be devastating and usually demand total replacement of affected areas. However, the total replacement of waterproofing systems can be a destructive and tedious process. This paper presents a non-destructive method of waterproofing remedial treatment of Multiple Protection Injectable System (MPI) to treat water ingress defects such as interfloor seepages, basement water intrusion and external facades water infiltration. This MPI system has been applied successfully and refined for the past five years over a total of 250 water intrusion cases in Singapore. Three successful local case studies of each interfloor seepage, basement seepage and external façade water infiltration, resolved by MPI system would be demonstrated in this paper.

KEYWORDS

Non Destructive, Waterproofing remedial treatment, Mutiple Protection Injectable System.

INTRODUCTION

All buildings start to deteriorate from the moment they are completed. This inevitable process can be regulated and hence, the life span or ultimate failure of the building or elements can either be avoided or accelerated according to the way in which it is maintained (Chew et.al., 2004b).

A failure can be considered a shortcoming in the function and performance of a building, and manifest itself within the structure, fabric, services or other facilities of affected building. Excess dampness caused by water ingress or leakage is the most widespread and damaging cause of deterioration affecting buildings (Watt, 1999). Wet areas in buildings are defined as areas where they are subjected to constant damp conditions with alternating dry and wet cycles (Chew et. al., 2004a). The effects of high levels of moisture can be devastating when it accelerate conditions for both chemical and biological degradation.

EXISTING PRACTICES FOR WATERPROOFING REMEDIAL TREATMENT

In Singapore, where majority of the building stocks are of full or part masonary construction, key areas experiencing persistent water intrusion defects are interfloor seepages from bathrooms, basements and external wall seepages, with interfloor seepages being the most widespread.Though the percentage of areas occupied by the wet areas (bathrooms and toilets) are usually not more than 10 % of the building gross floor area, the annual maintenance cost for wet areas can range from 35% to 50% of the total maintenance cost of a building (Building Construction Authority, 2000). Typical interfloor seepages are as in Table 1.

Table1: Interfloor seepage defects

Common repairs and remedial to the above mentioned water intrusion defects are typically carried out by general contractors or waterproofing contractors. Two main methodologies frequently adopted by the local contractors are either total replacement wet area and/or polyurethane (PU) grouting at seepage areas.

Total replacement of wet area comprises of hacking out the existing wet area waterproofing and tiling system. Due to the size of wet areas, total replacement and reinstallation of waterproofing, tiling system and sanitary fixings is often considered the preferred method but the destructive nature and high cost often lead owners to low cost quick-fix alternative such as PU grouting. (Figs. 1 and 2).

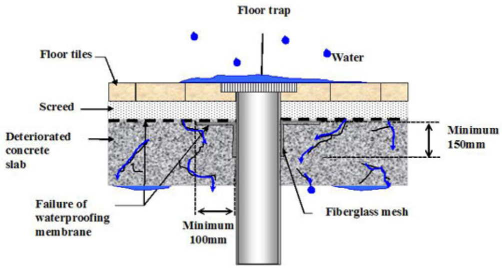

Figure 1: Typical deteriorated wet area

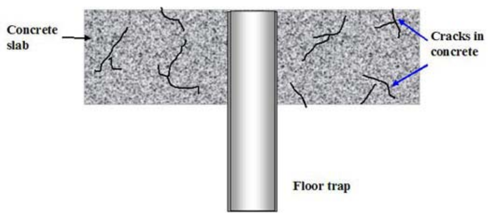

Figure 2: Removal of entire waterproofing and tiling system for wet area

The technique of PU grouting to stop water leakage has existed in the industry as early as 1960s (Woods, 1982; McBrayer and Wysocki, 1998). Typically in a situation of leaking bathroom where waterproofing membrane has failed and cracks are inherent in concrete, the main application of PU serves to inject chemical from the underside of the leakage points or cracks at a high pressure ranging from 800 - 1500 PSI (Fig. 3). As high viscocity PU comes into contact of water, the chemical will swell to form foam-like susbtance to "plug" the leakage points. Under such high pressure, PU grouting risks the propagation of existing cracks. In addition, in many cases, the cured PU can only plug water leakage at localised area, thus redirecting any persistent water intrusion to other weak areas without addressing the water source.

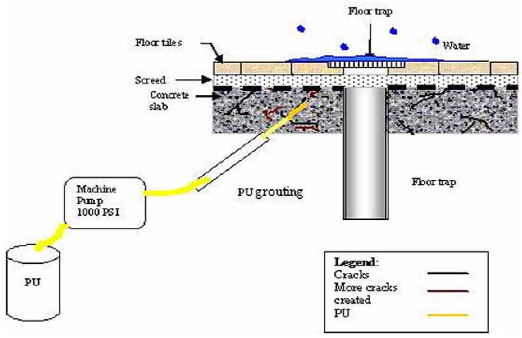

Figure 3: Application of PU to leakage area

As owners become better informed of water intrusion defects, they demand more efficiencies in remedial methods to resolve water intrusion defects in a hassle-free and less destructive manner. Total replacement of wet areas no longer appeal to owners being the need for total removal of existing finishes and fixings. Gradual recognition in the maintenance industry of the inefficiencies of PU grouting not able to provide a durable solution to water intrusion defects has also set in. These compel the owners to demand more innovation and attention in treating water intrusion cases.

This paper presents a non-destructive method of waterproofing remedial treatment of Multiple Protection Injectable System (MPI) to treat water intrusion defects such as interfloor seepages, basement and external façade water intrusion and its case studies.

MULTIPLE PROTECTION INJECTABLE SYSTEM (MPI)

The system

MPI is non-destructive as the actual waterproofing remedial treatment is carried out without large scale hacking, total removal of existing tiling system or sanitary fixings. MPI is a comprehensive multi-pronged approach, specially developed to suit our tropical climate, to target at eliminating the inefficiencies of the remedial practices of total replacement of wet areas and PU grouting. MPI system actively combines waterproofing chemicals and techniques synergistically to repair water intrusion areas at its source to achieve a watertight environment. For the past five years, MPI was mastered and refined, and have successfully rectified more than 250 complex water intrusion cases (Fig. 4). Arepresentation of the methodology of MPI is shown in figure 5.

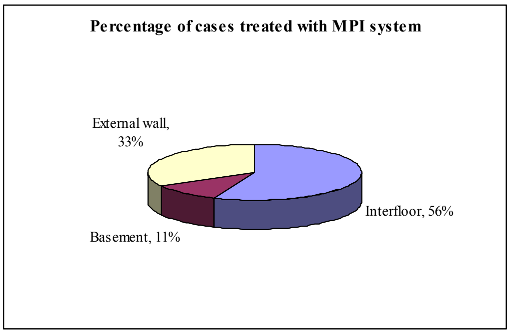

Figure 4: Percentage of cases treated with MPI system

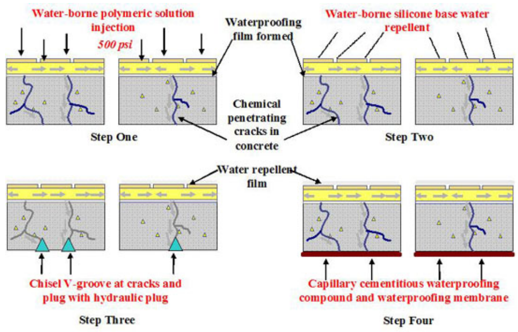

Figure 5: Multiple Protection Injectable System for Interfloor Seepage

Step 1: At the existing bathroom, a low viscosity polymeric solution is injected in between tiles joints. The chemical flows via low pressure of 150-500 PSI and by gravity downwards through the tile screed and the concrete. Upon curing, the chemical gels and seals up all the cracks in the screed and concrete, forming a watertight environment. A waterproofing film is also formed at the existing waterproofing membrane, thus "repairing" the existing membrane (Fig 5).

Step 2: A water repellent is sprayed onto the existing floor tiles to form protective waterproof layer over the entire floor area.

Step 3: From below, grooves are chiseled at visible cracks and plug with hydraulic cementitious compound to seal off the cracks where water and polymeric solution had leaked.

Step 4: To complete MPI system, capillary cementitious compound and flexible waterproofing membrane is applied to the underside to treat and waterproof the soffit.

Flexibility of this system

MPI was carried out to other areas where complex water intrusion cases also occur, such as basements and external walls. During a building's lifespan, basements and external walls are often exposed high risk of water intrusion. Water intrusion cases in basements demand more novel methods to treat as existing wall basements are subjected to constant water table and total replacement of external tanking is not practical and PU grouting can not effectively rectify such complex water intrusion.

Whereas for external facades, difficulties of access is the main bane to provide cost effective repairs to water penetration. However, with constant research and development and onsite experimentations on the feasibility of MPI on these two areas, MPI has proven to be successful not only in resolving interfloor seepages, but has the flexibility and capacity to resolve water intrusion for basements and external wall of all cases. Below are three typical case studies for complex interfloor seepage, basement and external wall water intrusion solved by MPI.

CASE STUDIES

Interfloor seepage

	
Plate 1: Interfloor seepage	Plate 2: Efflorescence formation on floor

			
Plate 3: Injection of chemical to fill up porous screed and waterproof	Plate 4: Chisel groove to the underside of bathroom	Plate 5: Stand by to apply hydraulic plug to underside of bathroom	Plate 6: Application of cementitious waterproofing compoound to underside

Plates 1 and 2 depict typical signs of interfloor seepage. Works are carrried out at existing affected areas with selective drilling locations, coupled with low pressure and gravity, the chemicals were able to penetrate into the cracks and voids without any hacking. Plates 3 to 6 illustrate actual onsite case study of the application of MPI to resolve interfloor seepage. Plates 3 and 5 show application of hydraulic plug to areas where the injected chemical on the top leak out from the underside of the concrete slab. This indicates that the injected chemical has filled up all cracks and void of concrete slab. To complete the system, cementitious waterproofing compound is applied to seal off any large visible cracks and set to cure.

External wall water intrusion

	
Plate 7: Water intrusion at 1st level external wall	Plate 8: Water intrusion at 2nd level external wall

			
Plate 9: Chisel groove to receive hydraulic plug	Plate 10: Injection of chemical into wall from internal	Plate 11: Injected chemical flow out from the internal	Plate 12: Application of water repellent

Plate 7 and 8 shows water intrusion through external wall. Plate 9 shows application of hydraulic plug to grooves to seal off large cracks. Plate 10 and 11 depict the injected chemical to penetrate the voids in the external wall and upon curing, forms a gel-like susbtance to achieve watertightness of wall. Plate 12 shows the last step of MPI to provide a second line of defense to the completed system.

Basement water intrusion

	
Plate 13: Overview of water intrusion	Plate 14: Overview of water intrusion

			
Plate 15: Scrape off defective paint	Plate 16: Injection of chemical into basement wall	Plate 17: Application of capillary chemical to wall	Plate 18: Completion of remedial waterproofing

Consequences of complex water intrusion are shown in Plates 13 and 14. Plate 16 shows how the injected chemical penetrated the substrate and travel distance before flowing out at the weak point, thus sealing off all voids as it migrates. Application of capillary cementious compound to the entire substrate is illustrated in Plate 17. Plate 18 shows completion of waterproofing works. Comparatively, Plates 13 and 18 demonstrate the before and after of a complex basement water intrusion case.

CONCLUSION

This paper illustrates the methodology of a developed non-destructive waterproofing remedial treatment of MPI, tried and tested on over 250 complex water intrusion cases in Singapore. Three typical case studies were presented to show how the MPI system was adopted at interfloor, basement and external wall seepages. With synergistic combination of material chemicals and technology, the MPI system has achieved flexible application while not comprising on its effectiveness and efficiencies to arrest water intrusion. Its application has eliminated the inefficiencies of the destructive nature of total replacment of wet areas and recurring issues contributed by PU grouting and has also proven itself as a more viable alternative to today's water intrusion defects.

REFERENCES

Watt, S. D. 1999, *Building Pathology: principles and practice*, Blackwell Science Ltd, United Kingdom.

Woods, G. 1982, *Flexible polyurethane foams : chemistry and technology*, London ; Englewood, N.J. : Applied Science Publishers.

McBrayer, R. L. and Wysocki, D. C. 1998, *Polyurethane foams formulation and manufacture*, Lancaster, Pa.,Technomic Publishing Company.

Chew, M.Y.L, Tan, S.S and Soemara, E. 2004a, *Serviceability of materials in the tropics*, Journal of Architectural Engineering, **10** (2), pp 69-76.

Chew, M.Y.L, Tan, S.S. and Kang. K.H. 2004b, *Building maintainability-review of State of Art,* Journal of Architectural of Engineering, **10** (3), pp 80.

Building Construction Authority 2000, *Enhancing maintainability of buildings*, BCA Pilot Study Report, Singapore.

Environmental characterisation and mapping with respect to Durability

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ABSTRACT
Service life planning calls for characterisation and classification of the exposure environment for the constructed asset(s) in question. Lack of knowledge of environmental exposure data and models among the building sector players is an important barrier for further progress towards service life prediction. The ever more evident climate change highlights even more the need for data and models on the exposure, when it comes to address its impact on the built environment.
In general, requirements for establishing and implementing systems for quantitative characterisation and classification of durability of materials and components are: 1) well defined, and relatively simple damage functions for the materials in question, 2) availability of environmental exposure data/loads, including methods and models for assessing their geographical distribution, and 3) user friendly IT systems for storage, processing and modelling the environmental loads onto structures.
Service life functions related to environmental degradation are today available for a range of building materials and components. As for availability of environmental data and models, as well as proper IT systems, it is shown that for most European countries, such data and models are available from meteorological offices and the environmental research area, and that these data and the work performed are directly applicable for service life planning and life cycle management of constructed assets. A short review of some of the most applicable models for environmental exposure and for degradation and damage of building materials and structures is included.

The global climate system is likely to undergo changes, regardless of the implementation of abatement policies under the Kyoto Protocol or other regimes. Both the functionality of the existing built environment and the design of future buildings are likely to be altered by climate change impacts, and the expected implications of these new conditions are now investigated.

The data and models are often directly exhibited in computer-based systems, often on GIS based platforms. With the rapid development of IFC based standards for digital object oriented models of building products there is a huge need for property sets, such as durability and service life data, linked directly to the building elements. The significant drive within the AEC/IFC community to provide for relevant location based data (GIS) via IFC format will be a major facilitator for access to site specific durability data, described by degradation models containing environmental (and other) degradation factors.

KEYWORDS
Environmental characterisation, service life, durability, climate change

1 INTRODUCTION

Service life planning calls for characterisation of the exposure environment for the constructed asset(s) in question, as described in ISO 15686-1 [ISO 2000]. Lack of knowledge of environmental exposure data and models among the building sector players is an important barrier for further progress towards service life prediction. The ever more evident climate changes under global warming highlights even more the need for data and models on the exposure, when it comes to address its potential impact on the built environment. The significant drive within the AEC/IFC community to provide for relevant location based data (GIS) via IFC format will be a major facilitator for access to site specific durability data, described by degradation models containing environmental (and other) degradation factors.

In general, requirements for establishing and implementing systems for quantitative characterisation and classification for durability of materials and components are: 1) well defined, and relatively simple damage functions for the materials in question, 2) availability of environmental exposure data/loads, including methods and models for assessing their geographical distribution, and 3) user friendly IT systems for storage, processing and modelling the environmental loads onto structures.

2 DEGRADATION MODELS AND ENVIRONMENTAL DEGRADATION FACTORS

Establishing proper dose-response and damage functions for families of common building materials have been the subject of extensive studies for more than a decade [Jernberg *et al.* 2004]. Although many models and functions now are available the lack of knowledge and implementation of the damage function approach still constitute a major barrier for progress concerning the durability and service life aspects within the building and construction community.

The degradation of the buildings and infrastructures are influenced by a whole set of factors such as environmental degradation agents, type and quality of the materials and components, protective treatment, etc. [ISO 2000].

The relationship between the environmental degradation agents and the observed effects are expressed as dose-response functions. The dose-response functions are not directly suitable for service life assessments. To transform the degradation into service life terms, performance requirements or limit states for allowable degradation before maintenance or complete renewal of material or component, have to be decided. The dose-response function then transforms into a damage function, which is also a performance over time function, and a service life assessment can be made.

In order to characterise and report the right type and form of the environmental degradation agents, they have to be related to the degradation mechanism and dose-response functions for the specific materials in question. Further, a holistic approach modelling the physical processes controlling the corrosion needs to be considered across a wide range of physical scales, from macro through meso/regional to local, micro and lastly micron [Jernberg *et al.* 2004, EOTA 1999, Cole 2003], see **Figure 1**. Macro refers to gross meteorological conditions (polar, subtropical etc.), meso or regional refers to regions with dimensions up to 100 km, local is in the immediate vicinity of a building, while micro refers to the absolute proximity of a material surface. The microenvironmental conditions which are crucial to the materials degradation can vary enormously over a real construction.

Surface response then refers to largely physical responses of a surface such as deposition and retention of pollutants or condensation and evaporation. Micron refers to interactions within the buildings and infrastructures/metal/oxide/electrolyte interfaces. In this approach, models on different dimensional scales are linked together so that the models on micron level are informed by models on the macro-, meso-, micro- and surface response regimes.

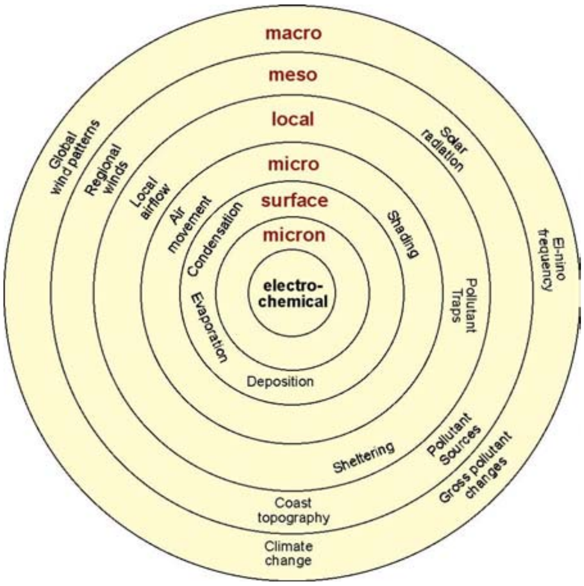

Figure 1 Framework for holistic model of corrosion [Cole 2003]

Valuable contributions are provided from the world wide studies within the environmental research area to establish such functions. They are a necessary basis for the cost-benefit analysis preceeding policy making in this area. The materials studied comprise structural metals (carbon steel, weathering steel, zinc, aluminum, copper, etc), stone (lime-and sand), paint coatings (on coil coat, steel, and wood), electric contact materials, glass- and polymer materials.

For porous and composite materials the functions can with this model become rather complex, as shown by the recently developed models for concrete within the European research projects DuraCrete [Duracrete 2004], and further applied in the object specific project DARTS [DARTS, 2004], as well as LIFECON [Sarja 2004]. In the latter project parametric sensitivity analysis have been performed to establish the ground for a quantitative classification of the main climatic parameters [Lay 2004].

Wind is an environmental mechanical degradation factor, and wind induced damages to buildings is at the same time a complex of several causes. In accordance with the factor method [ISO 2000] both design and craftsmanship are variables that will influence on the damage ratio of buildings exposed to strong winds [Thiis 2005].

Table 1 shows some very important primary environmental parameters/data as extracted from the damage functions for some important building materials, as well as for the building stock as such. Some examples of such functions and resulting environmental mapping and classifications are given below.

Deterioration mechanism	*RH*	*Temp.*	*CO_2*	*Precipi-tation*	*Wind*	*Radia-tion*	*Chloride Conc.*	*Freeze -thaw cycles*	*$[SO_2]$*	*$[O_3]$*
Reinforced concrete (DuraCrete models)										
-Carbonation induced corrosion	X	(X)	X	X	X					
-Chloride induced corrosion	X	X		X			X			
-Propagation of corrosion	X	X		X			X			
-Alkali-aggregate reaction	No model									
-Frost attack internal/scaling	(X)	X		X	(X)	(X)	(X)	X		
Other materials (Dose-response functions)										
-Galvanised steel/zink coating	X	X		X			X		X	
-Coil coated steel	X	X		X					X	
-Sealants/bitumen	No function									
-Polymers	No function									
-Aluminium				X			X		X	X
Building stock										
					x					

Table 1 Relevant environmental primary data for degradation models linked to buildings and infrastructures

3 SYSTEMATIC AND OPTIONS FOR CLASSIFICATION OF ENVIRONMENATL DEGRADATION FACTORS AND CORROSIVITY

Characterising and subsequently classifying the exposure environment in order to assess the aggressivity towards buildings and infrastructures have been attempted for about three decades, and some systems do exist. Some of the most relevant systems are described in the following.
For *quantitative* classification of atmospheric environmental loads there are two *options*. Some systems try to classify the *generic atmospheric aggressivity* on a global to local scale [ISO WD/15686-4, 2002 and EOTA 1999], without specific knowledge of damage functions, but based on overall experience of materials degradation at large. The other option and systematic is material (family) specific and based on knowledge of their damage functions, such as ISO 9223 that are specific for metals. This is the preferred systematic but requires that the type and format of the ingoing environmental agents are defined, and that the function(s) are relatively simple for practical purposes. Some examples are:

3.1 EOTA – Annex A Building context [EOTA 1999]

Quoted from the EOTA document on "Working Life of Building Products": "The wide variation in European climatic conditions and in the user stresses imposed on structures depending upon type of structure and use intensity will make it necessary with many construction products to restrict their usage to defined situations in order that these achieve the predicted working life". Then follow examples of possible sub-divisions of climatic zones in Europe, of orientation of products/components in structures, of internal exposure environments in buildings, etc.

3.2 ISO 15686 Service life planning – Part 4

The ISO 15686 suggestions for classification [ISO WD/ 15686-4, 2002], contains a proposal for:

1. Simplified global climatic classification with respect to two main factors, rainfall/humidity and temperature.
2. Simplified Global pollutant classification divided into two main areas, industrial pollution and marine pollution.
3. Detailed classification of moisture from rainfall and relative humidity. Another, more detailed approach for classification of moisture is to use the Annual Rainfall and Annual Relative Humidity.
4. Detailed pollutant classification of Airborne Salinity, frequency of significant salt deposition/frequency of rain.

3.3 ISO 9223-26 Classification of atmosheric corrosivity for metals

The standards ISO 9223–9226 *Corrosion of metals and alloys – Corrosivity of atmospheres* [ISO1992] have been developed for the classification of atmospheric corrosivity of metals and alloys. Based on a huge amount of experimental data for empirical dose-response functions the standards use both the approach of classifying the degradation factors and the corrosion rates. The **ISO 9223 – Classification**, specifies and classify the key factors in the atmospheric corrosion of metals and alloys, which are *time-of-wetness (τ), sulphur dioxide (P) and air-borne salinity (S).* The classification can be used directly to evaluate the corrosivity of atmospheres under known conditions of these environmental factors, and for technical and economical analyses of corrosion damage and choice of protection measures. The ISO 9223 approach has since the mid 80-ies been used by many researchers to classify and map the atmospheric corrosivity [Jernberg *et al.* 2004].

3.4 Proposed system for Concrete

The recently endorsed European standard -"EN 206-1 Concrete - Specification, performance, production, and conformity" [CEN 2001] is a good basis for developing the quantitative system. EN206-1 contains an agreed *qualitative* classification system as a synthesis of "best available" knowledge, covering the relevant degradation mechanisms and exposures in atmospheres, fresh water, seawater, and soil, indicating the decisive character of moisture and chloride. This implies also that National annexes, describing the environmental classes in relation to geography, have to be developed.

The damage functions for concrete, the so-called Duracrete models, are very complex and will have to be simplified for classification purposes. In the LIFECON project this was performed via parametric sensitivity analysis, see **Figure 2** [Lay 2003, Hallberg 2005].

4 METHODS AND DATA FOR ASSESSMENTS, MODELLING AND MAPPING OF DEGRADATION AGENTS

4.1 Data from meteorological and environmental research networks

As for availability of environmental data and models, as well as proper IT systems, it is shown that for most European countries, environmental data and models are available from meteorological offices and the environmental research area, and that these data and the work performed are directly applicable for life cycle management of the built environment. Such gathering of environmental exposure data is necessary to give ground for quantitative classification systems when the service lives of building products are to be declared in quantitative terms [Sjöström and Lair, 2003].

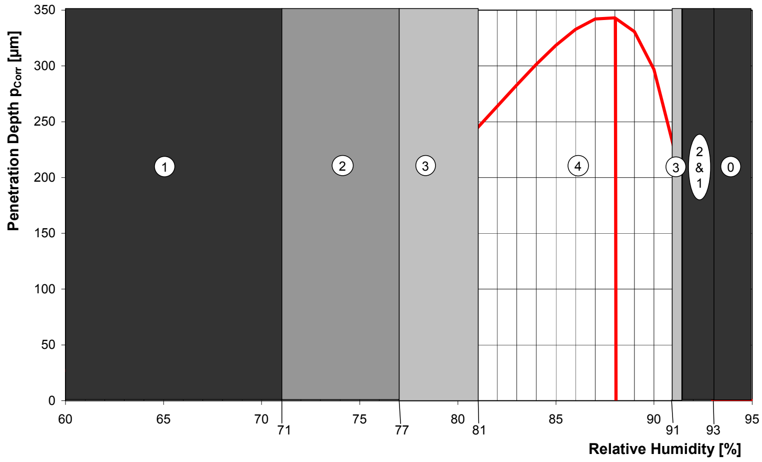

Figure 2 European classification of average annual relative humidity regarding carbonation induced corrosion, regarding 100 years of service life, ToW = 0; T = 10°C

In principle, characterisation of degradation agents has to be based on existing data. The measuring, testing and evaluation of air quality (pollutants) are gaining growing importance in developed countries as elements of a comprehensive clean air policy and geared to sustainable development. All countries in Europe have extensive meteorological and air pollution monitoring networks, many with GIS based information and management systems, allowing for the necessary assessment, modelling and mapping of the relevant environmental degradation parameters on various scales down to the local/micro scale on object level. Point measurements are very expensive, and for a broader assessment of air quality, needed for policy development and assessment, public information etc., measured data needs to be combined with modelling based on emissions inventories, to assess properly the exposure to, and thus the effects of the pollution on public health or on buildings.

The European Environment Agency (EEA) (www.eea.dk) was established in 1994, with the objective "to provide to the European Community and its Member States objective, reliable, and comparable information at a European level enabling the Member States to take the requisite measures to protect the environment, to assess the results of such measures and to ensure that the public is properly informed about the State of the environment " [Jernberg *et al.* 2004]. On the *regional scale*, there is extensive monitoring in addition to the EMEP network, and about 750 sites are in operation totally in Europe. This monitoring is very extensive for S- and N-compounds in air (gases and particles) and deposition, and also for ozone.

On the *local/urban scale*, monitoring is carried out at more than 5000 sites in Europe, operated by local, regional or national authorities. Most of these sites seem to be general urban background sites, while hot-spot sites (traffic, industry) are less well represented. The compounds of the EU Directives (SO_2, particles, NO_2, ozone, lead) are extensively covered.

4.2 Mapping and classification examples

4.2.1 Environmental parameters-on regional and microlevel

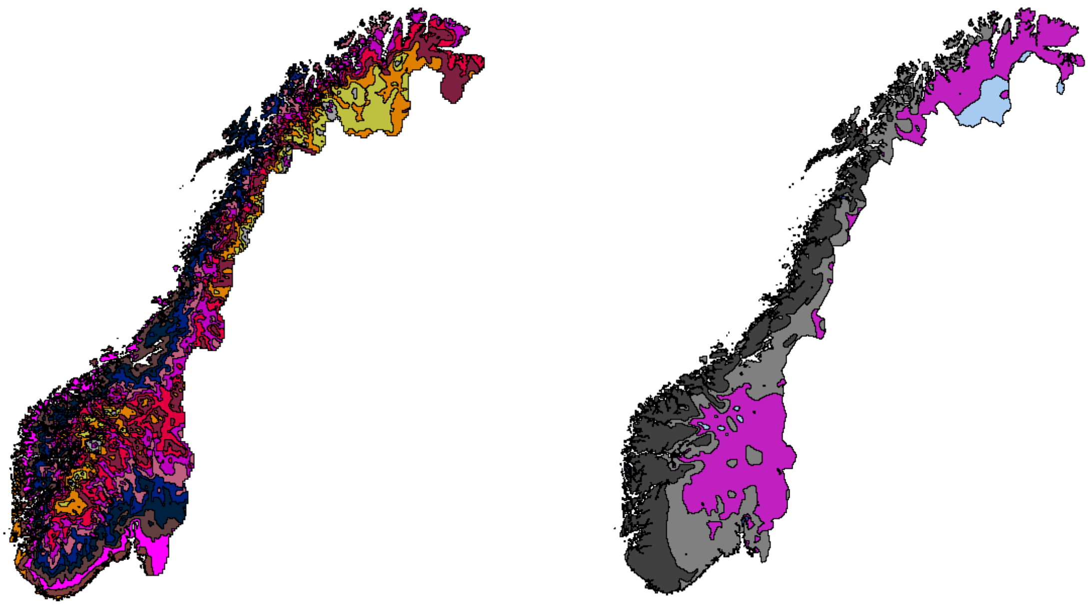

Figure 3a and b for Norway. a- average yearly temperature. The temperatures are classified from –6ºC to 8ºC. b- Average yearly precipitation in where Norway is divided into 4 zones. Zone 1< 400 mm, zone 2 < 800 mm, zone 3 < 1300 mm, and zone 4 > 1300 mm.

Important parameters, *average annual temperature* and *average yearly precipitation*, are shown for Norway in **Fig 3a and b**. Mapping is performed based on data from Norwegian Meteorological Institute, and the class boundaries may be chosen from any classification system. Shown here is for ISO 15686-4, but it could also be for developed concrete classes [Lay 2004].

Micro – environmental characterisation and classification can be done by use of Computational Fluid Dynamics (CFD) simulation, using existing climatic data. Numerical simulation of wind in complex terrain and around buildings has become a good supplement and sometimes a substitute to traditional wind tunnel simulations. Using CFD 3D models Haagenrud *et al.* [2002] have thus numerically simulated several different environmental loads acting on the Ormsundet wharf in the Oslo fjord basin. The models were constructed on the basis of 1:5000 maps and graphs, and used as input to CFD simulations.

The study shows that CFD can be a useful tool in assessing environmental loads, and thus degradation risk zones on dock structures. Such studies are a "first approach", and simulations needs to be validated with measurements. Point measurements performed at Ormsundet wharf indicate a good correlation with the CFD studies .

4.2.2 Degradation
Mapping of environmental parameters and of areas and stock of buildings with increased risk of corrosion, are performed and co-ordinated for Europe within the UN ECE Working group on Effects (WGE) under the Convention of Long Range Transport of Air Pollutants (CLRTAP), and specifically under its International co-operative Programmes (ICPs), for Modelling and mapping, and for Materials (ICP Materials), respectively [Tidblad and Kucera 2003].The dose/response functions for unsheltered materials are of the type:

$$K = \text{dry}\ (T, Rh, [SO_2], [NO_2], [O_3], t) + \text{wet}\ (\text{Rain}\ [H^+], t) \quad \textbf{(1)}$$

where K is the corrosion attack, T is the temperature in degree C, Rh is the relative humidity in %, [] is the concentration in $\mu g/m^3$ (SO_2, NO_2 and O_3), t is the time in years, Rain is the amount of precipitation in mm and $[H^+]$ is the acidity of precipitation in mg/l. The corrosion attack can, depending on material, be quantified as either mass loss (ML, g/m^2), surface recession (R, µm), ASTM D 1150-55 1987 (ASTM, 1-10), depth of leached layer (LL, nm) or weight increase (WI, $\mu g/cm^2$).

4.3 Climate classifications and exposure indexes

The "robustness" of the Norwegian building stock, including the development of methods for classifying different climatic parameters and their impact on building enclosure performance, are now being addressed as part of the NBI research & development programme "Climate 2000" [Lisø and Kvande 2004, Lisø et. al. 2003]. An important aspect of the programme will be the preparation of a thorough overview of the relevant climatic loads that should be taken into account during the planning, design, execution, management, operation and maintenance of the built environment. A navigable way of ensuring high-performance building enclosures is to develop climate classifications or climate exposure indexes for different building materials and building enclosures [Lisø et. al. 2004]. A new method for assessing driving rain exposure based on multi-year records of synoptic observations of present weather, wind speed and direction coupled with average annual rainfall totals has been proposed by Rydock *et al.* [2004]. Lisø et. al. [2004] are now developing a simple method for risk and vulnerability assessment of frost decay based on multi-year records of one time daily registration of maximum and minimum air temperature and daily precipitation totals in different parts of Norway.

Many meteorologically related damages usually happens during extreme load events as opposed to chemical degradation. Hailstorms is one such load and the hail induced damage to buildings is directely related to the hail kinetic energy [Hohl et.al., 2002]. Modelling of hail damage is today a commercial product. A similal relation between meteorological load and damage to building has been established by using historical events [Thiis et.al., 2004]. By using records of known wind storms and connect this to the actual insurance loss, a close relation can be obtained. This type of models of the load-response of the building stock can be a tool for determining the response of the building stock in the present climate as well as in a climate in change.

4.4 IFC based geographical information-IFG

With the rapid development of IFC based standards for digital object oriented models of building products there is a huge need for property sets, such as durability and service life data, linked directly to the building elements. The significant drive within the AEC/IFC community to provide for relevant location based data (GIS) via IFC format (IFG) will be a major facilitator for access to site specific durability data, described by degradation models containing environmental (and other) degradation factors [Wix et al, 2005].

This will be essential for example for facilitating a seamless, internet based zoning and building plan permit. Key to this development is the integration of GIS information in a central building and property registry with AEC/FM information about the individual buildings that are registered
With the use of the IFC/IFD/IFG technology these functions and the resulting service life for the product can now be directly linked to the building element in question.

5 REFERENCES

CIB report Publication 256, 2000, *GIS and the Built Environment,* Haagenrud, S.E., Rystedt, B., Sjöström, C. (eds.), October 2000, Rotterdam.

Cole, I.S., Paterson, D.A. and Ganther, W.D., 2003, A holistic model for atmospheric corrosion-Part 1-Theoretical framework for production, transport and deposition of marine salts, *Corrosion Science, vol 38, issue 2.*

DARTS, 2004, Durable and Reliable Tunnel Structures, Draft-Final Technical Report: "Comprehensive Decision Tool for Durable and Reliable Tunnels", The European Union - Fifth

Framework Programme, Competitive and Sustainable Growth Programme (Growth 2000). Project GRD1-25633, Contract G1RD-CT-2000-00467, June 2004, pp 98.

DuraCrete, 2000, Probabilistic Performance based Durability Design of Concrete Structures, Report No. 15: General Guidelines for Durability Design and Redesign. EU – Brite EuRam III. Contract BRPR-CT95-0132, Project BE95-1347, Document BE95-1347/R15, February 2000, pp 113.

European Commitee for Standardisation-CEN (2001)- EN 206-1 Buildings and Infrastructures – Part 1 Specification, performance, production and conformity

European Organisation for Technical approvals, 1999, *Assessments of working life of products.* PT3 Durability (TB97/24/9.3.1).

Hallberg, D. 2005, Quantification of the exposure classes in The European Standard EN 206-1, In 10DBMC, Lyon, France

Hohl R, Schiesser HH, Aller D, 2002, Hailfall: the relationship between radar-derived hail kinetic energy and hail damage to buildings, *Atmspheric Research 63 (3-4): 177-207*

International Organisation for Standardisation, 1984, Performance standards in buildings – Principles for their preparation and factors to be considered. Geneve (ISO 6241-1984).

International Organisation for Standardisation (1992) Corrosion of metals and alloys – Corrosivity of atmospheres - Classification. Geneve (ISO 9223:1992).

International Organisation for Standardisation, 2000, Service life planning of buildings and constructed assets-Part 1 –General principles Geneve (ISO 15686-1, 2000).

ISO/WD 15686-4, 2002, Buildings – Service Life Data Sets – Part 4 – Service Life prediction data Requirements.

Jernberg, P., Lacasse, M.A., Haagenrud, S.E. and Sjöström, Ch, 2004, *Guide to Bibliography to Service Life and Durability Research for Building Materials and Components*, CIB W080/RILEM 175–SLM Publication 295, CIB, Rotterdam.

Kucera, V. and Gregor, H.D., 2000, *Mapping Air Pollution Effects on Materials Including Stock at Risk-Proceedings of a UN ECE Workshop*, SCI Bulletin 108E

Lay, S. Schissel, P. Cairns, J. (2003) *Instruction on methodology and application of models for the prediction of the residual service life for classified environmental loads and types of structure in Europe*, Lifecon deliverable D3.2 Final Report, Public usage, Lifecon G1RD-CT-2000-003788

Lisø, K.R., Kvande, T., Thue, J.V. and Harstveit, K. 2004, A Frost Decay Exposure Index for Porous, Mineral Building Materials, *Journal of Thermal Envelope and Building Science* (submitted).

Lisø, K.R., Aandahl G., Eriksen S. and Alfsen K.H. 2003, Preparing for climate change impacts in Norway's built environment. In R. Lorch (ed.) *Build. Res. & Information*, 31(3-4), pp. 200-09.

Rydock, J.P., Lisø, K.R., Førland, E.J., Nore, K. and Thue, J.V. 2004, A driving rain exposure index for Norway, *Building and Environment* (submitted).

Sarja, A., 2004, EU project "G1RD-CT-2000-00378 Lifecycle Management of Concrete Infrastrucures for improved sustainability (LIFECON)". Final report, VTT, Helsinki.

Sjöström, Ch., Jernberg, P., Caluwaerts, P., Kelly, S., Haagenrud, S.E., Chevalier, J.L. 2002, Implementation of the European Construction Products Directive via the ISO 15686 standards, *9th Conference of Durability of Building Materials and Components, Brisbane 2002.*

Thiis, T., Lahus, O., 2003, Prolonged service life of concrete structures with CFD simulations of environmental exposure, *Concrete the sustainable construction material - Technical opportunities for the future, conference, Oslo 2003.*

Thiis, T., Einstabland, H., 2005, Damage functions for wind induced building damage based on meteorological and insurance loss data in Norway. *10DBMC International Conférence On Durability of Building Materials and Components, Lyon 2005*Tidblad, J. and Kucera, V., 2002, *Mapping of acid deposition effects on materials in Sweden*, Paper no 39, 15th International Corrosion Congress, Granada (Spain), 22 to 27 September 2002.

J. Wix, L. Bjørkhaug; Ø. Rooth, J. Sjögren (2005) Industry Foundation Classes – Facilitating a seamless zoning and building plan permission. *10th Conference of Durability of Building Materials and Components, Lyon, 2005.*

Providing Life Cycle Planning services on IFC/IFD/IFG platform- a practical example

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ABSTRACT

With the rapid development of IFC[1] based object oriented models of building products there is a great need for standardised data that can be linked directly to the building elements. Data for service life, life cycle analysis (LCA) and life cycle costing (LCC) are typical data necessary for an improved life cycle based planning of buildings and constructed assets. The significant drive within the IFC community to provide support for Geographic Information Systems (GIS) based data via IFC format will be a major facilitator for access to such type of data for the specific building site.

ISO 15686[2] provides guidance on steps to be taken at various stages of the building cycle to ensure that the resulting building, or other constructed facility, will last for its intended life without incurring large unexpected expenditures. Of particular importance is the concept of Reference Service Life (RSL) and the Factor method (Fm) for estimation of Service life in specific projects, as contained in Part 8. The Factor method is used to modify a RSL to obtain an estimated service life (ESL) of the components of a design object, while considering the difference between the project-specific and the reference in-use conditions, thus overcoming the mismatch of the in-use conditions.

This paper describes the implementation of the ISO 15686 Servivice life planning (SLP) process on an IFC platform extended with GIS information throug IFG[3] and knowdlege deliverd throug use of IFD[4] based reference data. A simplified service at the Norwegian Building Research Institute[5] is developed and delivered, using open standards. The user is given a web based application, where an IFC model from the project can be uploaded. Enriching the model with lifetimes for materials used in the building, demonstrates the compliances with the ISO 15686, and adds value in terms of much improved information exchange in the building process resulting in higher quality and lower cost for the construction.

KEYWORDS

Life Cycle planning, Product modelling, Industry Foundation Classes (IFC), Service life, Geographical Information Systems (GIS)

1 INTRODUCTION

[1] IFC - Industry Foundation Classes – ISO/CD PAS 16739 - Exhange standard developed by International International Alliance for Interoperability (IAI). www.iai-international.org
[2] ISO 15686 - Buildings and constructed assets -- Service life planning
[3] IFG - Industry Foundation Classes for GIS - A
[4] IFD - International Framework for Dictionaries – ISO/DIS 12006-3 – currently being harmonized with IFC.
[5] The Norwegian Building Resarch Institute – Byggforsk – www,byggforsk.no

With the rapid development of IFC based standards for digital object oriented models of building products there is a great need for standarised data that can be linked directly to the building elements. Data for service life, life cycle analysis and life cycle costing are typical data necessary for an improved life cycle based planning of buildings and constructed assets. The significant drive within the International Alliace for Interoperabiliy (IAI) to provide for relevant location based data throug the IFG project will be a major facilitator for access to such type of data for the specific building site.

The ISO 15686 standard provides guidance on steps to be taken at various stages of the building cycle to ensure that the resulting building, or other constructed facility, will last for its intended life without incurring large unexpected expenditures. Of particular importance is the concept of Reference Service Life (RSL) (i.e.-the expected service life in a well defined set of in-use conditions) and the Factor method (Fm) for estimation of Service life in specific projects, as contained in Part 8. The Fm is used to modify a reference service life to obtain an estimated service life (ESL) of the components of a design object, while considering the difference between the project-specific and the reference in-use conditions, thus overcoming the mismatch of the in-use conditions.

2 DESCRIPTION OF THE IFC-IFD-IFG CONCEPT

2.1 Background

The IFC standard has been developed by the International Alliance for Interoperability (IAI) for the building sector, based on the generic EXPRESS[6] standard. IAI is an alliance of organizations within the construction and facilities management industries dedicated to improving processes within the industry through defining the use and sharing of information. Organizations within the alliance include architects, engineers, contractors, building owners, facility managers, manufacturers, software vendors, information providers, government agencies, research laboratories, universities and more.

EXPRESS is the data modelling language in wich the IFC, IFG and IFD standard are based. International Framework for Dictionaries (IFD), extends IFC to include unique and specific definitions of concepts used in the building industry. While the IFC standard describes objects, how they are connected, and how the information hould be exanged and stored, the IFD standard uniquely describe what the objects are, and what properties, units and values they can have. IFD provides the dictionary, the definitions of concepts, the common understanding necessary for the communication to flow smoothly. The IFD standard gives each concept a global unique identifier, or "personal number" and a set of names and definitions in different languages. While the identifier is used to capture the information the names and definitions are used merely for communication the idea to the end user. IFD is a multilingual dictionary where identical concepts carry the same identifier regardless of language.

The current building process is primitive in terms of information exchange. Various stages in the building process, although depending on the same information as other stages, will normally not share that information. The information is re-entered, re-invented, or worse, wrongly re-invented or re-entered, at each stage. A large amount of resources is wasted in these transactions, and errors are made, resulting in building damages, shortened lifetime and higher costs. There is a strong need to find an alternative way of working.

The IFC format allows, among other things, two or more unrelated applications in the building process to exchange information about the underlying model the project wishes to realize. It thus supports the move away from drawings and onto real 3D models, allowing for a continously enrichment of the model with relevant information throughout the building process.

[6] EXPRESS ISO 2001, Product data representation and exchange, ISO/DIS 10303-11, Is the latest version of the standard. EXPRESS is used for most of the STEP (Standard for Exchange of Product model data) standards.

Within the IAI framework several pilots and –development projects are paving the way for implementation of IFC/IFD based models within the sector. Thus the IFC standard has already reached an implementation level where it now proves real value for users in projects. The US General Services Administration (GSA) has announced that it will require drawings and plans to be delivered on IFC format from 2006 on. In Norway the Directorate of Public Construction and Property (Statsbygg) is already allowing (and promoting) the same [Kvarsvik et al, 2005]

The IAI developers have defined and included concepts and properties related to a whole range of facility management aspects, such as life cycle costing, environmental impacts and service life, relying and referring to international standards, such as ISO 15686.

2.2 The Norwegian IFD dictionary "BARBi"

"BARBi[7]" is an acronym play on the Norwegian word for a reference library for the Norwegian Building and Construction Industry. It is a fully compliant with the IFD standard. This allows producers, engineers, architects and users to completely define products and concepts within a standardised framework. The product will be available through an open and clearly defined Application Programming Interface (API), and be free for use by the Norwegian Building Industry. The API is in the process of being internationally standardised through the work of the IFD organisation[8]. The IFD organisation will publish an API to all available IFD libraries such as BARBi, LexiCon[9] and EDIBATEC[10]. The goal is to synchronize the definitions within all other national IFD based dictionaries, As of April 2004 the open part of Netherlands' LexiCon where fully integrated with the Norwegian BARBi library.

The database foundation for the BARBi platform is native EXPRESS and can thus easily accommodate and comply with the standard. EPM Technology's product, EDM[11], provides this database engine.

The API's used for accessing the database are currently under development, but will be based on standard web technology, and can thus be implemented using a large range of technologies and programming languages. A simplified API exists today, using standard HTTP URLs and XML. A web interface is also currently being developed to allow the Norwegian community to populate the database, and add terminology as more and more domain experts get together to define their domain specific products and concepts.

[7] BARBi – The Norwegian Building and Costruction Reference Data Library- www.barbi.no
[8] IFD – Organisation for International Framework for Dictionaries,www.ifd-international.org.
[9] LexiCon – The Dutch STABU foundation reference library - http://www.stabu-lexicon.com/
[10] EDIBATEC- French dictionary compliant with IFD - http://www.edibatec.org/
[11] EDM, EXPRESS Data Manager, EPM Technology – www.epm-technolgy.com

The BARBi database currently contains the following:

- 9 different languages
- 1000 activities or processes
- 40900 names
- 6600 definitions
- 40 actors or roles
- 2000 properties
- 9000 objects and subjects
- 700 groupings
- 2000 relations between concepts

2.3 IFC based geographical information-IFG

One recent development project within the IAI network is the development of IFC based links to geographical information (IFG[12]) [Wix et al, 2005]. This will be essential for example for facilitating a seamless, internet based zoning and building plan permit. Key to this development is the integration of GIS information in a central building and property registry with information about the individual buildings that are registered. For a building that is to be developed, information is taken from the registry to provide information about location map data including property data, utility services, demographics, zoning, risk factors. IFG extends IFC to seamlessly integrate with GIS based systems. The goal is to provide relevant information from GIS to IFC based building models while updating GIS data with relevant information from the IFC building information model.

2.4 Byggforsk Knowledge Base

The Norwegian Building Research Institute (Byggforsk) has for over 50 years developed and collected knowledge about buildings. One part of the knowledge base is the Byggforsk design sheets, unique and highly regarded in the Norwegian Building Industry, with over 800 topics, describing best practices, research results, and experience, which is frequently used in the construction industry. They are used by professional carpenters, lawyers, builders, amateurs, etc., for practical advice on the construction site, as legal foundation for contracts or as knowledge support during various phases of a building project.

Byggforsk knowledge base is currently kept in an internal electronic format, optimized for publishing design sheets as paper documents, on CDROM or on the web.

3 BYGGFORSK'S IFC/IFD/IFG SERVICES ON SERVICE LIFE PLANNING

3.1Introduction

Byggforsk is currently developing several services for the end users, including the way to deliver these services to ensure openness, usability and effectiveness. Many requirements must be met at the same time, and some pilot case studies are most likely necessary to ensure a service that the market needs.

As a proof of concept, a tiny fraction of the information in our knowledge based system, lifetime of materials, was taken and stored the information using IFC and IFD technology. As an example, we have stored a list of materials, using definitions and concept IDs from BARBi, in the IFC format. This information is gathered according to the ISO 15686 concept.

[12] Ifc for GIS. GIS workgroup in IAI. http://www.iai.no/ifg/

3.2 ISO 15685 methodology

Service life planning of a design object involves the estimation of service life of its components (ESL), and compliance with the requirement of estimated service life being ≥ design life. If not other types must be considered untill the criterion is satisfied [ISO 2000].

In order to achieve an appropriate estimated service life, there is a need of modifying a reference service life available by taking the differences between the object-specific in-use conditions and the reference in-use conditions into account. The Factor method [ISO 2000 and Jernberg et al, 2004] provides a systematic way of carrying out such a modification.

One straightforward way of applying the Factor method is to multiply a reference service life *RSL* by a number of modifying factors *A* to *G*, each of which reflecting a difference between the object-specific and reference in-use conditions within a particular factor class, [Table1], thus obtaining an estimated service life *ESL*:

$$ESL = RSL \times A \times B \times C \times D \times E \times F \times G.$$

Factor class	Designation
A	inherent performance level
B	design level
C	work execution level
D	indoor environment
E	outdoor environment
F	usage conditions
G	maintenance level

(1)

Table 1. Factor classes

For the application of the Factor method, except from knowledge of *RSL* itself, of course also information of the reference in-use conditions as well as the object-specific in-use conditions must be available in order to allow estimation of the modification. Thus, the reference in-use conditions should be provided together with the *RSL*, while the object-specific in-use conditions are determined from the knowledge of the design object and site.

3.3 Service life functions and environmental data

The Byggforsk knowledge base contains a range of dose-response functions or damage functions relating the degradation to the exposure environment; e.g. addressing the factor E, as well as relevant exposure data. Those have been obtained from various sources [Jernberg et al, 2004]. One such source is from the UN ECE programme for mapping of environmental parameters and of areas and stock of buildings with increased risk of corrosion. This program is performed and co-ordinated for Europe within the UN ECE Working group on Effects under the Convention of Long Range Transport of Air Pollutants, and specifically under its International co-operative Programmes (ICPs), for Modelling and mapping, and for Materials (ICP Materials), respectively [Tidblad and Kucera, 2003].The dose/response functions for unsheltered materials are of the type

$$K = dry\ (T, Rh, [SO_2], [NO_2], [O_3], t) + wet\ (Rain\ [H^+], t)$$

(2)

where K is the corrosion attack, T is the temperature in degree C, Rh is the relative humidity in %, [] is the concentration in µg/m^3 (SO_2, NO_2 and O_3), t is the time in years, Rain is the amount of precipitation in mm and [H^+] is the acidity of precipitation in mg/l. The corrosion attack can, depending on material, be quantified as either mass loss (ML, g/m^2), surface recession (R, µm), ASTM D 1150-55 1987 (ASTM, 1-10), depth of leached layer (LL, nm) or weight increase (WI, µg/cm^2).

Such dose-response functions have been mapped for many places in Norway by use of existing environmental data, [Figure 1]

With the use of the IFC/IFD/IFG technology these functions and the resulting service life for the product can now be directly linked to the building element in question.

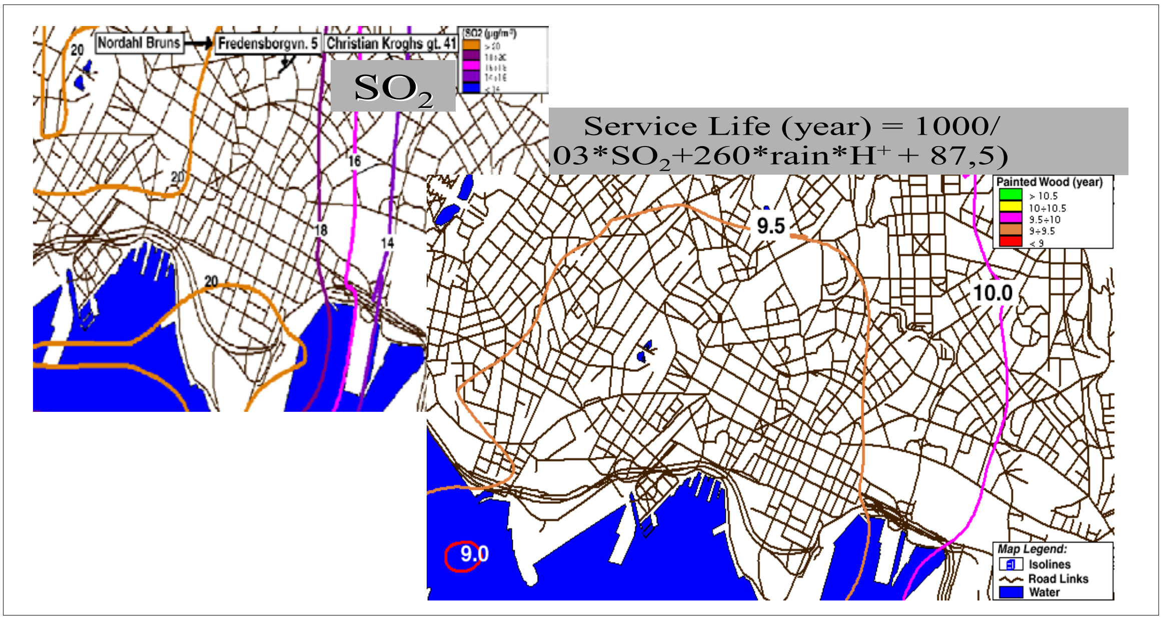


Figure 1 Modelling distribution of SO2 and resulting service life of painted wood in Oslo based upon the damage function SLPwood (year) = 1000/ (1,03*SO2+260*rain*H+ + 87,5)

3.4 Proof of Concept

Again, as a proof of concept we developed a small service targeting human users directly, using web technology. The service chosen was lifetime of building components.

Starting with a model in IFC format, most likely from a model server, the user must upload a partial model of the building, containing the parts she wants to estimate the lifetime for. The upload is done using the standard IfcXml format. To estimate lifetime, the user must first choose the location of the building (or perhaps *planned* location), and then through a 3D viewer[1], click on the part of the building for which the lifetime is required, [Figure 2]

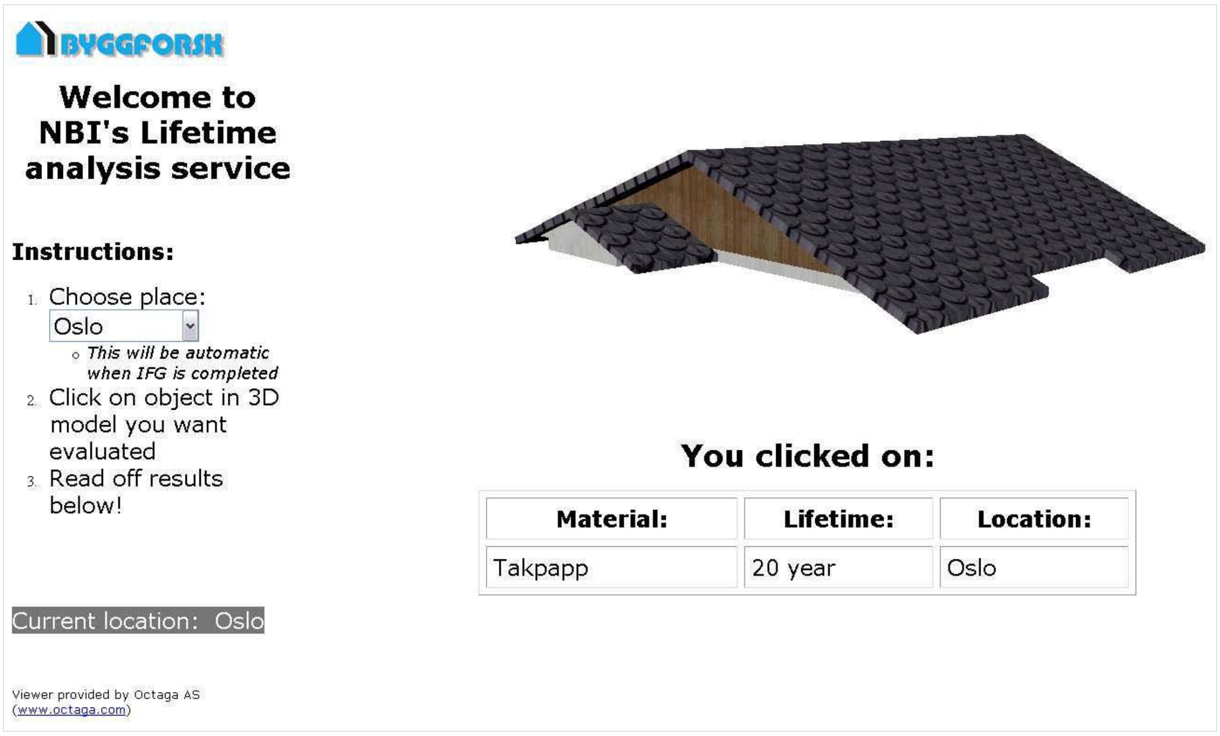
When the user clicks on a building object in the 3D viewer, the system does the following:

- Checks with BARBi what kind of material the object clicked on contains
  Looks up in the Byggforsk knowledge base for the lifetime of the material, using the material and environmental definition from BARBi.

Region	SO_2 (µg/m_3)	O_3 (µg/m_3)	H+ (mg/l)	Rain (m/year)	ToW
Halden	5,0	55,0	0,040	0,80	0,38
Sarpsborg	16,0	50,0	0,032	0,88	0,39
Fredrikstad	7,0	50,0	0,032	0,79	0,38
…….					
…..					
Oslo	5-24	17-51	0,025	0,60	0,32

Table 2 Air quality and climatic data for regions in Norway

- In this case the material is "bitumen" with the following damage function
  $$t = 1000/47{,}7 + 0{,}327\,(SO_2) + 0{,}080*Rain(H^+) \quad \textbf{(2)}$$
  and environmental data picked from the [Table 2] above, for Norway.
- Returns the lifetime to the user
- The user can proceed according to the ISO 15686 design process, choosing in the end a material/component complying with the requirement of estimated service life being ≥ design life



We assume that the IFC model contains material ids from BARBi.

Three completely stand alone technologies come into play in this demonstration:

- BARBi, IFD library
- Byggforsk Knowledge Base
- Byggforsk Service

They are all necessary to deliver the end result for the user, and their interaction is made possible due to IFC and IFD.

4 CONCLUSIONS

An example of a simplified Byggforsk Service is developed and delivered, using open standards such as IFC and IFD. The user is given a web based application, where she can upload part of an IFC model from her project, enrich the model with lifetimes for materials used in the building and thus increase the information available to the project.

The power of IFD is demonstrated, since the underlying calculations depend on data delivered on IFD format, which in this case are data from the Norwegian BARBi library. It is also shown that all the independent technologies can smoothly work together, provided they are used in a compliant way.

The IFD standard is necessary in addition to IFC to allow for conceptually and semantically understood and clearly defined information exchange. It no longer matters what language you are familiar with, knowledge and experience based data can be available to end users in productive ways, as long as the IFD concept is properly used.

5 REFERENCES

All referenses to products and standards are given as footnotes in the paper when itroduced. The following is a list of references to reports and publications.

CIB report Publication 256 2000, *GIS and the Built Environment,* Editors. Haagenrud, S. E Rystedt, B, Sjöström, C, Oct 2000, Rotterdam.

Cole, I. S., King, G. A., Trinidad, G. S., Chan, W. Y. and Paterson, D. A. 1999, An Australian-wide map of corrosivity: A GIS approach. In 8th DBMC, Canada.

Jernberg, P., Lacasse, M.A., Haagenrud, S.E. og Sjöström, C. 2004, *Guide to Bibliography to Service Life and Durability Research for Building Materials and Components.* CIB W080/RILEM 175–SLM Publication 295, CIB, Rotterdam, 2004.

Tidblad, J. and Kucera, V. 2002, Mapping of acid deposition effects on materials in Sweden Paper no 39, 15th International Corrosion Congress, Granada (Spain), 22 to 27 September 2002.

Wix. J, Bjørkhaug, L., Rooth, Ø., Sjögren, J. 2005, Industry Foundation Classes – Facilitating a seamless zoning and building plan permission. 10th Conference of Durability of Building Materials and Components, Lyon, 2005.

Bell, H., Bjørkhaug, L., Rønning, J. 2004, ICT- platform for Object Oriented Knowledge in the Building and Construction Industry. B4E Conference “Building for a European Future”, Maastricht, 14. Oct 2004.

Kvarsvik, O. K., Thi Lê, M. A., Rognlien, S., Mohus, F. 2005, Project management and the building process – A holistic view using IFC as a tool and carrier of information. 10th Conference of Durability of Building Materials and Components, Lyon, 2005.

Quantification of exposure classes in The European Standard EN 206-1

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ABSTRACT

The recently completed EU-project Life Cycle Management of Concrete Infrastructure for Improved Sustainability (Lifecon) has developed a generic and predictive Life Cycle Management System (LMS) for maintenance optimisation and planning of buildings. The system facilitates the change of today's reactive practice of maintenance management into a predictive life cycle based maintenance management system. To enable simplified prediction of service life and maintenance interval in such a predictive life cycle management system, a quantitative classification system for environmental loading is needed. At present there are a number of standards containing quantitative classification of environmental loading onto structures and building materials, e.g. ISO 15686-4, EOTA and ISO 9223. The governing standard for concrete structures such as bridges and tunnels is the European Standard EN 206-1 Concrete – part 1: Specification, performance, production and conformity. This standard divides the environmental loading into 18 exposure classes, which cover environmental loads from atmosphere, seawater, fresh water, groundwater and soil, but also the decisive parameters for moisture and chlorides. Almost all exposure classes within the standard include only qualitative descriptions. To make the standard EN 206-1 valid for LMS the standard has to be further developed into a quantitative classification system for environmental loading. A proposal of a quantitative classification of the exposure classes within the standard EN 206-1 regarding corrosion induced by carbonation is presented in this paper. The proposed classification is partly based on the extensive work performed in the Lifecon project, partly based on literature studies. The proposed classification is validated through comparison of real measurements made on a bridge located in Sweden and calculations using a full probabilistic degradation model. It is believed that such exposure classification is possible to use in a LMS to provide simplified service life analysis and possibilities to map the risk of degradation.

KEYWORDS

Quantitative classification, exposure classes, carbonation, EN 206-1

1 INTRODUCTION

In the beginning of 2001 the EU-project, the G1RD-CT-2000-003788 "Life Cycle Management of Concrete Infrastructures for Improved Sustainability" (Acronym: Lifecon) was launched. The aim of the project was to develop a European model of a predictive and generic Life Cycle Management System (LMS) and to promote the turn of today's reactive approach of maintenance management towards a predictive life cycle based approach [Söderqvist & Vesikari 2003]. LMS is aimed to support all types of decision-making within maintenance management of buildings and infrastructures. It will include optimal planning of maintenance, repair and rehabilitation (MR&R) over time. The system includes an inventory part and condition assessment part, a service life and maintenance analysis part and a maintenance optimisation and planning part. The system is predictive, which means that it is possible to predict the service life performance of a building or a component over a period of time. Consequently, it will be possible to find the most appropriate maintenance action and intervals. The service life performance analysis is based on degradation models developed for a number of material families. These degradation models describe the material degradation rate affected by environmental loads. Simplified service life analysis cannot be done without systems for characterisation and quantitative classification of environmental loads. At present there are a number of standards containing quantitative classification of environmental loads onto structures and building materials e.g. ISO 15686-4, EOTA and ISO 9223 [Haagenrud & Krigsvoll 2003]. The governing standard for concrete structures such as bridges and tunnels is the European standard EN 206-1 [EN 206-1 2000]. This standard contains a qualitative classification system of environmental loads describing 18 different exposure classes. When to utilise the standard in service life performance analysis a quantitative description of the exposure classes is preferred. This paper presents a proposal of possible quantitative classification of the exposure classes within the standard EN 206-1 regarding corrosion induced by carbonation. The proposal is partly based on the extensive work performed in the Lifecon project, partly based on literature studies. The proposed classification is applied on a bridge in order to validate the quantitative exposure classification. The validation is done through comparison of real measurements and calculations of corrosion induced by carbonation.

2 BRIEF INTRODUCTION TO EN 206-1

The objective of the European standard EN 206-1 is to provide a harmonised Euro standard that will work as a base for CE-marking of concrete [EN206-1 2000]. The standard is valid for concrete for in-situ constructions, prefabricated constructions and for prefabricated structural elements. The requirements of concrete due to its performance to withstand influences from the surrounding environment is expressed in limit values or derived from performance based dimension methods. The requirements must take service life aspects into consideration, where the intended service life should be at least 50 years including moderate maintenance [EN 206-1 2000]. When to specify the requirements of concrete property for a certain structure, the environmental loads have to be characterised and classified due to the location of the structure. Exposure category as corrosion induced by carbonation, chlorides from seawater and from de-icing salts, freeze/thaw attack and chemical attack are classified and described. The only exposure category that provides a quantitative description of the exposure classes is chemical attack. This study will, however, focus on corrosion of reinforced concrete induced by carbonation and how a qualitative description of the exposure classes could be translated into a quantitative description. The current qualitative description of the classification due to corrosion induced by carbonation is to be seen in table 1.

Corrosion induced by carbonation		
Where concrete containing reinforcement or other embedded metal is exposed to air moisture, the exposure shall be classified as follows:		
XC1	Dry or constantly wet	Concrete inside buildings with low air humidity Concrete permanently submerged in water
XC2	Wet, rarely dry	Concrete surfaces subject to long-term water contact Many foundations
XC3	Moderate humidity	Concrete inside buildings with moderate or high air humidity. External concrete sheltered from rain
XC4	Cyclic wet and dry	Concrete surfaces subject to water contact, not within exposure class XC2

Table 1. Exposure classification due to carbonation [EN 206-1 2000]

3 INFLUENCING PARAMETERS ON CORROSION INDUCED BY CARBONATION

Corrosion of reinforcement in concrete induced by carbonation consists of two partial processes; 1) the initiation phase where the carbonation process takes place, 2) the propagation phase where corrosion of the re-bars starts and propagates. The whole process is a diverging process where a rise in relative humidity will decrease the carbonation process but increase the corrosion rate on the re-bars on condition that the surrounding concrete is depassivated. Except for material resistance parameters and relative humidity, environmental loading parameters represented by climate factors such as temperature and rain and pollution factors such as carbon dioxide, will have an influence on the degradation rate. Although carbon dioxide have a great impact in the carbonation process the exposure classes within EN-206 due to corrosion induced by carbonation only takes into account moisture factors as relative humidity and rain.

3.1 Relative humidity

Because corrosion of reinforced concrete induced by carbonation is a diverging process, affected by the relative humidity, the classification of the relative humidity needs to take both partial processes into account. Steffens *et al*. [2002] assumed that a maximum carbonation reaction rate is reached when the relative humidity exceeds approximately 90 %. Based on experimental tests they derived an empirical function that showed a distinct decrease in diffusion coefficient at a relative pore humidity of 60 %. At a relative humidity above 82 % the diffusion coefficient were only 10 % of its value for dry concrete [Steffens *et al*. 2002]. According to Neville [1995] the highest rate of carbonation occurs at a relative humidity between 50 and 70 %. In Betongrapport 11 [2002] a description of the influence of relative humidity on concrete is divided into four classes, table 2.

RH in concrete [%]	Carbonation	Corrosion induced by carbonation
< 45	1	0
45-65	3	0
65-85	2	1
85-99	1	3
100	0	1

Table 2. Risk of deterioration due to carbonation influenced by relative humidity (RH). Risk factors: 0 = negligible, 1 = low, 2 = moderate, 3 = high [Betongrapport 11 2002]

Tuutti [1982] concluded that the highest corrosion rate due to carbonation was obtained at a relative humidity of 90-95 %. He also concluded that corrosion rate induced by carbonation were insignificant at relative humidities lower than 80 %. In the Lifecon project a parameter study was performed, using a full-probabilistic model of carbonation where both the initiation phase and the propagation phase were taken into account [Lay *et al*. 2003]. The influence from average annual relative humidity were divided into five classes (0-4), see fig. 1. According to the parameter study the relative humidity has the greatest influence on the propagation between 81 and 91 % and lowest influence below 60 % and above 93 %, see table 3.

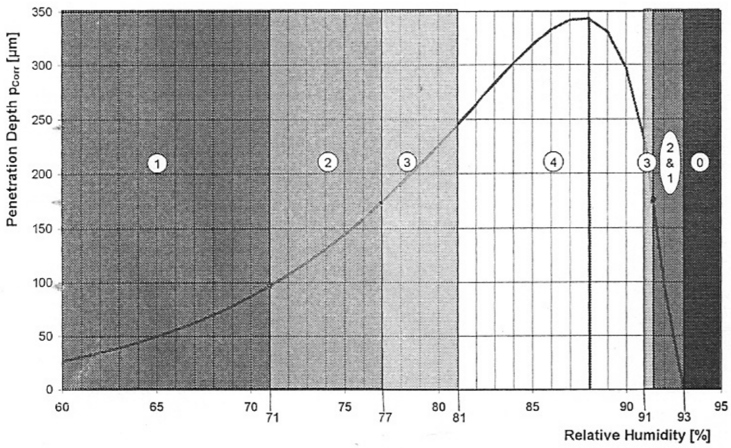


Figure 1. Variation of penetration depth in the re-bars due to the relative humidity regarding a sheltered structure 100 years of service life [Lay *et al.* 2003]

Class	0	1	2	3	4
RH (%)	< 60	60 – 71	71 – 77	77 – 81	81 – 90
	>93	-	-	90 – 91.5	-

Table 3. Classification of average annual relative humidity due to corrosion induced by carbonation [Lay *et al.* 2003]

3.2 Time of wetness

As the exposure classification due to carbonation induced corrosion within the EN 206 includes the effect from wetness this has also to be included in a quantification of the exposure classes. The time of wetness has the same diverging effect on the carbonation process as the relative humidity. The longer the concrete is exposed to wetness, the slower the carbonation rate. On the other hand, the higher the moisture content, the higher the corrosion rate. Concrete constantly exposed to water, including a surrounding relative humidity of 100 %, will not suffer due to carbonation. A simulation of the influence from the time of wetness (ToW) on corrosion induced by carbonation was performed assuming a 100-year-old horizontal structure located in an average relative humidity of 75 % [Lay *et al.* 2003]. The conclusion from the simulation is that corrosion will occur if the ToW is between 0 and 6.5 %. The ToW is defined as the quotient between the numbers of days with decisive rain more than 2.5 mm per day and the number of days per year [Lay *et al.* 2003].

4 QUANTITATIVE CLASSIFICATION

When concerning corrosion induced by carbonation it is important to take the corrosion rate into consideration. Limit values of relative humidity affecting the corrosion rate presented by Tuutti [1982], Lay *et al.* [2003] and in Betongrapport 11 [2002] is roughly summarised in table 4. Based on the limit values presented in table 4 and the qualitative descriptions in EN 206-1 the suggestion is that the exposure classes could adopt the quantitative limit values presented in table 5.

	RH limits according to:		
Corrosion rate	Tuutti	Lay *et al.*	Betongrapport 11
Insignificant	<80 or 100	<60 or >93	<65

Moderate	80-90	60-81	65-85 or 100
High	90-95	81-90	85-99

Table 4. Summary of limit values of relative humidity affecting corrosion rate induced by carbonation

Class	Qualitative description	Quantitative description, RH [%]	Quantitative description, ToW [%]
XC1	Dry or constantly wet	< 60 or ≥ 100	0 or 100
XC2	Wet, rarely dry	> 95	> 6.5
XC3	Moderate humidity	60 – 85	0
XC4	Cyclic wet and dry	85 – 95	0 – 6.5

Table 5. Proposal of quantitative description of the exposure classes due to corrosion induced by carbonation

The ToW limit values are based on the study made by Lay *et al*. [2003]. In order to prove the validity of the proposed classification, it has to be validated. In this study the validation is performed by comparison of the results from calculations and measurements. The calculations are based on the same degradation model as was used by Lay *et al*. [2003] while the results from measurements are based on carbonation depth measurements performed on a bridge located in Gävle, Sweden [Grändås 1994].

5 DEGRADATION MODEL DUE TO CARBONATION INDUCED CORROSION

The carbonation depth and the corrosion rate are estimated by using the same degradation model as used by Lay *et al*. [2003]. The model, described by Gehlen [2000], includes both the initiation phase and the propagation phase. The initiation phase is defined in eq.1. It describes the carbonation depth due to a number of parameters.

$$Xc = \sqrt{2 \cdot K_{RH} \cdot K_c \cdot (K_t \cdot R_{ACC,0}^{-1} + \varepsilon_t) \cdot \Delta C_s} \cdot \sqrt{t} \cdot \left(\frac{t_0}{t}\right)^w \qquad (1)$$

Xc is the carbonation dept at time t, K_{RH} is a relative humidity factor, K_c is a curing factor, K_t is a test factor, $R_{ACC,0}$ is the effective carbonation resistance measured in an accelerated test, ε_t is an error factor, ΔC_s is the gradient of CO_2 concentration, t is the time in service, t_0 is a reference period, i.e. when the accelerated test is performed and w is a weather exponent. In this study, the humidity factor and the weather exponent are of special interest. The humidity factor is a function influenced by the relative humidity while the weather exponent is a function influenced by the time of wetness.
The propagation of corrosion on the re-bars is defined as:

$$P(t) = V_{corr} \cdot \alpha \cdot t_{prop} \qquad (2)$$

P(t) is the progressive loss of re-bar diameter, V_{corr} is the corrosion rate related to surface area, α is the pitting factor, t_{prop} is the time during propagation ($t-t_{init}$), i.e. the time after initiation phase. Hence, the link between the two phases is the time during initiation. Redefining eq. 1 the time during initiation phase is solved, eq. 3.

$$t_{init} = \left(\frac{Xc}{\sqrt{2 \cdot K_{RH} \cdot K_c \cdot (K_t \cdot R_{ACC,0}^{-1} + \varepsilon_t) \cdot \Delta C_s} \cdot (t_0)^w}\right)^{\frac{1}{0,5-w}} \qquad (3)$$

It is assumed that during the initiation phase, no propagation of corrosion will take place. Applying Faraday's law the corrosion rate is expressed as:

$$V_{corr} = 11.6 \cdot i_{corr} \qquad (4)$$

where i_{corr} is the corrosion rate density which can be expressed as:

$$i_{corr} = \frac{k_0}{p(t)} \cdot F_{cl} \cdot F_{galv} \cdot F_{o_2} \qquad (5)$$

k_0 is a constant regression parameter, F_{cl}, F_{galv} and F_{O2} take into account the influences from chlorides, galvanic effects and availability of oxygen respectively. The p(t) factor takes into account a number of factors concerning temperature, relative humidity, chlorides, test method and curing.

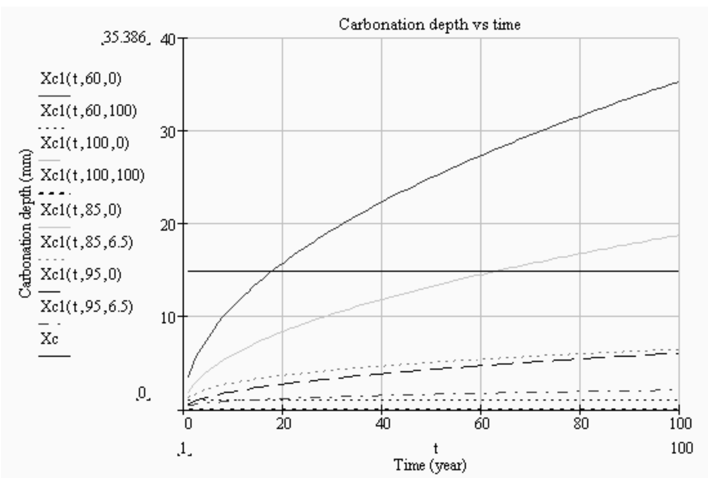
6 RESULTS FROM CALCULATIONS AND COMPARISONS TO REAL MEASUREMENTS

In 2004, the bridge in Gävle had been in service for 66 years. In 1994, i.e. after 56 years in service, carbonation depth measurements were performed on the side of the beams and on the underside of the slab [Grändås 1994]. Two samples were taken from the side of the beams and three samples from the underside of the slab. The result from the measurements is given in table 6. According to the classification in EN 206 both the beams and the underside of the slab would be classified as XC3. The result from the measurements shows, however, an appreciable difference in carbonation depth between the two structure types.

Component	Concrete cover [mm]	Exposure class EN 206	Sample	Carbonation depth underside [mm]	Carbonation depth under the surfacing [mm]
Beam	30	XC3	1	33	-
Beam	30	XC3	2	23	-
Slab	15	XC3	3	5	0
Slab	15	XC3	4	5	0
Slab	15	XC3	5	8	0

Table 6. Carbonation measurements on the bridge in Gävle [Grändås 1994]

Many parameters in the degradation model are hard to define without measurements, therefore the same values as was used by Lay *et al.* [2003] and by Gehlen [2000] are used in this validation. The input values of relative humidity and time of wetness refers to the proposed classification in table 5. According to the calculations, the carbonation depth varies between 14 mm and 26,5 mm considering a structure in exposure class XC3, made of "poor" concrete, sheltered from rain. The results well correspond to the measurements on the beams but not to the measurements on the slab. As can be seen in figure 2, the variations in carbonation depth are quite large. As an example, for a 60-year-old structure in exposure class XC3, the carbonation depth will vary from 15 mm to 27 mm. If the structure is exposed to rain, the carbonation depth will be very low, below 10 mm after 100 years of exposure. Even if calculations of corrosion propagation on the re-bars are not comparable to measurements due to the fact that there are no corrosion measurements done, it is important to present the calculation result in order to show the effect from the proposed classification. For a 56-year-old structure in exposure class XC3, made of same type of concrete as used above and with concrete cover



of 15 mm, the corrosion depth will vary from 0 to 180 μm. In figure 3 the corrosion rate varies due to different relative humidity regarding a 56-year-old structure with a concrete cover of 15 mm. The highest rate of corrosion occurs at a relative humidity around 70 to 80 %. In figure 4 the corrosion rate is presented over time for different relative humidity. The higher the relative humidity, the higher the rate of corrosion. On the other hand, the higher the relative humidity, the later the corrosion will begin.

Figure 2. Different carbonation depths due to the limit values stated in the proposed classification described in table 5

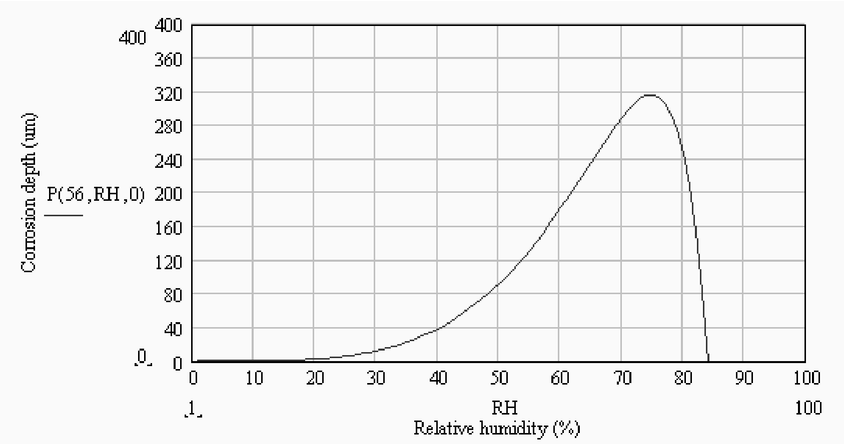


Figure 3. Varying corrosion depth due to variations in relative humidity. The concrete cover is 15 mm.

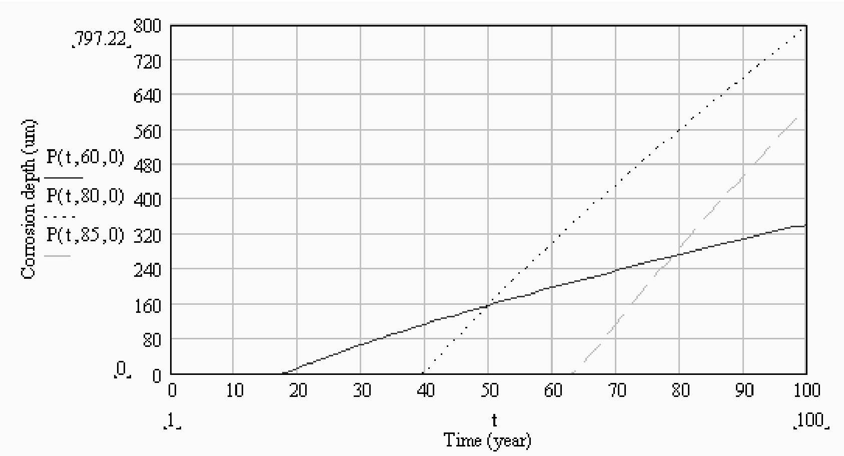


Figure 4. Varying corrosion depth over time due to variations in relative humidity. The concrete cover is 15 mm.

7 CONCLUSIONS AND RECOMMENDATION OF FURTHER WORK

A quantitative description of the exposure classes due to corrosion induced by carbonation is presented and validated through a case study of a bridge structure. It is believed that such quantitative exposure classification is possible to use in LMS in order to provide simplified service life analysis and provide possibilities to map the risk of degradation on structures. Validation of the quantitative classification shows some differences between calculations and measurements of carbonation depth. There is also some differences between the calculations made in this study and the study made by Lay *et al*. [2003]. The measured carbonation depths are different at different location of the bridge. This is expected, due to the fact that different locations of the bridge are exposed to different environmental loads. The environmental loads, exposing the bridge, have, however, not been identified nor quantified. Corrosion measurements have not been made on the bridge, which makes it impossible to compare calculations and measurements of corrosion depths. Nevertheless, the bridge demonstrate severe corrosion damages due to either carbonation or chloride ingress. It is recommended one improve the validation of the proposed quantitative exposure classification by extending the number of case studies. A prerequisite is that case studies should include well-documented structures, well-documented damages and well-defined environmental loads. The latter include mapping of relative humidity and wetness spreading at the structure surface. Such mapping, including measurements and photography, could be added in to a 3-D model of the structure in order to provide 3-D virtual demonstration of different exposure zones. The including parameters in the degradation model have to be correctly defined. Preferable by a thorough investigation of the material properties of every including structures. Other future work would be to quantify the other exposure classes in EN 206 in order to make the complete standard valid for a LMS.

8 REFERENCES

Söderqvist, M-K. & Vesikari, E. 2003, *Generic technical handbook for a predictive life cycle management system of concrete structures (LMS)*, Lifecon G1RD-CT-2000-00378 Lifecon Deliverable D 1.1, Final Report, Public usage, December.

Haagenrud, S. E. & Krigsvoll, G. 2003, *Instructions for quantitative classification of environmental degradation loads onto structures*, Lifecon G1RD-CT-2000-00378 Lifecon Deliverable D 4.2, Final Report, Public usage, October.

European Standard EN 206-1 2000, *Concrete – Part 1: Specification, performance, production and conformity* (Swedish edition SS-EN 206-1:2000), Ref. no. EN 206-1:2000 Sv

Steffens, A., Dinkler, D. & Ahrens, H. 2002, 'Modeling carbonation for corrosion risk prediction of concrete structures', *Cement and Concrete Research*, **32**[6], pp. 935-941.

Neville, A.M. 1995, *Properties of concrete*, 4:th edition, Longman, Harlow

Betongrapport 11 2002, *Vägledning för val av exponeringsklass enligt SS-EN 206-1* (in Swedish), Swedish Concrete Association, Stockholm, November

Tuutti, K. 1982, *Corrosion of steel in concrete*, Swedish Cement and Concrete Research Institute, Stocholm

Lay, S., Schissel, P. & Cairns, J. 2003 *Instruction on methodology and application of models for the prediction of the residual service life for classified environmental loads and types of structure in Europe*, Lifecon G1RD-CT-2000-003788 Lifecon deliverable D3.2 Final Report, Public usage, November

Grändås, J-Å. 1994, *Provning av betongkärnor* (in Swedish), Rapport 78940711, Vattenfall Utveckling AB Betongprovning, Älvkarleby

Gehlen, C. 2000, *Probabilistische Lebensdauerbemessung von Stahlbetonbauwerken* (in German), Deutscher Ausschuss für Stahlbeton, Heft 510, Beuth Verlag GmbH, Berlin

Industry foundation Classes - Facilitating a seamless zoning and building plan permission

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ABSTRACT

With the rapid development of IFC based standards for digital object oriented models of building products the vision is to make the zoning and building efficient and internet based. Key to this development is the integration of GIS information in a central building and property registry with AEC/FM information about the individual buildings that are registered. For a building that is to be developed, information is taken from the registry to provide information about location map data including property data, utility services, demographics, zoning, risk factors

An IAI based project was launched for developing the concept of using the Industry Foundation Classes (IFC) model as the specification for the exchange of limited but meaningful information between GIS and AEC CAD systems and vice versa. The aim was to use entities that are already established within the Coordination and Code Checking views of IFC 2x so as to be able to reuse insofar as possible the tools, techniques and capabilities already developed by vendors at the AEC side of the demonstration.

The standardisation project is recently finalised and its implications are described, and illustrated with examples. Both the AEC - and GIS world share the concept of lifecycle based information provision and will have similar approaches to portfolio and capital project development, design processes, costing and cost management, asset management, durability, maintenance, and other factors.

AEC and GIS share an interest in systems development for purposes of distribution systems. Whilst in GIS, these are generally utility systems within regional infrastructure as opposed to the local distribution mechanisms applied in AEC, the approaches for system definition are likely to be remarkably similar.

KEYWORDS

Building permit, Life Cycle planning, Industry Foundation Classes, Geographical Information Systems (GIS)

1 INTRODUCTION

The AEC/FM industry has been using computer aided design (CAD) for design and construction for more than 25 years. With a similar technology basis, geographic information systems (GIS) have been developing for a similar period. Yet whilst they share many common ideas, the markets for CAD and GIS have developed separately and in largely independent ways. This has also applied to the development of information standards in the two industry sectors, AEC CAD following the ISO STEP/IFC route whilst GIS followed the ISO TC211/OGC route.

There is clearly an interface between AEC CAD and GIS. Ultimately, a building sits in geographic space and has to know something about the space and terrain within which it exists. Similarly, geography is punctuated by buildings and other structures of interest in AEC/FM. So how does this interface get defined.

This was a question asked by the Statens Bygningstekniske Etat (BE)[1] in Norway. They recognised a cycle of information creation and capture involving both AEC CAD and GIS that contained a number of discontinuities, see Figure 1. For instance, a central property and building registry captures information about streets, buildings and their address from which information is required by a building project planning/design team. They can get the information in paper or, Norway, by the national standard SOSI format for geographic information (www.statkart.no/standard/sosi/html/sosi.htm).

From project planning and design, submission is made for planning and building code approval either on paper or using an electronic format such as DWG or DXF. Approval by the authority is typically paper based. Once the construction process is completed, there is no connection back to provide information to the central registry. Information has to be gathered separately and entered to the GIS system by hand.

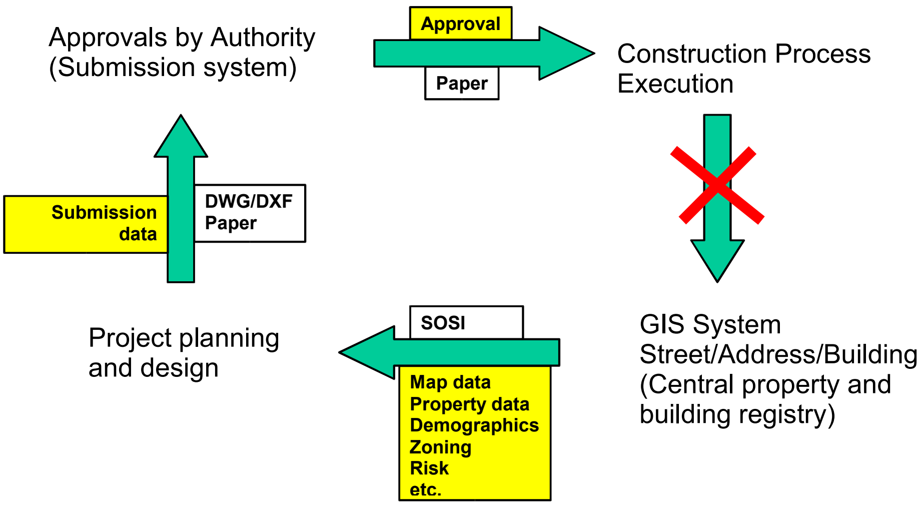

Figure 1 Traditional Building permit process

BE determined that there had to be a better way of doing this that connected all parts of the cycle in a consistent electronic way and that additionally, would support the code submission and approval process. Looking for potential solutions, they noted the development of the ePlanChecking system which is automating the submission/approval process for building planning and building services design in Singapore. After further investigation, BE determined that the same technology, if further developed to capture geographic information, could be used to meet their needs. Resulting from this, they commissioned the development of the IFC for GIS (IFG) information model, see Figure 2.

[1] National Office of Building Technology and Administration

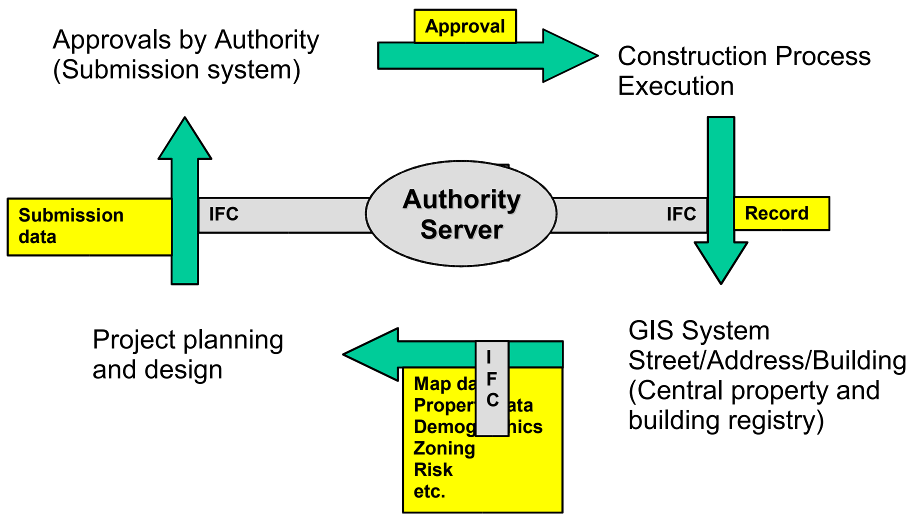

Figure 2 IFC for GIS (IFG) building permit model

2 WHY USE IFC?

There are a number of reasons why IFC was selected by BE as the basis for the IFG information model:

- IFC provides a model of the information that needs to be captured in the AEC/FM process, see Figure 3.
- It covers the entire lifecycle from initial requirements specification through design and construction and into operation and maintenance
- It is independent of any particular software application
- It has wide and growing support amongst the AEC/FM software community and, particularly important, is supported by all of the leading AEC CAD vendors
- It has been in development since 1996 and, through successive versions, has become a large, powerful and mature model
- IFC is written in the EXPRESS data definition language which is the same language as the ISO STEP standard for the manufacturing industry. In fact, IFC has adapted significant parts of the STEP standards for AEC/FM. Adoption of the geometry capability from STEP is particularly important since building design can use geometry every bit as complex as a car body or aeroplane.
- It is widely accepted as the standard information model for AEC/FM industry with a subset of the complete model being designated as an ISO/PAS standard.
- IFC uses an object oriented approach in its design, particularly the concept of inheritance. This allows ideas to be developed at a high level in the model structure and inherited by elements at a lower level in the model. This leads to making model extension quite straightforward.
- IFC has a powerful 'user extension' capability through the provision of property sets. This allows new capabilities to be added easily without having to change the nature of the model.

IFC was developed initially as an improved data exchange format for the emerging generation of object oriented CAD systems. However, it has proven to be also capable of providing the basic structure for shared databases for use on AEC/FM projects (the first production versions of which are now emerging) and also as a means for representing knowledge. It is this capability which is used in the Singapore ePlanChecking system and which attracted BE.
The principle is that all information in IFC format is object based. That is, objects have names that identify their purpose (such as wall, space, pipe, roof etc.). Each object also has attributes that provide more information. Attributes can include such things as who made it, weight, size, performance, cost as well as its geometric shape.

Building codes can be represented as rules using a version of the EXPRESS language in which IFC is specified. Objects in buildings can then be tested against the building code 'rules' to determine if they comply or fail and the result returned to the user.

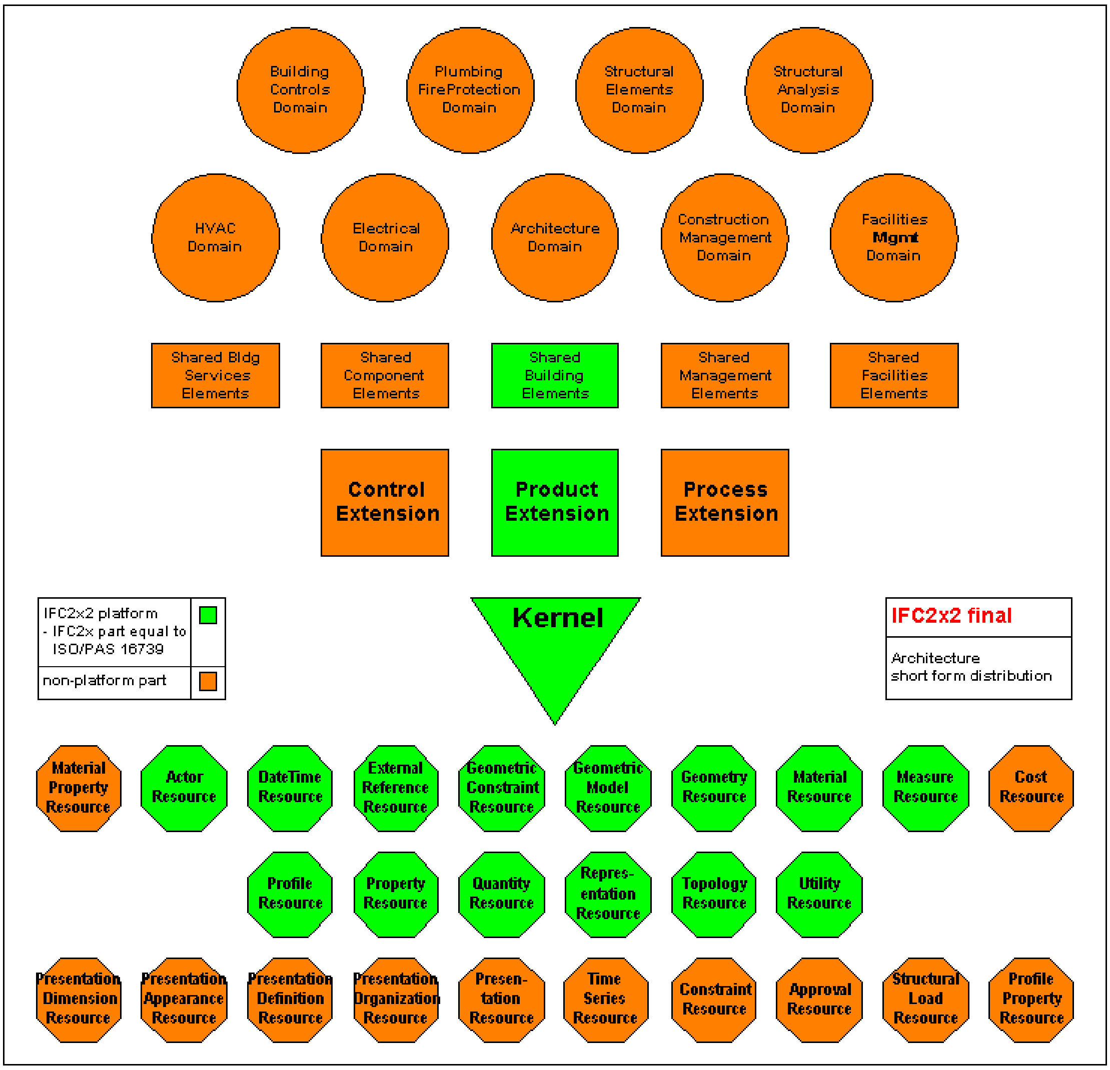

Figure 3 The IFC Building Information model

3 THE IFC FOR GIS (IFG) PROJECT

3.1 The strategic approach

The IFG project aims to provide mechanisms whereby a bridge can be made between AEC/FM systems and GIS systems. It does not seek to create a complete geographical information inside IFC. Instead, it recognizes the existence of other competent models for this purpose, notable the model underlying the Geographic Markup Language (GML) produced by the Open GIS Consortium (OGC). However, by providing the bridge, IFC facilitates the mapping of information between AEC/FM and GIS.

The approach to creating the bridge has been to create an 'overlap' between such models through the provision of entities whose occurrence and location in coordinate space can be recognized in both the AEC/FM and GIS world. Practically, this means understanding AEC/FM location information in the context of a map projection at a reasonable accuracy. Broadly, the area of overlap is seen as a one kilometre square centred upon a particular building of interest. Within this range, the maximum error at the limits of the square due to the projection of the curvature of the ellipsoid onto a Cartesian plane is estimated at four millimetres.

The following outlines the reuse of entities within IFC already that are of interest in IFG and identifies some of the new capability that has been added.

3.2 Buildings and Spaces

- IFC uses a spatial structure arrangement for organizing information according to spatial arrangements. Both the idea of a building and a space are captured within this hierarchy (together with building storey and site). All of these ideas are relevant to geographic information capture either directly or by extension of the current IFC concept:
- Building (also building complex and building section) to identify buildings (and other structures within a mapped region. Buildings can be identified according to their:
- type to indicate the purpose of the building (or other structure),
- status including whether planning is proposed or approved or whether the building has been constructed (including a history of planning applications)
- age through the provision of date/time information (also allowing negative dates for archaeological artifacts).
- footprint of the building onto the ground
- Space (also partial space) which, not only identifies internal space within a building, but can also be extended to identify external spaces required for planning purposes. The space entity can be identified as:
- a planning zone for community/municipal purposes,
- an area having special protection, or presenting certain hazards, or having particular properties (e.g. soil properties, geological interest),
- an identifiable property with legal boundaries (cadastre) and title
- a space within a property within which building is allowed
- a space for a prescribed purpose (e.g. parking)

- In addition to buildings and spaces, and of interest in a geographic context, the IFC spatial hierarchy also contains:
- Site (also site complex and part of site) as a place where work is carried out
- Building story (also partial building story) for planning purposes and also for more complex city maps
- Each type of spatial structure element in IFC has
- a name
- a description
- an object type

The Name attribute is used to store the name of the building, and building story. The ObjectType attribute should be used to specify the type of spatial structure from a dictionary or classified list of acceptable names. It is probable that such names will be defined in a standard feature catalog appropriate to the locality in which the model is used. For instance, in Norway, the type would be taken from the SOSI standard catalog (and eventually from its planned replacement).
To assist with naming, the IFG project in Norway is also integrating with the development of the BARBi dictionary which uses a standard dictionary framework defined in ISO 12006 part 3 and which is designed to be able to work with IFC and similar information models in related industries.

3.3 Distribution Systems

Building services are a significant feature of buildings and consequently, pipes and cables are already well covered by IFC. Existing concepts can be simply extended to handle the provision of utility services that need to be identified in GIS. The identification of particular systems is handled by an IfcSystem entity and this is used to group occurrences of distribution elements.

A particular issue with distribution systems however is the extent to which the components of a system are identified by a GIS system. Whilst IFC can break a system down to all of its individual parts (and go right down to the screw in a wall attaching a support bracket), such detail is not typically required in GIS. In this case, the aggregation capability of the IFC model can be used to create less rich component breakdowns with the higher orders of aggregation having separate shape representations that are more suitable to the typical requirements of GIS.

3.4 Coordinate Systems

IFC provides a set of entities that enable positioning in the Cartesian coordinate space typically used by AEC CAD systems to be related to known geodetic coordinate systems.

A general coordinate operation (IfcCoordinateOperation) handles any operation (transformation or conversion) between two coordinate reference systems. Presently, only the conversion of a local engineering coordinate system into a map coordinate reference system is dealt with. It is anticipated that future versions will expand upon this provision.

A map conversion (IfcMapConversion) deals with transforming the local engineering coordinate system, often called world coordinate system, into the coordinate reference system of the underlying map. It also allows conversion of the local origin of the local engineering coordinate system to its place within a map (using easting, northing, orthogonal height as the coordinates) and rotation of the x-axis of the local engineering coordinate system within the horizontal (easting/westing) plane of the map. A scale factor is provided for future usage.

Definition of a coordinate reference system (IfcCoordinateReferenceSystem) is by means of qualified identifiers (name of the coordinate system, geodetic datum, vertical datum) only. Interpretation of the identifier is expected to be well-known to the receiving software according to well defined authorities e.g. the European Petroleum Survey Group (EPSG)[2].
Software used to transport IFC engineering models into GIS applications (and vice versa) is expected to have knowledge about the implementation specifications of the Open GIS Consortium.

A projected coordinate system (IfcProjectedCRS) is a coordinate reference system of the map to which the map translation of the local engineering coordinate system of an AEC/FM project relates. The MapProjection and MapZone attributes uniquely identify the projection to the underlying geographic coordinate reference system, provided that they are well-known in the receiving application.

3. 5 Features

The key new entity added to the IFC model for geographic information is the IfcGeographicalElement. This allows for an occurrence of any type of feature that may be declared in a feature catalog to be used. IfcGeographicalElement is a subtype of IfcProduct and therefore inherits all of its attributes. Thus it can have several concurrent shape (geometric) representations, be classified according to whatever classification system is used, be constructed of identified materials, be grouped into larger geographic elements or decomposed into smaller elements. Most importantly, it can be defined as an type object (called IfcGeographicalElementType) which means that it can have externally specified property sets attached.

Type objects in IFC allow a definition of something to be defined once and then reused many times. Each occurrence of a type object has a unique identifier, location and orientation. Property sets that define type information are defined for the IfcGeographicalElementType. An initial property set (Pset_GeographicElementTypeFeatureCatalog) that captures principle concepts of the feature catalog is published with the IFC 2x3 and is shown below, :

[2] use of this EPSG is currently specified in several OGC Implementation Specifications

Property	*Type*	*Datatype*	*Definition*
Catalog	IfcPropertySingleValue	IfcLabel	The catalog from which the geographic element type concerned is identified
Identity	IfcPropertySingleValue	IfcIdentifier	The identity or code of the geographic element type within the catalog.
ElementName	IfcPropertySingleValue	IfcLabel	The name of the geographic element type as used in the catalog.
ElementItem	IfcPropertySingleValue	IfcLabel	A further qualification of the element name as used in the catalog. The element item may represent a subtype of the named geographic element type as in name = Building, item = House.
Description	IfcPropertySingleValue	IfcText	Narrative description providing further information about the element

Table 1 Principle concepts of the geographic feature catalog

3. 6 Qualified Geometry

Generally, geometry entities in IFC do not have their own globally unique identifier. They rely on the object to whose shape representation they contribute to provide unique identification. However, it has been recognised that, for geographic information and for planning support, there is a class of geometric entities that can be used to show a common idea. An example of this is a contour line on a map. Although theoretically, a line of constant elevation should be able to be interpreted from 3D data, GIS experts associated with the IFG project insisted that the provision of such lines as a distinct entity was a requirement.

Resulting from this, the concept of qualified geometry has been added to IFC. Qualified geometry provides a geometric entity with its own unique identifier. Entities for this are IfcAnnotationPoint, IfcAnnotationCurve and IfcAnnotationSurface. Thus, qualified geometry with 0D, 1D and 2D representation can be used (a rationale for a qualified volume has not yet been identified but, if it becomes required, it can be easily added).

Although originally developed for contours on maps, the idea of qualified geometry was quickly identified as having more general purpose possibilities. These included the identification of other equipotential lines (e.g. isolux lines for lighting, isobars for pressure etc.), survey areas and survey points, building façade outlines, ground control points, points for location of GPS recording stations and more.

Qualified geometry is identified as a type of annotation within the IFC model as it effectively represents information that would be annotated on a map. Each geometry entity is qualified as to its purpose using the Name attribute inherited from the root entity in IFC (IfcRoot).

3. 7 Terrain Modelling

IFC already contains all the facilities required to model the shape of a terrain including its representation either by a grid or as a triangulated irregular networkl (TIN). To date, these have been applied to the IfcSite entity. However, IfcSite is insufficient to fully describe the meaning of a geographic space and so the ability to describe terrain has now also been applied to occurrences of IfcSpace.

3.8 Proximity

Again a requirement of GIS experts and particularly needed for planning purposes is the proximity of one object to another. Previously, IFC had sufficient information to enable proximity to be determined geometrically (using a geometric query or a specialized geometry engine) but this was considered to be insufficient. As a result, a new proximity relationship has been added to IFC that enables not only the capture of actual distances between objects but also the required distance (minimum or maximum) as a constraint. Essentially, this provides a rule directly within the IFC model.

4 BEYOND IFG

In the above, discussion has centred entirely on the requirements for information expressed through the IFG project. However, there are also existing concepts within the IFC model that can be applied to geographical elements that extend the range of possibilities into new application areas. Because entities are being reused, this means that IFC support comes 'for free'. Maybe not exactly free since guidance has to be given to implementers and the necessary view definitions specified but at least the model does not need further extension. Two key areas are given below. They are not the only possibilities; remember that the IFC model is large, competent and mature so much more may be feasible.

4.1 Permits

The concept of a permit to allow work to be done is captured through the IfcPermit entity. This has the specific attribute of a permit identity and also points to a document within which more specific information concerning the permit can be elaborated. Permits are occurrences of the general IfcControl capability that allows constraints or control measures to be applied to specific objects and can therefore be assigned by relationship (IfcRelAssigns) to any product or process. This could be to a building (as a building permit) or to a work order (to allow maintenance work in a sensitive or secure area).

4.2 Lifecycle Information

Lifecycle information about objects within a building is as relevant as such information about buildings and other external structures. To support maintenance and facilities management requirements, IFC added information about the service life of objects and the service life factors contributing to the service life determination as part of the IFC 2x2 release. Service life data capture is based on the specification set out in the relevant ISO standard.

As well as to individual objects, IFC also allows lifecycle information about assets (since an asset is considered to be a group of objects treated as a single entity for a given purpose; usually financial or operational).

Further supporting the lifecycle concept is the capture of the condition of objects and the ability to record condition in an event based log. Constraints can also be applied to objects to define trigger conditions at which point some specified action should be taken on the object.
IFC also contains a set of entities that enable cost models to be developed and these can be applied to enable lifecycle cost to be determined (or even the anticipated cost at any point in time given the effect of various influencing factors).

4.2.1 Environmental Impact Information Capture

The cost model in IFC proved to be particularly interesting when the idea of developing environmental impact information arose. It rapidly became clear that environmental impact could be treated in the same way as financial cost (only the units needing to be changed). This led to the development of a general idea known as an IfcAppliedValue. For environmental impact assessment, it can be used to determine things like CO2 emission, sulphur effects and much more.

4.2.2 Sensors

As part of the building controls capability of the IFC model, there is an IfcSensorType entity (whose equivalent occurrence is IfcDistributionControlElement). Various sensor types are predefined including CO2, fire, flow, gas, heat, humidity, light, moisture, movement, pressure, smoke, sound and temperature. Other types can be user defined. Most sensor types also have predefined property sets that handle ranges, set points and other relevant attributes. Sensors can be applied externally as well as internally and identified as sensing various types of external conditions. In conjunction with the new geographical capabilities of the model, this offers the potential to include pollution monitoring as a supported use case (even though it is not yet included on such a support list).

5 CONCLUSIONS

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The IFG project set out to define a connection between the AEC/FM and geographic information worlds. It considered that the IFC model provided a rich basis for capturing information about buildings and their contents that could be further extended to provide that bridge. This has proven to be the case. The extent to which new capabilities have had to be added to the IFC model for this purpose has been less than was initially anticipated. The major addition required was the ability to relate Cartesian coordinates used in AEC CAD to geodetic coordinates used in GIS. By controlling the area of overlap between the two worlds, a relatively simple relationship has been defined. More complex geographic coordinate referencing is left to specialized geographic information models.

Some of the documentation and guidance provided with the IFC model has been substantially updated to reference more complex requirements that are placed upon entities by geographic information. However, this is a normal part of continuing IFC development as further richness is sought for and found to exist within the model.

As well as the requirements expressed by the IFG project, the potential to use the IFC model for further geographic data requirements has already been identified including pollution monitoring and actual building lifecycle measurement. It is likely that other capabilities will be identified.

Part of the IFG project has been early implementation testing. This has demonstrated the validity of the approach and the potential offered. The next stages of the IFG development will be to promote the bridge between IFC and GIS and for it to be implemented more extensively.

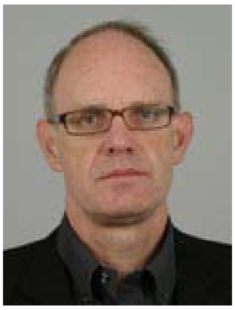
6 REFERENCES

IFC2x Edition 2 Addendum 1, International Alliance for Interoperability, http://www.iai-international.org

Liebich T., Comparison of gml3.0 and IFC2x(2), https://projectplace.com/pp/ifg, 27 May 2004

SOSI, http://www.statkart.no/standard/sosi/html/sosi.htm

From lines to object models in building design



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ABSTRACT

The IFC standard (International Foundation Classes) developed by IAI (International Alliance for Interoperability) is developed for the exchange of product model data in the building industry. This enables Building Information Models (BIM) to be exchanged between the actors of the building industry in a way which opens up for completely new forms of cooperation and organization of building projects.
With the development of the IFC standard, there will be a request for object oriented building design. To obtain maximum benefit from object orientated design there is a need for comprehensive object models from the earliest possible phase in the design value chain, i.e. the architects. When such models become available from the architects, they may be used for all purposes by other members of the design team, i.e. for calculation of energy use, design life planning, LCC analysis, electronic building permits, tendering, the contractors bidding and production planning, and for the subsequent facility management. C. F. Møller Architects is part of the design team for a major Norwegian hospital, Akershus University Hospital (AHUS). This is a large project with 116 000 m^2 new building and 20 000 m^2 renovation.
The architects have decided to establish an object model of the building instead of working with traditional drawings. The necessary paper drawings will be 'pictures' or views of the object model.
An important matter while populating object models is of course the quality assurance of the model quality. When the model is being used for e.g. quantity take off or cost calculation or energy use calculation, it has to be consistent.
Thus IFC is used to export data from the architects drawing tool to specific applications for model checking in order to secure the object models quality. IFC is also used to build a bridge between the briefing tool and the drawings, making it possible to keep track of the programmes massive 5 500 rooms and 50 000 articles (equipment, furniture and fixture) represented in the drawings.
Besides it has been decided to do a more comprehensive test for a limited part of the project where an extensive use of IFCs will be tested between as many actors and applications as possible.
We expect this project to be a reference project for large object models in the construction industry, and we expect it to give both answers and experience concerning the problems connected to large object models. In addition, it will give an answer to the usability of the IFC model in a large project, and to whether it is mature for comprehensive use among the different actors in a building project.

KEYWORDS

Large building project, building object model, IFC, briefing, model checking,

1 INTRODUCTION

The existing buildings of Akershus University Hospital outside Oslo are run down, has major concrete problems and a structure and floor height, which is unsuitable for modern hospital functionality. Thus the owner (the public health region) has, after an architecture competition won by C. F. Møller Architects, decided to build a new building for all hospital functions. Most of the old buildings will be demolished, however, parts of the old buildings will be rebuilt to house administration, university functions and a patient hotel.

The project consists of 116 000 m^2 new building and 20 000 m^2 renovation and has a budget of approximately 900 mill. €. The new hospital comprises 565 beds, mainly in single rooms, and 22 operating theatres.

The project has approximately 4 000 rooms for hospital functions, and 1 500 rooms for communication and technical purposes. Within those rooms there are approximately 50 000 articles of medical technical equipment, furniture and fixture, out of which approximately one half, 20 000 articles are represented in the drawing model.

Excavation works and foundation started in March 2004, the main part of the building project, will be finished by the end of 2007. Thereafter almost a year will be used for the installation of medical technical equipment, commissioning and gradually test use before the hospital will be officially opened in October 2008. Thereafter demolition and renovation of the old hospital will take another 3 years before the project is completed in 2011.

The existing hospital treats approximately 50.000 in-laid patients a year and has 130 000 out-patient treatments. With annual total costs of 300 million € and 4 000 employees, it is obviously most important to achieve maximal functionality and have a size and organization which minimizes all kinds of transport and communication.

The architects office has decided to work in an object based way. Thus the following description applies primarily for the architects part of the project, however, it should fit also for other members of a project team.

2 WORKING WITH OBJECT MODELS

2.1 The building information model

Even though the term building information model is new, it is not necessarily more mysterious than a number of 'drawing' files on your computer. In our project we have approximately 100 model files (plans) representing different parts of the building, organized in directories named after the buildings different section. What's makes it different is that the files mainly consists of virtual building objects, not lines and graphical objects as you would expect in drawings. The CAD tool allows us to define data objects out of building elements like rooms (spaces), walls, doors, windows, columns and beams. Everything we want to visualize is 'built' with those objects. The objects (themselves) or templates or styles for scaleable objects are established in 3D in a central object library for the whole project. They have to be assessed in a quality assurance procedure before they are distributed to the project team. Thus, the single user doesn't really have to understand all aspects concerning 3D and object modelling in order to 'draw' in 3D. The team members normally work in a 2D plan view of the model, dragging and dropping 3D objects into the drawing and adjusting them. However, in this way a complete model is built which makes it possible to view it in 3D anytime.

What is covered by one model file is decided by what is a natural functional unit and what is a natural responsibility area, combined with a look on the file size. In order to get correct sections, the architect has also established a model of the building structure, columns and slabs even though this would normally be the task of the structural engineer. To view the complete model all those approximately 100 model files has to be referenced together. Even though this is only the architect's part of the model and includes no technical installations, it results in a file structure of approximately 500 megabytes. This is of course not manageable with the available computers so for the time being we are forced to look upon parts of the model. The handling of such large models requires software, which is able to handle partly loading of the model and is intelligent enough to load the right parts of the model.

2.2 Drawings are views of the model

In order to make ordinary paper drawings, separate files are established where one file corresponds with one drawing. Those files consist of references to the necessary model files, defined views in the requested scale (plan, section, elevation), headings and ordinary drawing information.

In our project this comprises many hundred drawings (and files) for the architect.

2.3 Benefits

2.3.1 Quality assurance

The use of predefined objects, styles or templates, secures that the drawing are built up in a consistent way. The definitions of the objects, includes both the visualization and the information that is to be reported out about the different objects. As opposed to a linebased electronic drawing where a large number of team members might do things differently, this ensures that everything is done in the same way.

Instead of checking every line, it's sufficient to check object definitions and whether the correct objects have been used. As this might be reported out from the drawing in list formats, it's possible to do the quality assurance in new ways.

As the architects team in this project consist of up to 50 persons in the most intensive period where both the user participation process and the main project is running in parallel, this is an very important aspect of the object model way of working.

However, the fact that the 'drawing' contains much more information than what's visible for the eye, also arises new needs for control and checking of the models consistence. Thus the building information model establishes new needs for software and raises new demands to the software.

2.3.2 Efficiency

It's difficult to benchmark efficiency from one project to another, and it might be questionable whether we have save time in this project due to the developing and learning processes we have been through. However, there is no doubt that this process have established a building information model which should represent major savings downstream in the project organization if it could have been used in an optimal way by the engineers and contractors. There is also no doubt that the architect would have major efficiency improvements in the next project if we could use everything we have learned about objects and reuse the objects and methodology which has been established in this project.

2.3.3 Different views from the same model

The planning process includes a comprehensive user participation process where more than 50 user groups will study something like 1 000 unique rooms and 100 standard rooms in detail. For this process elevations are made for most of the rooms. Those are generated from the 3D model, securing consistency with plans and sections. This is a good example of how the model is used in many ways and it should be quite obvious that this is a major improvement of drawing efficiency.

2.3.4 Ability to change

Due to organizational changes in the public health system and budget matters, the project has been scaled down twice during the project period, in all approximately 20-25% of the area. The changes have been so thorough that the project has been more or less completely reorganized. Our experience is that the object based working order which forces through a very systematic work order, has been a major condition in order to fulfil very short timeframes. This has made it possible to keep control through very hectic periods.

2.3.5 Keeping track of articles

When also articles are objects within the object model, it is possible to report their existence and make computerized checks against the briefing tool where they are defined and keeps track of all kinds of articles.

2.3.6 Visualizing the architecture

3D modelling makes it efficient and affordable to make 3D views of different quality levels for studies, illustrations and/or communication about the project development.

3 IFC

3.1 Freedom from specific format demands

The IFC standard [ISO 2004] makes it possible to export the building information model to an open and transparent format. This enables use of different software within one project team and through different processes and through the buildings life cycle. The building owners traditional demand for specific software and delivery of files in a specific software format (normally without any specification of the information content), should be replaced by demand for a delivery in a standardized format. This should be supplied with a thorough specification of the information to be contained in the objects and delivered with the 'as built' model.

3.2 Third party software

The standardized format also enables the use of software made for very specific needs. Small software developers suddenly have a huge market if they include an option for importing from or exporting to IFC format. In this way we'll probably get new software for narrow purposes we never thought about before, giving us the possibility to serve the client in new ways. Their software might be used for special tasks by users who use large software systems for their main purposes.

3.3 Model checking

One obvious need while working with object models is special software for model checking, e.g. securing that the building information model is logical and complete. For this task we use the tool Solibri Model Checker [Solibri 2004], which is an example of new software that imports the model from IFC format. It allows us to check the model concerning predefined constraints or establish new and comprehensive constraint. Examples of constraints may be:
- checking whether there are duplicates of objects
- checking conflicts between objects which should not conflict
- checking whether objects have reasonable measures within expected limits

With the standardized format it should also be developed tools for specific checking of the model for the clients needs. The demanding client might do model checks himself in order to secure that the project team is working in line with his demands.

Another purpose of model checking is to secure compliance with building codes and regulations. This might be initiated by the local building authorities, which might establish mandatory checks with specific constraint sets in order to obtain a building permit, like what is done in Singapore [Singapore 2004]. Alternatively, in systems where the responsibility to a larger degree is delegated to the project organization, suppliers of industry software might sell checking tools, which secures and documents that the building is according to codes and regulations.

3.4 Quantity take off

Another obvious use of the building information model is quantity take off for calculation or for bills of quantities for tendering, bidding or other contract related purposes. The standardized format allows this to be done by different actors and with any tool that may import the IFC format. Thus quantity take off for calculation, could be done by the architect or the building owner while quantity take off for bills of quantities could be done by any actor according to the project- and contractual organization. In Norway tendering documents traditionally contains bills of quantities, thus we also use those tools for quantity take off in our project.

This possibility to do spontaneous quantity take offs at any time should change the attitude in construction processes where there traditionally has been lots of mistrust connected to the quantities and change of quantities.

3.5 Linking programme and data model

In this project the building owner has established a comprehensive programme database where the requirements from the briefing process concerning rooms, their functions and need for equipment are stored. With the large number of rooms and articles it is a challenge in itself to secure consistency between the database and the data model. In order to bridge this gap between the programme and the data model, the programming database has been made IFC compliant. This makes it possible to run automated checks concerning the consistence of rooms and articles in the database and the data model.

It also turns possible to get direct access to information from the other information source; when a user has a room open in the database it's possible to look directly into the same room in the model and the other way, it's possible to pick a room or article in the data model and get access to all information about the same object in the programming database.

4 FUTURE ASPECTS

Digital 2D drawing is in many ways not more than an efficiency improvement of a traditional, task based design process, known from the days when building where drawn by hand. So even though the drawing media has been digital for a couple of decades the structures and procedures has not changed radically.

Designing and building by BIM, opens a window to a world of new possibilities. Today we stand on the threshold to a future generation of planning, designing and administrating. The experiences from frontline projects within the following five to ten years will have a dominant influence on the agenda for the future building process.

Our work with object based modelling, has generated a huge interest by the top management of our client for the economical potentials. We have agreed upon two test-case studies which in the present state is still in pipeline:
The idea of the test-cases studies is to test out the potentials and challenges of lifecycle management (LM) in building information modelling.
The test-case is a relative small part of the hospital, containing the main entrance and facilities for the staff. This part is chosen due to its modest scale and architectural trademark-qualities.

5 SHARING THE BIM

The first test-case study is to establish and use a shared BIM between the architect and the engineers.
So far the architect has included the information from the other parts of the project team in the model. When we work with a shared model all members of the project team will contribute with their parts of the buildings. The ownership and thereby responsibility of the building model will be shared between the architects and the engineers. Instead of coordinating all cross discipline actions in meetings, the aim is to use the BIM it self as a coordinating tool.

What the architects might loose in control by handing over a part of our responsibility we assume to gain on synergy, improved control of the shared model and automated coordination between the project team partners. The gains of working with a shared (ifc-)BIM includes the possibilities of simulations and testing a wide range of conditions such as: Statics, HVAC, energy balance, acoustics and fire risks.

6 MODEL BASED CONTRACTS

The time perspective changes when working with BIM. One has to be more specific in defining the model objects than in 2D projects. As a result of that, the digital object model contains even on an early stage information about quantities and materials. If added with a set of general details, the object model is sufficient representative for the project to invite tenders from contractors.

The EU-legislation for tendering tends to improve the competition between contractors in order to open the European market. This means that architects in increasing scale are prohibited to recommend or use specific products while detailing the project. This makes the process of detailing difficult because one can not be as specific as needed. Often it ends up with last-minute-changes of the details once the contractors are chosen. The BIM enables engagement of the contractors before the project is detailed which increases the possibility to work more specific both in detailing as well as with the building process in cooperation with contractors.

7 FROM BIM TO BUILDING

The other test-case study will be to integrate the contractors in the work with BIM. The traditional information-flow by handing over drawings and specifications which is an interpretation of the model, that again has to interpreted by the contractor in order to make their own construction drawing, includes many risk of misinterpretations and thus expensive mistakes. The BIM contains all building information approx. to a scale 1:50 (metric). When the contractors can generate their working drawings directly from a digitally checked BIM the risk of mistakes is reduced radically.

Building part information can also be streamlined directly to manufacturing machines. E.g. HVAC tubes can be produced more or less directly from the HVAC engineers part of the BIM to be delivered just-in-time and with an object specific 3D coordinate. The economic incitement of improving performance of information-flow is huge. The costs of mistakes, failures and bottle-necks in building projects is estimated by different surveys to amount to 20-30% of the total building cost. Undoubtedly this is the main challenge of the building industry.

8 EDUCATION AND SPECIALISING

A traditional task based work process goes from A to B, B to C etc. and is in general taught by experience. Specialising is traditionally divided by the different tasks; e.g. planning, sketching, designing, projecting, building etc. Working with BIM does not necessary change those specialties. But a restructuring of the work process to fit the information to the buildings lifecycle, demands additionally that the management has a broader academic view over the entire process.

Working with the Nye Ahus hospital project has also shown to demand higher technical skills of modelling and handling information. More resources are used on education and coordination the procedures.

9 CONCLUSION

Standard CAD software has now reached a level where it is possible for architects and engineers to make a 3D building information model of the buildings to be built. This makes it possible to build virtual prototypes in this industry, where there has been no tradition for prototypes.

The availability of a 3D model of the building, enables a completely new approach for the information flow. Where the tradition has been an information push from the consultants and the client to the contractors, this new approach opens up for an information pull, all the actors may extract the information they need, when they need it. This is in line with contemporary industry thinking concerning just in time delivery of both the information and the physical products.

10 REFERENCES

ISO (International Organization of Standardization) 2004 ISO/CD PAS 16739 IFC 2.x Platform specification - International Alliance for Interoperability
[Solibri 2004] Solibri Model Checker, http://www.solibri.com
[Singapore 2004] http://www.bca.gov.sg/ http://www.corenet.gov.sg/

CIB W106 Geographical Information Systems – Work Period Report

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TT9-194

ABSTRACT
In the building and construction sector all features are location based. The world of geographic information and application orientation is moving extremely fast, thus challenging the building sector to facilitate and implement this new technology and applications. In order to deal with these issues CIB in 1996 established the task group CIB/TG20-GIS. Based on the report CIB 256 from this group and its recommendations, CIB in 2000 established the working commission CIB W106 "Geographical Information Systems", with the overall objectives to provide an international platform for R&D of GIS applications for the built environment, and to promote and encourage the use of GIS in the building sector. The W106 has members from 14 countries/organisations and will present its final report for work period 2001-04 due for the 10DBMC conference in 2005.

The work is divided into the following four Tasks: TG1- GIS-requirements and availability of geographic standards-, -data and infra -structures, TG2- GIS-based analysis and modelling of flow and distribution of materials in the built environment, TG3- Spatial dynamic modelling for Simulation of the interaction between the natural and the built environment, TG4- GIS in Education and Info sources. Objectives and work programme for each of these tasks are given and illustrated with examples, taken from state –of-the-art reports on the use of GIS elaborated by the participating countries/organisations.

With the rapid development of IFC based standards for digital object oriented models of building products there is a huge need for property sets, such as environmental exposure data, reference service life, service life models, factor distributions, LCA and LCC data, which can be linked directly to the building elements. The significant drive within the AEC/IFC to provide for relevant location based data (GIS) via IFC format will be a major facilitator for access to relevant durability data on the specific building site.

It is concluded that time is ripe for a broad implementation of GIS based applications in the building sector. Hence, it is recommended that the work programme of W106 for the coming working period includes a focus on support for an IFC based fully integrated design and planning process for the built environment, as well as a close link to the European based R&D frameworks for integrated life cycle management of the built environment.

KEYWORDS
Service Life, Geographical Information Systems, Spatial Data Infra-structures, Environmental characterisation

1 INTRODUCTION

In the building and construction sector all features are location-based and spatially-referenced. The world of geographic information systems (GIS) and its application in industry are moving extremely fast, thus challenging the building and construction sectors to facilitate the implementation of this new technology and the associated applications.

In order to deal with this issue, in 1996 CIB established the task group CIB/TG20 on GIS. Based on the first report from this group, [CIB 256 2000], CIB established the working commission CIB W106 “Geographical Information Systems” with the overall objectives to provide an international platform for R&D of GIS applications for the built environment and to promote and encourage the use of GIS in the building sector. Currently, CIB W106 has members from 14 countries/organisations. The progress report to the CIB World Congress in 2004 in Toronto contained overall objectives and operating strategy, as well as a summary of the National reports [Haagenrud et al, 2004]. These were based upon draft reports produced by Australia, Canada, France, Italy, Japan, Norway, and Sweden up to the W106 meeting in Milan in June, 2003. CIB W106 will present its final report for the work period 2001-04 at the 10DBMC conference in 2005.

The work in W106 is divided into the following four Task Groups:

- TG1- GIS-requirements and availability of geographic standards, data and infrastructures,
- TG2- GIS-based analysis and modelling of flow and distribution of materials in the built environment,
- TG3- Spatial dynamic modelling for simulation of the interaction between the natural and the built environment,
- TG4- GIS in education and information sources.

The objectives and work programme for each of these Task Groups are illustrated with examples taken from state-of-the-art reports on the use of GIS elaborated by the participating countries/organisations. This is available at www.cibworld.nl.

2 TG1: GIS-REQUIREMENTS AND AVAILABILITY OF GEOGRAPHIC STANDARDS, DATA AND INFRASTRUCTURE

2.1 Motivation

The world of geographic information and its application in industry is moving extremely fast, as shown by the business plan of ISO/TC211 on geographic Information/geomatics (www.isotc211.org). This is challenging the building sector to facilitate and implement this new technology and its applications.

For more than 10 years, countries and regions have tried to define and implement the concept of a spatial data infrastructure (SDI). Over the last few years, these activities have been structured in a more homogeneous way, one specific implementation is the work of the Global Spatial Data Infrastructure (SDSI) initiative [Norwegian Mapping, 2003].

The Building sector, via the IFG[1] initiative, has recently started to present their requirements to the GIS community. [Wix et al, 2005].

2.1 Objectives

The objective of TG1 is to:

- increase the understanding and usage of geographic information;
- promote the exploitation of efficient, effective, and economic use of digital geographic information, and
- contribute to a unified approach to addressing global performance requirements.

2.2 Overview of Activities concerning Spatial Data Infrastructure (SDI)

[1] Ifc for GIS. GIS workgroup in IAI. http://www.iai.no/ifg/

2.2.1 Standardisation ISO/TC211

The mandate for ISO/TC211 is to develop an integrated set of standards for digital geographic information concerning objects or phenomena that are directly or indirectly associated with a location relative to earth (www.isotc211.org). These standards specify geographic information, methods, tools and services for data management (including definition and description) or deal with acquiring, processing, analyzing, accessing, presenting and transferring such data in digital/electronic form between different users, systems and locations.

The standards development work shall link to appropriate standards for information technology and data wherever possible, and shall provide a framework for the development of sector-specific software applications using geographic data.

Many national and international agencies are actively engaged in the work of ISO/TC 211. These include national standardization bodies (27 member countries), the OpenGIS Consortium (OGC)-(www.opengeospatial.org), international professional bodies, UN agencies, and sectoral bodies.

2.2.2 EU project INSPIRE

The European Commission launched an initiative to establish a European spatial data infrastructure in 2001. This initiative is called INSPIRE- Infrastructure for Spatial Information in Europe (eu-geoportal.jrc.it/gos). The initiative aims at making available relevant, harmonized and quality geographic information for the purpose of formulating, implementing, monitoring and evaluating EU environmental policy making. At a later stage, the initiative will be broadened to other sector policy areas such as transport and agriculture, and shall eventually culminate in the establishment of a multi-sector spatial data –infrastructure. INSPIRE recognizes ISO standards as a foundation for its work.

An initial survey of web sites and literature on National Spatial Data Infrastructures (NSDI) was conducted for 32 countries (15 EU Member States, 10 Accession Countries, 3 Candidate Countries, 3 EFTA countries and 3 non-European countries). This information was compiled for 29 of the 32 countries with the help of national GI- and SDI-experts, including a series of important recommendations for the implementation of INSPIRE. Furthermore, a detailed country report with a description of the state of practice of SDI is available for all 32 countries [Orshoven 2003].

From this wealth of compiled information, the report concluded that operational NSDIs made up of the integrated components as identified in the Global Spatial Data Infrastructure Cookbook (GSDI www.gsdi.org/pubs.html) do not exist in Europe. However, various components of NSDIs are definitely in place or being developed. This happens almost exclusively in the public sector sphere of every studied European country. Driving forces are modernization of government, modernization of National Mapping Authorities (NMA) or similar institutions, creation or modernization of cadastres, programmes related to the promotion of e-government and information society, shortcomings in disaster prevention and management, and the need to enhance and make more cost-efficient administrations.

In 18 of the 32 countries, including all Scandinavian countries and most Accession Countries, a 'National Data Producer (NDP)' (that is, the NMA or a similar agency -Cadastre or Land Survey Agency), is taking the lead to: (1) coordinate its traditional geodetic and mapping activities with other data producers, and (2) interact with the major user groups of spatial data in order to better meet their needs (to a variable extent across countries but definitely most advanced in the Scandinavian countries). In this way, the agency fulfils an already existing, traditional mandate of coordination or takes up a more recent formal mandate. In both cases, the awareness raising by international initiatives such as GSDI and INSPIRE have had great influence, although the term 'SDI' is not always used.

The W106 Final Report will contain more detailed reports on the SDI work in each country
A major milestone was reached for the use of geographical information in Europe when the INSPIRE Proposal for a Directive was adopted by the Commission in July 2004 [CEC 2004]. This is a first step in a co-decision procedure that should lead to the formal adoption of the INSPIRE Directive, which

then must be implemented in every EU Member State. The INSPIRE Proposal will also be used as a starting point for the practical preparations of the future implementation of the INSPIRE Directive.

2.3 The IFC/IFD/IFG initiative

With the rapid development of IFC (Industry Foundation Classes) based standards for digital object oriented models of buildings/products, the vision of the Norwegian Building Authorities [Wix et al, 2005]. This initiative is attempting to make the planning, design, construction, commissioning and operations of the built environment more efficient and Internet-based. Key to this development of IFCs is the integration of GIS information with AEC/FM (Architectural and Engineering Construction/Facility Management) information about the individual buildings and constructions. For example, for a building in the planning phase, information can be obtained from the land registry to provide data about location, neighbouring lots, property data, utility services, demographics, zoning, risk factors. This integration is illustrated in Figure 1.

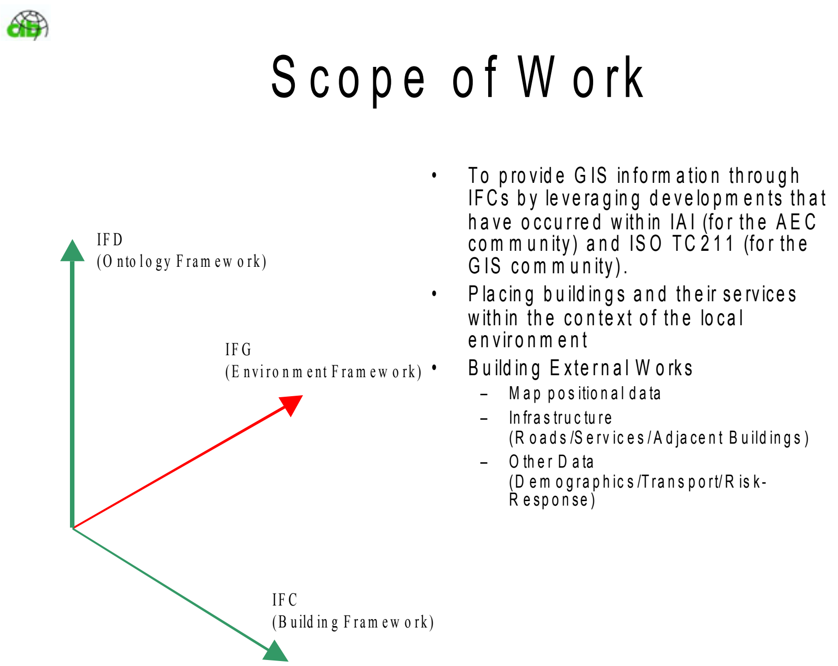

Figure 1 IFC-IFD[2]-IFG concept

An IAI based project was recently launched for developing the concept of using the IFC model as the specification for the exchange of limited but meaningful information between GIS and CAD systems [Wix et al, 2005]. The aim of the project is to use entities that are already established within the Coordination and Code Checking views of IFC 2x so as to be able to reuse the tools, techniques and capabilities already developed by vendors from the AEC/FM community .

This standardisation project was recently finalised and its implications are described and illustrated with examples in the proceedings of this conference [Wix et al, 2005]. In general, both the AEC/FM and GIS worlds share the concept of the provision of lifecycle based information and have similar approaches to portfolio and capital project development, design processes, costing and cost management, asset management, durability, maintenance, and other factors.

AEC and GIS share a common interest in systems development for purposes of distribution systems, as described later in this paper in subsection 4.2.2. Whilst GIS is currently being used in utility systems (roads, bridges, buried utilities, land planning) within regional infrastructure systems, as opposed to the local distribution mechanisms applied in AEC/FM, the approaches for data modelling and implementation are remarkably similar.

[2] IFD - International Framwork for Dictionaries, ISO/DIS 12006-3. http://www.icis.org/tc59sc13wg6

3 TG2: GIS-BASED ANALYSIS AND MODELLING OF FLOW AND DISTRIBUTION 0F MATERIALS IN THE BUILT ENVIRONMENT

3.1 Motivation

The building and construction sector is a major consumer of materials and energy resources. In the industrialized world, the construction sector is estimated to account for some 40% of the total energy consumption. In addition, construction itself produces approximately 40% of all man-made waste. The transport of building materials to the construction site is also energy-intensive and contributes to the burden on the traffic system.

Therefore, the development of GIS-based techniques for the modelling of the amount, distribution and flow of materials in the built environment is, in this perspective, a priority area.

3.2 Objectives

The objective of TG2 is to:

- promote the exploitation of geographic information technology as a tool to model the amount, distribution and flow of building materials, and
- explore the material data availability and data sharing possibilities for an efficient, effective, and economic use of digital geographic information for modelling and mapping materials (amounts, distribution, flow) on various geographic scales.

3.2 Case Study Review

France addressed "The use of GIS as a tool for waste management", in their national progress report for TG2 [Lair 2003]. The report presented the main European and French regulations concerning waste classification and transportation, especially the management of construction and demolition wastes. It also included a national plan for waste management. The implementation of this national plan would be strongly facilitated by the extensive use of GIS in the construction community.

More examples and national reviews will be included in the W106 Final Report .

4 TG3: GIS-SPATIAL DYNAMIC MODELLING OF THE INTERACTION BETWEEN THE NATURAL AND THE BUILT ENVIRONMENT

4.1 Motivation

Interaction of the environment with infrastructure is a complex process, involving a range of environment factors whose impact is very sensitive to spatial position and form. [Jernberg et al 2004, Sjöström et al 2005]. This is both the case with the infrastructure's reaction to severe events (flooding, cyclones, earthquakes, etc.) as well as to long term exposure to the "normal" environment. GIS offers a tool that can integrate the data and models on the critical factors within the natural environment and the resulting response of the built environment to these natural events. The IFC/IFD/IFG initiative and resulting standards for information exchange between the CAD and the GIS world will facilitate a paradigm shift, and permit common access to wide variety of geographically sensitive information. This information can range from climate and pollutant information to landscape or hydrological data. In many parts of the world, GIS-based air quality surveillance and planning systems that include modelling tools down to the micro-level, are being used to provide information on the exposure environment causing degradation and other damages [Jernberg et al, 2004].

4.1 Objectives

The objective of TG3 is to:

- promote the exploitation of the geographic information technology as a tool to model the degradation environment to buildings and infrastructure;
- explore the availability of environmental data and data sharing possibilities for an efficient, effective, and economic use of digital geographic information for modelling and mapping the degradation environment on various geographic levels, and
- contribute to a unified approach to characterize the exposure of the built environment.

4.2 Case Study Review

4.2.1 Australia

The Australian progress report provided a good description and extensive examples of the use of GIS within the TG 3 field of work as presented in the Toronto report [Haagenrud et al, 2004].

> *"While geographic information systems are widely used in Australia in assisting land-use planning and management [Trinidad and Marquez, 1998], they are also of increasing use in managing risk to infrastructure [Trinidad and Cole, 2000] and in transport design [Marquez et al, 2001]. In most cases they are used primarily for information storage and retrieval, however, in some cases their capacity to integrate data and to make decisions based on this integrated data is being used in commercial settings".*

In the research arena, a much greater emphasis is placed on active use of GIS not only to integrate diverse data sources but also to transform data though manipulation within the GIS system [Trinidad, 1999]. In particular, significant advances to model the interaction between natural and man made environments have occurred over the last few years. This work has been based, in part, on the particular advantages of geographic information systems and on the ability of computer models to incorporate an intensity and breadth of data not possible in the experimental or engineering models previously applied. Consider the example of the prediction of corrosion in a marine environment as defined in a series of papers by Cole et al, [2003a-c, ,2004] A schematic diagram of the types of information being combined is shown in Figure 2.

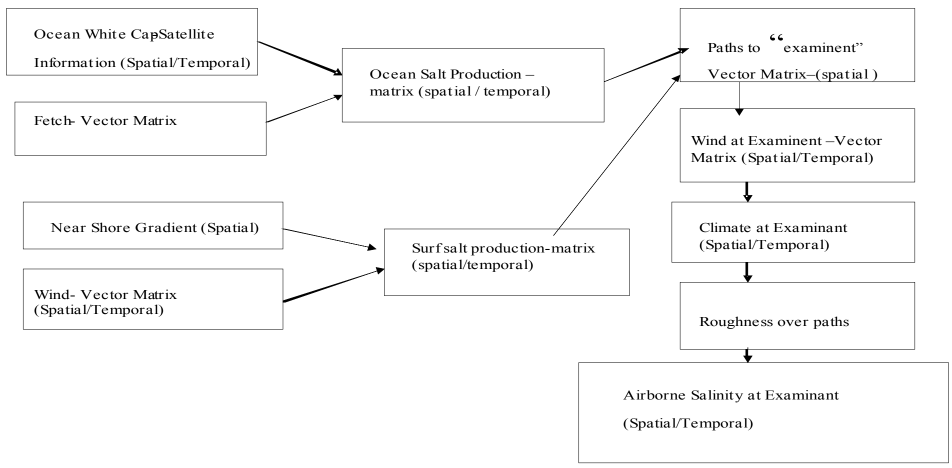

Figure 2 Schematic Diagram of Information Flow for Prediction of Airborne Salinity

This research highlights issues relating to data form, such as:

- the production of marine aerosols is derived from knowledge of the ocean white cap activity (spatial and temporal data) and the fetch (vector);
- surf salt production as derived from knowledge of fetch, near shore seabed gradient (spatial data) and wind speed and direction data (vector), and
- in determining the airborne salinity at inland points it is necessary to define paths (vectors) to the inland points, wind (vector) and climate (spatial and temporal data) at the inland point and the roughness of the terrain (spatial data) along the paths from the examinant to coast points.

The analysis indicates some of the capabilities required for modelling interactions between the natural and man made objects. Models need to be able to:

- manipulate and combine data of different forms, vectors, scalar and temporal;
- derive spatial data (particularly fetch and path vectors), and
- process temporal data with significantly varying time scales (from 3 hourly to seasonal data)

Such capability is readily managed by GIS systems but very difficult to construct in non-spatial systems. In particular, the manipulation of vector data and linking these data to temporal data required to link the wind, fetch and land form on salt production and transport would be impossible in conventional (non-GIS) frameworks.

4.2.2 Municipal infrastructure
Municipalities are *now* collecting a considerable amount of digital data, while also possessing a considerable amount of paperware. The alphabet soup of input technologies described by Vanier (2004b) such as CAD, SCADA, AVL, RWIS, CAS, GIS, and PDAs provide almost unmanageable amounts for heterogeneous data with little framework to integrate these data.

In a position paper for the W78 workshop in Toronto, Vanier [2004] posed the following problematic: how do infrastructure managers assist society to ensure that the existing and future infrastructure can be sustained? That is, how can this industry be sustainable? He answered and postulated that “ this can only be done with the aid of ICT, and data integration is the key.”

.As evidenced by the recent survey on the state of asset management in Canada (Vanier and Rahman, 2004), decision support tools are not readily available to infrastructure managers and decision makers. One of the reasons could be the lack of data standards and protocols to allow these tools to co-exist and to interoperate. In many instances the data are not dynamic and it’s difficult to change input data and hence the output results (i.e. “what if” scenario).

Following a detailed survey of state of the art, challenges and opportunities, Vanier concluded the following:

> *“Municipalities are facing unprecedented challenges due to the increasing number of aging infrastructure assets combined with declining maintenance budgets. Leveraging the use of information technology, in general, and of GIS and asset management systems, in particular, to improve the efficiency and effectiveness of asset management work processes is considered as a crucial strategy to address these challenges.”*

5 TG4: EDUCATION ON GEOGRAPHICAL INFORMATION SYSTEMS AND INFORMATION SOURCES

5. 1 Motivation
Graduate study courses often are unaware of the strategic importance of GI application in the construction industry, and especially today, when sustainability issues are rising for architectural and engineering design. In architecture and civil engineering undergraduate courses in most curricula around the world, the topics of GI and GIS are considered a matter for geographers, thus leaving the construction sector trailing behind in the adoption of this technology.

The target of the built environment assessment techniques is to supply both methodologies and effective management techniques to support decisions in all different phases of the design process seen as analysis, planning, design specification, management, exploitation and control chain of the built asset It may be applied to public and private assets, the infrastructures sector and also for management of the cultural heritage. Apart from the planning and design of the object itself, it comprises several other aspects like the environmental ones as well the socio-economic variables and the cultural variables that characterises a specific project site, see also subsection 2.3, Figure 1.

5. 2 Objectives
The objective of TG4 is to develop:

- education requirements for GIS use by architects and engineers;

- a glossary representing the common language among the GIS- and the construction community;
- courses presenting GIS as a technology within the architectural and engineering design process;
- a list of the info sources of free spatial data sets location accumulated across Europe.
- pilot education interoperable applications (e.g. for distance learning);
- an Internet discussion list to gather participants subscription and comments in real time, and
- a broad participation within European initiatives and programmes, especially the Marie Curie Programme.[3]

5. 3 Case Study Review

One example of courses aimed to develop both the cultural aspects of GI impact on the design process as well as the technical aspect of GIS use is the pilot course titled "Assessment techniques for Built Environment" (www.dpmpe.unifi.it/histocity/esposito/mae_courses.html) started in 2000 at the Faculty of Architecture of the University of Florence. The course's aim is to teach advanced techniques needed to integrate GIS technologies in the design process [Esposito, 2003].

7 CONCLUSIONS

The world of geographic information is moving extremely fast. It is driven by rapidly-emerging NSDIs in almost all countries and regions, as well as by the rapidly-developing infrastructure for telecommunication. This severely challenges the building sector to facilitate and implement this new technology and its applications within its sphere of influence.

The significant drive within the AEC/FM and IFC community to provide for relevant location-based data (GIS) via IFC format will be a major facilitator to access relevant property data sets, such as environmental exposure data, reference service life, service life models, factor distributions, life cycle analysis (LCA) and life cycle costing (LCC) data, which can also be linked directly to construction elements in the digital building model.

It is concluded that time is ripe for a broad implementation of GIS-based applications in the construction sector. As a start, it is recommended that the work programme of W106 for the coming working period includes a focus on support for an IFC based fully integrated design and planning process for the built environment, as well as a close link to the European based R&D frameworks for integrated life cycle management of the built environment.

8 ACKNOWLEDGEMENTS

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9 REFERENCES

CIB 256, 2000, *GIS and the Built Environment,* Haagenrud, S.E., Rystedt, B., Sjöström, C. (eds.), October 2000, Report Publication 256, Rotterdam.

Cole, I.S., Chan, W.Y., Trinidad, G.S. and Paterson D.A. 2004, An Holistic Model for Atmospheric Corrosion: Part 4 – A geographic information system for predicting airborne salinity Corrosion Engineering, Science and Technology, 200 4, vol. 39, no. 1.

Cole, I.S., Ganther, W.D., Paterson, D.A., King, G.A., Furman, S.A. and Lau, D. 2003a. A Holistic Model for Atmospheric Corrosion: Part 2 – Experimental Measurement of Deposition of Marine Salts in a Number of Long-range Studies, Corrosion Engineering, Science and Technology, 2003, vol. 38, no. 4.

Cole, I.S., Paterson, D.A. and Ganther, W.D. 2003b. A Holistic Model for Atmospheric Corrosion: Part 1 – Theoretical Framework for the production, Transportation and Deposition of Marine Salts. Corrosion Engineering, Science and Technology, 1 June 2003, vol. 38, no. 2, pp. 129-134(6).

[3] It must be recognized that the activities of TG4 are part of an EU sponsored project.

Cole, I.S., Paterson, D.A., Ganther, W.D., Hinton, B., McAdam, G., Chotimongkol, L., Bhamornsut, C, Hue, N.V., and Purwadaria, S. 2003c. A Holistic Model for Atmospheric Corrosion: Part 3 – Effect of Natural and Man-made Landforms on Deposition of Marine Salts in Australia and South East Asia, Corrosion Engineering, Science and Technology, 2003, vol. 38, no. 4.

CEC, 2004, Commission of the European Communities, Proposal for a Directive of the European Parliament and of the Council - Establishing an infrastructure for spatial information in the Community (INSPIRE), COM(2004) 516 final, 2004/0175 (COD) , (SEC(2004) 980), Brussels, July 2004.

Esposito, M.A. 2003 *Education on Geographical Information Systems for Sustainable Management of Built Environment., TAed-GIS_Lab, Universita degli Studi di Firenze, Italy-state of the art report.*

Haagenrud, S. E., Cole, I. Sjöström, Esposito, M. A. 2004, Geographical information systems for Sustainable Management of the Built Environment, CIB Trienniel Congress, Toronto.

Jernberg, P., Lacasse, M.A., Haagenrud, S.E. and Sjöström, Ch, 2004, *Guide to Bibliography to Service Life and Durability Research for Building Materials and Components*, CIB W080/RILEM 175–SLM Publication 295, CIB, Rotterdam.

Lair, J. 2003, *Organisation and Use of GIS for the Built Environment, CSTB, France -state of the report.*

Marquez, L., Smith, N., Trinidad, G. and Guo, J. 2001 *Activity pattern and pollution exposure*: *a case study of Melbourne*, in European Journal on Transport Infrastructure Research, Issue Number 4, pp 371-390.

Norwegian Mapping Authority 2003 *Norway Mapping*. Ed: Sverre Steen, Hønefoss, Norway.

Orshoven, J. V. 2003, *Summary of findings of activities 1 and 2 of the INSPIRE State-of-Play project*, K.U. Leuven.

Sjöström et al, 2005, Product Declarations with respect to Durability – A progress report, *10th Conference of Durability of Building Materials and Components,* Lyon, 2005.

Trinidad, G. and Cole, I.2000. The role of GIS in modelling the performance of building materials, Chapter in GIS and the Built Environment (ISBN 91-631-0272-2), University of Gavle, Sweden.

Trinidad, G. and Marquez, L. 1998. Interfacing GIS with models for urban planning and analysis, In Information Technology Management, Volume 2, pp. 33-38.

Vanier D.J 2004. *Opportunities for Integrated ICT in the Management of Sustainable Municipal Infrastructure*, Position paper, W78 Annual Meeting, Toronto (submitted to ITCon Special Edition on Sustainable Infrastructure).

Vanier, D.J. 2004b. G*eographic information systems and interoperability of software for municipal infrastructure applications,* Institute for Research in Construction, National Research Council Canada, Ottawa, ON, Canada. B-5123.4, <http://irc.nrc-cnrc.gc.ca/uir/miip/docs/B5123-4.pdf> (16 Feb. 2005).

Wix, J., Bjørkhaug; L., Rooth, Ø., Sjögren, J., 2005 Industry Foundation Classes – Facilitating a seamless zoning and building plan permission. *10th Conference of Durability of Building Materials and Components,* Lyon, 2005.

Risk Based Asset Management Strategies for the Built Infrastructure

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ABSTRACT

Strategic asset management in the housing area involves the provision of an appropriate level of service at a cost that is acceptable for the community concerned. There is a need for asset planning and prioritisation models to be developed to allow the long-term implications of differing asset management and operational strategies on the building performance to be assessed. Such models should also allow a range of 'what-if' scenarios to be analysed to determine the effects of providing differing responses or levels of service life-care and where relevant, need to incorporate whole-of-life cost data including externalities and customer impact. It is suggested that such models should be capable of utilising failures of individual building components rather than relying upon the current practise of only predicting failure of population cohorts. A number of models use this approach; some relying on expert opinion to provide failure information with others using condition data supplemented with expert opinion to model the component failure. This second approach is currently in use by housing authorities with the research now focusing on verifying failure models and minimising data requirements for the models. Because the development of planning models worldwide is in its infancy, these issues need to be considered in the current development of international standards.

This paper discusses a reactive/proactive maintenance approach to the asset management of building components. In doing so, it proposes a risk-based response strategy that provides a systematic, yet flexible approach to prioritising housing and component renewal. This strategy is set within a context of the component service life as defined in ISO 15686. Consideration of varied and comprehensive option strategies such as, mean time to service, repair, replacement etc, are seen as important as is the need to secure the best value for least cost. The paper concludes by suggesting that a critical component of risk based asset management strategies is statistical and physical/probabilistic failure models which are needed to allow us to reduce costs whilst striving to provide a specific level of service.

KEYWORDS

Strategic planning, Asset management, Replacement cycle, Risk prioritisation, Service life

1 INTRODUCTION

Worldwide annual expenditure for the maintenance and rehabilitation of constructed building components is rising, whilst ageing building components continue to fail at an ever-increasing rate. Because of increasing concerns over the levels of expenditure required to maintain housing stock, asset management practices are gaining increasing attention. The long-term cost implications of poor asset management practices are significant, especially for major building owners or housing authorities, where the replacement cycle will see increasing need for life-care maintenance. These issues will only be exacerbated as new regulations come into force increasing performance requirements such as energy and water efficiency and other factors such as the impact of changing population demographics.

Traditionally, asset management by building owners and local housing authorities have concentrated on maintaining buildings at a minimal cost (Vanier 1999), except where aesthetics come into play. Issues such as the effects of different management and operational strategies on the delivery of customer service levels, whilst receiving some attention, have generally been relegated to the background in comparison to the ever-present issue of minimising costs, except in the situation where higher rents can be obtained by a more expensive finish! For urban communities, however, the prime objective of asset management involves the provision of an appropriate level of housing service at a whole-life cost that is considered valid for the community concerned. Here the challenge for housing authorities is how best to manage with limited replacement funds, while maintaining a satisfactory level of service for their aging mixed construction housing stock.

It can be argued that the ability or willingness to pay for a specific level of service is the critical issue, as it largely determines the levels of expenditure that the building owner can normally justify. Consequently, depending on regional economics and community expectations, the levels of service provided by housing authorities may, and indeed should, vary significantly across a country. This is especially true where decisions are based on economic grounds with the community bearing all of the whole-life costs. In all of these situations, a valid estimation of the future housing maintenance expenditure is required at a national and housing authority level, both in order to provide different but appropriate levels of service and also to permit longer-term decisions to be made for the renewal/repair of housing stock under different management/operational strategies.

To do so, asset management planning models need to be developed to allow the forecasting of maintenance expenditure, based on a risk based methodology (probability multiplied by consequence). This will allow prediction of the future performance of the housing stock, under different management and operational scenarios and, by inference, the whole-life costs associated with managing its building components under these scenarios. Whilst these models need information on the costs associated with component failure (consequences), they also require both an understanding, and development, of failure functions (probability) that may use statistical and physical/probabilistic models (EMPA 2003, Burn et al 2004), as well as comprehensive details of the building components being managed. Once such planning models are in place and the expenditure to maintain the housing stock has been estimated, a methodology is needed to determine how this expenditure should best be prioritised on the basis of risk to justify and rank building components for maintenance or replacement.

Generally, such management strategies for planning and prioritisation would be applied to building components for which the consequence of failure is low, for example an internal painted stairwell wall surface. Where the consequence of failure is high, for example for structural components, a different management strategy is necessary. As for low consequence components, management of these higher consequence building components also requires the use of an integrated risk-based strategy. However, because the information available for predicting failure is rarely available, the probability of failure models often require the incorporation of condition monitoring to improve the risk calculations, thus allowing risk reduction procedures to be implemented. These risk reduction strategies (for that is what the decision outcome leads too) may include factors such as active protection systems or premature component replacement. But in all cases there is a need of an effective asset management system to be in place, in order to secure whole life value as opposed to costs.

2 ASSET MANAGEMENT STRATEGIES

A number of asset management systems are available to allow the analysis of the long-term cost implications of different maintenance scenarios for housing stock. Each of these asset management systems involves a different strategy which may or may not meet the exact needs or requirements of a given housing authority, and, if their use is contemplated, the relative benefits of each would need to be compared. Generally, when applied to housing stock, asset management strategies should address both maintenance planning and prioritisation (Shen 1999, Johnson and Wyatt 1999) to allow a valid estimation of the future expenditure required (planning) and where this expenditure should occur (prioritisation). This should also include the options of whether to provide different levels of service, or to permit long-term decisions to be made for different management options. At its best a strategic planning/prioritisation process could be described as a tool kit for data analysis, visualisation and reporting. It should not prescribe one particular strategy as being the best, instead, it should support the user within a systematic process in order to reach the most beneficial decision. Ideally the planning/prioritisation process should evolve as the user learns about the problem and potential improvements, or as a reaction to changes in the operational environment.

When identifying and assessing which approach to adopt in managing housing stock, a major factor to consider is whether a proactive or reactive approach is to be taken; for example, whether the components will be left to operate to failure, or whether a dwelling's building components will be replaced or proactively maintained before an unacceptable failure occurs, and in the later case whether condition monitoring or active protection techniques will be introduced to mitigate risk. A critical component of any housing asset management model is that it should be flexible for any management or operational strategy and be able to predict the necessary expenditure to maintain that strategy, including the long-term implications of that strategy on whole-life costs, system performance and customer service levels. Figure 1 shows one possible approach which could be applied to building components, where the building components are divided into proactive and reactive building components and different practices are applied to these component classes.

Under the strategy illustrated in Figure 1, "reactive" building components with low consequence of failure would generally be left to operate until failure, and then a decision made to replace/repair these building components (prioritised renewal/repair), i.e. painted surfaces. Such a decision would include consideration of the economics of continuing to operate the existing component past failure and its potential effect on other building components, including externalities such as, the social impacts of ongoing failures, i.e. disruption to the community associated with maintenance. Generally, the condition of these building components can be predicted using statistical methods because significant quantities of failure data are available. For example McFallan and Tucker (2002) use a statistical methodology to derive component failure models using condition data. The methodology is currently used by managers to plan maintenance programs and forecast future maintenance needs but it does not yet allow true risk assessment or prioritisation of maintenance activities. However, for newer materials the quantity of data is limited, so physical/probabilistic models need to be used to predict condition. Unfortunately, in most case these models do not exist. As the consequence of failure increases (for example larger structural components), building components can no longer be operated to failure and they become "pro-active" where failure prevention measures need to be undertaken providing they are economically justifiable. In these cases, the probability of failure may not be well known and physical/probabilistic models need to be used, in conjunction with active condition monitoring, to determine the risk exposure and therefore the justification for using active failure prevention to reduce or maintain the risk exposure.

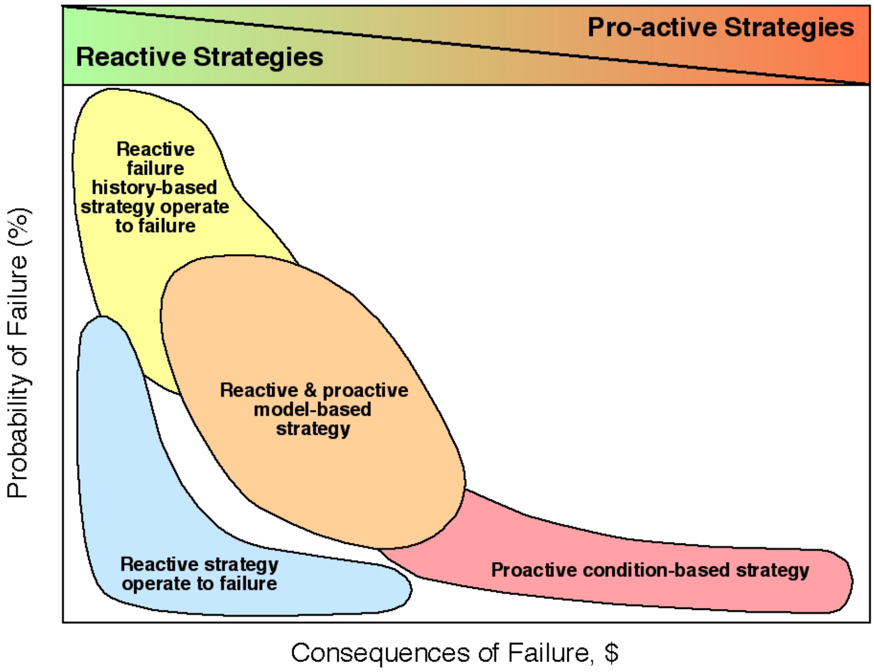

Figure 1. Asset management strategies for building components with different failure probabilities. (Burn 2004)

Note (i) It should be noted that whilst proactive strategies tend to be more justifiable at the high consequence end of the spectrum, they may also apply to the lower consequence building components if the economics of this are favourable, e.g. if low-cost condition assessment is available.
Note (ii) The converse is also true for reactive strategies, whereby even though the consequence of failure of an asset may be high, if the cost of condition assessment for failure prevention is prohibitively high, that component may be operated to failure.

When renewal/repair has been triggered by a factor such as component failure or a building component moving to a higher probability of failure, increasingly a risk-based strategy needs to be applied to this prioritisation process. By calculating the probability and consequence of failure, as well as the future frequency of failure, this can be used along with the cost of replacement to determine the priority/ranking for replacement. Building components ranked highly for replacement are then considered in a clustering exercise that considers adjacent building components, and develops a replacement work package that can be costed (for in-house replacement) or sent to tender (for outsourced replacement).

All asset management models regardless of their sophistication require significant asset lifetime and cost data, and in the case proposed in Figure 1, to allow a valid risk-based approach to be implemented, these data requirements need to be extended to include customer impact, externalities (such as traffic disruption associated with building works) and environmental costs. Using this data, a valid asset management strategy would thus provide information for decision making for three distinct purposes, viz.:

- Budgeting (setting limits on available funds and setting overall strategies).
- Scheduling (prioritising mitigative actions and their timing).
- Risk assessment, minimisation and mitigation (assessing costs and conditions to limit risk).

With the last point "risk assessment", allowing the consequences of the actions or options proposed to be assessed.

The usefulness of any asset management system depends on the accuracy of the underlying asset failure models and the quantity and quality of the data available, both being used to validate the asset based models and to describe the building components which the models use to predict component performance and consequences. But what failure, service life, mean time to failure or residual failure data exists that is easy to access, consistent in its values and useable? Unfortunately the availability of such data at the moment is very limited with the most comprehensive source being identified in the HAPM Manual (Construction Audit 1999). However; this situation should improve with the recognition that the insurance industry can also find benefit in assessing the remaining service life of a component or system (Mayer and Wornell 1999).

3 RISK ASSET MANAGEMENT STRATEGIES

Different risk management strategies may be considered depending on the consequence/probability of failure and, in portfolio and housing stock management terms, a mixture of such systems will be required in a consequence driven risk based philosophy which will include:

- Reactive Management of Building Components
- Replacement Prioritisation
- Proactive Management of Building Components

3.1 Reactive Management of Building Components

In a portfolio management sense, housing faces both political and economic constraints, as well as sudden or unexpected changes in the behaviour and ethics of the actors involved. This is reflected in the behaviour of many organizations where the priority of general maintenance is reactive and responds to either breakdown or to those who complain the loudest (Johnson and Wyatt 1999). Since the introduction of the Decent Homes Standard (ODPM 2000) in the UK and the revised Commonwealth State Housing Agreement (CSHA 2003) in Australia, authorities in those two jurisdictions have been required to demonstrate effective asset management through a range of KPIs. In Australia, the National Housing Data Agreement (NHDA 1999) provides authorities with guidelines for a minimum housing data set and performance indicators, while in the UK, Best Value Performance Indicators (BVPI 1999) provide local authorities with performance guidelines. This approach has lead to many authorities carrying out condition assessments and developing better asset maintenance models, with the result that authorities essentially know their portfolio better. The ratio of planned to responsive maintenance has moved from what was historically a ratio of 30:70 (with some as low as 15:85) to now aiming for and achieving a ratio of 70:30, particularly in the UK where there is a greater level of reporting. Moreover many managers are actively seeking to minimise known degradation/failure influencing factors, although the consequences of the interventions are not always understood. Many authorities have also improved communication with their tenants resulting in a better understanding of the service expectations.

The reporting requirements have resulted in authorities collecting significant amounts of data and while this data is largely collected for the purpose of reporting, some have recognised the potential and are actively using the data for managing the maintenance and upgrade programs. However, obtaining full value from the data would require extra effort, with the reason for failure often not recorded and the consequence of failure not considered. Without the relevant information, effective planning and prioritisation can not be accomplished. Determining whether the replacement criteria result is the most cost-effective practice is usually never undertaken, with the major impediment for undertaking such analysis being the lack of tools to interrogate the asset database and perform the analysis work simply and quickly.

Whilst this reactive method of asset management is valid, and may provide the lowest whole of life costs (if externalities such as customer satisfaction are excluded), it should be able to provide long term estimations of future maintenance costs. Thus a critical aspect of any planning model is scenario generation, as the performance requirements for the housing stock can differ significantly; for example for different levels of customer expectations. Once the performance requirements of the housing stock have been established, these scenarios should be able to be simulated to see what effect they might have on the resulting costs.

In a true risk analysis approach, the probability and consequence of each failure, would be determined and used to calculate a total risk cost associated with each scenario. In a reactive asset management methodology, this approach will not give a true estimate of the maintenance costs (because failures may occur multiple times within one year and with a high probability) and what is needed is a methodology to predict when failure will occur and the costs associated with this failure. A critical component of a reactive approach is thus to be able to forecast the expected annual number of failures for each type of building component for a set number of years, based on the operating and installation environment of each component. This can be accomplished by utilising a series of customised failure curves for each building component (based on statistical or physical/probabilistic models) within a housing authority's network, determined through analysis of the asset data, including the impact of component quality and the effects of the environment, as well as use and level (quality) of maintenance on an individual component. For each building component, the expected failure rates can thus be estimated for each year in the forecast period, based on the age and exposure conditions of the component in that year. The total number of failures in the system in any one year is the aggregate of the failures in each component, thus enabling the full building performance to be calculated.

3.2 Replacement Prioritisation

Once the long-term implications of different management and operational strategies on customer expectations have been assessed or are known, a budget using the above approach can be allocated to meet the year-to-year maintenance and planned rehabilitation needs of the respective local housing authority. Prioritisation models may then be used by housing authorities to focus on which building components their maintenance/rehabilitation budget should be spent. Any replacement prioritisation model should build on the planning models and provide a methodology for managing asset prioritisation in terms of the predicted failure scenarios of the housing stock, and the housing stocks likely responses to a range of (macro) renewal/repair strategies. Whilst this provides good guidance for spending renewal budgets and selecting general strategies, there is also a need for more detailed analysis on a micro level; especially to efficiently target renewal works to those building components where the net benefits are seen to be the greatest.

Housing authorities manage their housing stock in significantly different ways and any prioritisation model has to be flexible enough to account for these different practices. For example, operational environments vary in:

- The formulation of Key Performance Indicators – in particular indicators concerning customer expectations,
- Their operational environments – the varying climates of housing stock, varying operation requirements by the tenants, varying quality of materials and installation techniques and different methods of treating externalities, and
- Accounting practices and cost allocation approaches as these may vary significantly between repair and new work or improvements.

This heterogeneity in the operational environment has led to the occurrence of a wide range of building component renewal/prioritisation strategies by different housing authorities so that different problems require different solutions and standardisation of the processes involved may be difficult. Efficient communication and cooperation between housing authorities may be hindered by different priorities, language and tools. In the authors' opinion, the main feature of any prioritisation tool should be to allow the user to rank building component replacement based on a risk ranking approach using the predicted failure performance and the social (e.g. health risks), environmental (for example. flooding and loss of housing) and financial consequences (i.e. cost of repair and replacement) of building component failure.

As a measure of renewal urgency, the net present value (NPV) of future costs should be able to be calculated for a number of different housing component failure scenarios. Each option or scenario has associated cost/consequence values and an associated probability/likelihood. The incremental value, or incremental risk, to the NPV from each failure scenario is the product of consequence and the failure frequency corresponding to the probability of failure. Probabilities for each scenario are found via the aggregation (using probability generating functions) of the probabilities of single or multiple failures for individual building components. Failure probabilities and failure rates of individual building components are estimated based on statistical analysis of failure data using a non-homogeneous Poisson model, or where data is limited on physical/probabilistic models.

3.3 Proactive Management of Building Components

The strategies discussed above for asset planning and prioritisation are generally confined to "reactive" asset management, in which building components would generally be left to operate to failure. As shown in Figure 1, a different process is required for those building components for which "proactive" management for failure prevention is justified. These components generally represent a small proportion of the building components of any housing authority; however; they are in many cases the key structural components or protect other components from the elements, e.g. roofing systems. They are also components that, upon failure, result in severe consequences, so that the economics generally warrant their replacement before failure or the use of active protection strategies to control degradation. Although the probability of failure of these building components is low, failures result in catastrophic or severe consequences, incurring very high costs not only to housing authorities/municipalities, but also to large segments of society. For this reason, proactive strategies to ensure that failure does not occur tend to be justifiable.

Once the performance requirements of the housing stock are understood and can be related to some benchmark or datum, asset component scenarios can be considered to see what effect they might have on the resulting risk costs, such as

- Condition Based Assessment
- Thru Life Costs To date
- Serviceability Status Risks
- Access Matters
- Downtime
- Total Resource Commitments
- Budget Constraints
- Costs Estimates
- Risks remaining
- Service Life Gain/Loss
- Portfolio Costs v risk v value review

The scenarios for which each is evaluated are shown in Table 1
.

Service Life Assessment					Portfolio Strategic Framework		
Do Nothing	Clean	Service	Repair	Replace	Convert	Upgrade -Modernise	Dispose or Abandon

Table 1. Housing Asset management actions

To quantify the risk associated with any building component, the consequences of failure, the probability of failure and the corresponding projected frequency of failure need to be determined. But the major impediment for undertaking such analysis is the lack of tools to interrogate the asset database and perform the analysis work simply and quickly. That is until recently with recent developments in ISO TC59 and CIB W80 on service life development (ISO 15686, 2003, 2000a), though the adoption of this work must be handled with care and be conditional. This work however does embrace a systematic lifetime process through service life planning and encourages the adoption of both a whole life and service life approach for new and existing work (ISO 15686, 2000b). In turn enabling one to divide any dwelling or a housing portfolio into core elements and systems or clusters as shown in Figure 2. Here the balance between the consequence and the probability versus risk demonstrates the need to consider the focus on the individual component.

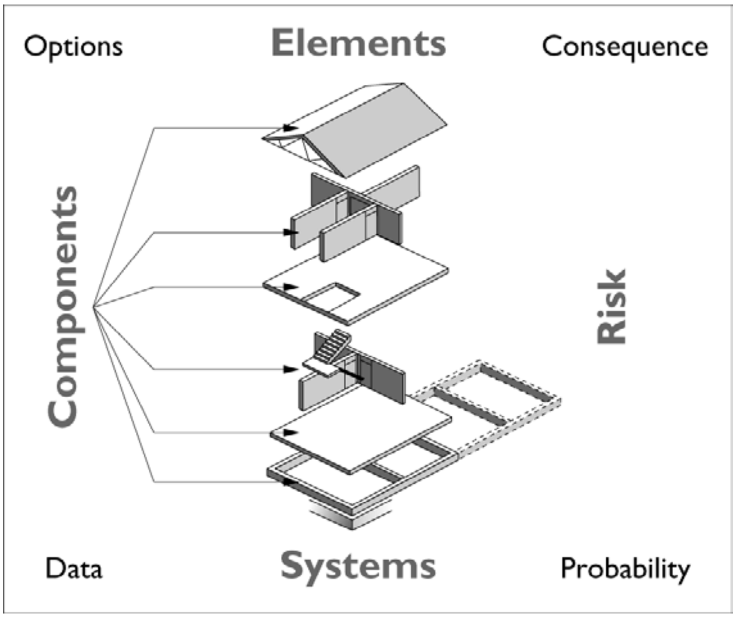

Figure 2. Strategic component model context

As each component that makes up the housing elements and systems has differing service lives, the clustering or grouping of components may require a range of options for their in use in terms of the reactive/proactive responses discussed in this paper and will have immense significance for the future of maintenance and whole life management of housing. Figure 2 shows that the component will be found in either a system or element and in the pictorial representation its maintenance or failure responses will depend upon the option selected.

For these types of proactive strategies the use of risk assessment becomes crucial and the process must be an integral part of strategic asset management.. The user should be able to risk rank the value of:

- Replacing the building component based on predicted failure performance,
- Repair or replacement of a building or the component to meet housing legislative requirements or
- Establish the risk costs associated with social consequences that are unacceptable, for example, the health and environmental risk from vermin or flooding.

3.3.1 Probability Assessment

The evaluation of probability of service life and end of service life failure is one of the most challenging aspects in a risk based approach for component risk-based decision taking (Moser 1999); not only because of the possible mixes and variation in degradation mechanisms in differing house and estate environments, but because of the level of proactive/reactive responses to maintain a high level of housing quality. Building components subject to proactive management tend to be major structural components or be a component of the building fabric. For proactive responses to components, the probability of failure needs to be determined from physical/probabilistic models or from statistical models based on very limited failure data. Furthermore it needs to be recognised that early failure may occur because the exposure or dose functions received may be higher in specific locations, for example, in a facade, parapet wall or room component. Here a westerly facing component may show more serious degradation and may require more maintenance, but such knowledge can also permit the asset management strategy for such components to be moved from reactive to proactive status by adopting the risk reduction approach over the service lifetime being managed. Whilst the development of such models may be possible if enough information is available, in most cases, validation of such models for specific building components needs to be conducted through condition based monitoring.

Generally, the utilisation of condition monitoring can only be justified where there will be an acceptable reduction in the risk level compared to the level of expenditure for condition monitoring. To minimise the high cost of condition monitoring, models are starting to evolve that will enable limited condition monitoring to be utilised in predicting the probability of failure of these types of building components (Davis et al 2004). The role of condition monitoring in asset management strategies has been identified (ISO 15686, 2003) but each technique carries a range of limitations and advantages for each building component or material being considered. Any limitations need to be considered when carrying out or interpreting condition monitoring results. Unfortunately, these limitations are not well publicised, nor well documented in the literature.

3.3.2 Consequence Assessment

The second critical component of risk analysis is determination of the consequences of failure, which requires assessment of the physical costs of failure such as materials, labour, transport, energy, and also assessment of the externalities associated with failure such as public disruption. Much of this information is currently difficult to assess and warrants a separate paper on the subject. Analysis of the consequences of failure also opens up the implications of time to failure and as well as introducing assessment and survivorship analysis of populations of components, systems and materials.

4. CONCLUSIONS

Asset management of housing components is becoming a critical issue worldwide, with expenditure levels forecast to ramp up rapidly due to aging infrastructure. It is also evident that with the longer life of constructed works and/or with poorer life-care and maintenance practices as the building ages, the more critical service losses, as well as increased problems in repair or restoration will become. A range of asset management strategies are available, but one methodology is to divide the building components into “reactively” and “proactively” managed building components. Reactive management will tend to apply to those building components with low consequences of failure with these components generally operated to failure. For these building components, the risk is allowed to be realised as failures. Failure history can be used to project future failure frequency and therefore future risk. Low consequence building components comprise the majority of components in housing and cost the most to maintain and replace. Because of this high cost, planning models are essential to allow the prediction of the future costs associated with different management and operational strategies.

Planning models should allow housing authorities to model long-term operational strategies and provide them with the ability to see how their actions will impact on the performance of their housing stock on their budgets and provide the information needed for long-term strategic planning. Following the allocation of budgets, housing authorities need prioritisation strategies to allocate their budgets for replacing/repairing building components with the greatest need. A risk-based strategy provides a systematic and flexible approach to prioritising housing component renewal. The risk approach should allow the user to account for the specific operational environments using a systematic process to reach the most beneficial decision and also allows for systematic processes to be specifically designed for a particular housing authority.

For building components justifying proactive management, a different approach is needed, in which renewal, maintenance, operational and condition monitoring strategies are required. To obtain a sufficiently cost-efficient strategy, this requires consideration of the risk (probability and consequence) of building component failure, the required customer expectations, proactive maintenance costs and condition monitoring costs. Unlike those building components which can justifiably be operated to failure, for proactively managed building components, risk must be determined in the absence of a failure history. Failure probability for the determination of risk must be established by either condition monitoring or degradation modelling.

5 REFERENCES

Burn, S., Tucker S., Ambrose M. and Moglia M. (2004). Management Strategies for Urban Water Infrastructure, *IWA leading Edge Conference on Strategic Asset management. 26-27 July, SanFrancisco USA*

BVPI (1999) http://www.bvpi.gov.uk/

CSHA (2003) http://www.facs.gov.au/internet/facsinternet.nsf/AboutFaCS/programs/house-csha.htm

Construction Audit Ltd (1999). HAPM Component Life Manual, EF SPON

Davis P, Moglia M, DeSilva D, Farlie M and Burn S. (2004) Estimating the failure probability of critical mains from non-destructive condition assessment. 6th National Conference of the Australasian Society for Trenchless Technology. Melbourne Exhibition and Convention Centre. 27-29/9/04

EMPA (2003) Engineering Design Methods for Service Life Prediction. Task group state of the art report on performance based methods of service life methodologies Cibw80/rilem 175 Swiss Federal laboratories for materials testing and Research.

ISO 15686 (2000a). Buildings and Constructed Assets - Service Life Planning- Part 3. Performance Audits and Reviews.

ISO 15686 (2000b). Building Facilities and Constructed Assets Part 1 Service Life Planning.

ISO 15686 (2003), Building Facilities and Constructed Assets Service life planning Part 7. Performance Evaluation and Feedback of Service Life Data from Existing Construction Works ISO TC59 SC14 Development Committee Draft.

Johnson M and Wyatt D (1999) Preparation and Prioritisation of Maintenance Programmes, Proceedings 8DBMC, NRC CRNC, Vancouver, Paper 152 Vol. 3, 1615-1624

McFallan, S.L. and Tucker, S.N. (2002). Probability of aa Dwelling Surviving or Failing in 9th International Conference on Durability of Building Materials and Components- 'Better Buildings for Tomorrow' Brisbane.

Mayer P and Wornell P. (1999). Assessing the remaining service life of existing building components for insurance. Proceedings 8DBMC. NRC CRNC Vancouver. Vol 2. PAPER 135

Moser K (1999). Towards the practical evaluation of service life illustrative application of the probabilistic approach. Proceedings 8DBMC. NRC CRNC Vancouver. Vol 2. Paper 123.

NHDA (1999) National Housing Data Agreement, ISBN 1-74024-082-2, AIHW Cat. No. HOU-48

ODPM (2000) Quality and Choice: a decent home for all: The Housing Green paper

Shen QP (1999) A Computer Aided Prioritisation Framework For Planned Maintenance Management, 8DBMC Proceedings, NRC CRNC, Vancouver, Paper 156, Vol. 3, 1655-1665

Vanier DJ (1999) Why Industry Needs Asset Management Tools APWA International Public Works Congress NRCC/CPWA Seminar Series Innovation in Urban Infrastructure pp 11-25

Sustainable and energy -saving building in China-the Snøhetta Modular concept

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ABSTRACT
The built environment is a huge energy consumer in China, with a potentially extreme increase by the rapid economic development going on. Energy saving in the building and construction sector can contribute for as much as one third of the targeted energy saving in China. It is thus an agreed and established policy of cooperation between Norway and China to facilitate and promote sustainable development and energy saving for the built environment.

The very low quality of housing is a major reason for the very high energy consumption. In order to increase the quality, the Ministry of Construction (MoC) uses two strategies. One is to introduce higher standards for new buildings and effectively enforce them; the other is the refurbishment of existing buildings, considering factors of existing building enclosure structures, building facilities and equipments system, energy supply systems, etc. The strategy to introduce higher standards and quality for new buildings is pursued via demonstration and pilot projects orchestrated by the MoC. A wide variety of building energy efficiency demonstration and pilot projects have been implemented in China over the years, providing important, practical experience in use of new technologies, materials and design concepts.

The most important such demonstration project is the "Future House" project in Beijing, managed by the MoC, Department of Science and Technology. The "Future House" demo/project site aims to make full use of architectural and ecosystem technology, including energy saving and environment protection technology, as well as intelligent architectural digitalization technology. It also aims to form systematic plans in the progress of layout, design, construction and decoration, which is quite necessary to improve scientific and technological level of buildings in China and guide the way of building construction as well as to put the modernization of national building industry forward.

Many countries are invited to participate with concepts demonstrating sustainable and energy-saving building. Norway is exhibiting the Snøhetta Modular concept, which is based on prefabricated modules of aluminum. Its benefits are durability, long lasting high quality, easy and industrialized production technology, facilitated by the IFC technology base, and well prepared for life cycle based planning, which is essential in China, due to the existing low quality and the aggressive exposure environment. A work plan is developed to provide "up – to date" and optimum solutions for IFC compliant sustaiability analysis, based on existing international standards and tools, such as the ISO 15686 "Service Life Planning" series.

KEYWORDS

Sustainable building, Product modelling, Industry Foundation Classes, Service life planning, Geographical Information Systems (GIS)

1 INTRODUCTION

The built environment is a huge energy consumer in China, with a potentially extreme increase by the rapid economic development going on. Energy saving in the building and construction sector can contribute for as much as one third of the targeted energy saving in China. It is thus an agreed and established policy of cooperation between Norway and China to facilitate and promote sustainable development and energy saving for the built environment.

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2 HOUSING AND ENERGY CONSUMPTION IN CHINA

The Chinese government has ratified the Kyoto Protocol, the framework convention for climatic changes, which shows that the Chinese government attaches great importance to and is willing to participate in solving global environmental problems. As early as at the beginning of 1980's, Chinese government has established the strategic role of energy efficiency as a long-term task.

There is a very high energy consumption pr building area due to the very low quality of housing, and overall it is considered that energy saving in the building and construction sector can contribute for as much as one third of the targeted energy saving in China [Ministry of Construction 2003].

The technical reasons that lead to high energy-consumption of per construction area and serious pollution of existing buildings can be summarized as follows:

- Bad performance of heat preservation and airproof of enclosing structure
- Irrational design of heating and air conditioning system, lack of adjustable and metering equipment
- "heat" as welfare supply
- Low standards and ineffective enforcement of higher building energy efficiency standards
- No incentives of renovation for building energy efficiency

The technologies of optimal retrofitting are not yet proven in China. The interrelation of the different elements of buildings structure, heating system and control are not yet well understood. There is very little technical experience with successful and energy efficient retrofitting. Some cities with better economy condition intend to renovate comprehensively some residential buildings, but their progress is only on plan period due to short of technology support and operation methods.

A wide variety of building energy efficiency demonstration and pilot projects have been implemented in China over the years, providing important, practical experience in use of new technologies, materials and design concepts. Demonstration efforts have generally focused either on building thermal envelope improvements or introduction of heat metering, but, unfortunately, rarely both.

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3 THE SNØHETTA MODULAR CONCEPT

The Snøhetta Modular business idea is to deliver standardised, modularised and affordable homes constructed in aluminum, targeted towards young and middle-aged couples who place importance in and base purchase decisions on architectural qualities, economic value and ecological performance, **Figure 1**.

Figure 1 The SM house and concept

To accomplish this it is imperative the products and business processes incorporate environmentally sustainable principles at all levels.

The core offering is modularised housing, but ancillary services and products are included as well, such as financial assistance, owner forums, consumer education, etc. These additional features will be developed in order to facilitate the purchase and to establish an SM community and identity that will develop a lifestyle brand.

The first focus is to develop product specifications for a house intended for regular home-buyers (individuals, associations or developers), where the guiding principles for the product development are direct follow-up of the business idea and focuses on:

- Ecology
- Sustainability
- Energy efficient
- Flexibility
- Structure
- Materials
- Infrastructure

- Ethics
- Esthetics
- Accessories
- Building technology
- Foundation systems

All manufacturing will be outsourced, and hence, an important process for SM will be to ensure that all processes are performed according to technical and a commercial specifications. This presupposes that SM must specify clear and rigid requirements for the manufacturing, and that SM must perform regular inspections.

An important resource is that the modules are designed in accordance with IFC- standardisation, giving the opportunity to create a 3D object oriented building information product model (BIM), containing detailed information about every building part and process. Software companies have developed good solutions for limited araes like calculation, CAD, GIS, building specification systems, logistics, tendering and facility management, etc [Wix et al, 2005]. Through IFC standardisation these areas will be able to communicate with each other to release the whole potential in the existing technology.

4 THE IFC-IFD-IFG PLATFORM FACILITATES COMPLIANCE TO BUILDING REGULATIONS AND REQUIREMENTS

The current building process is primitive in terms of information exchange. Various stages in the building process, although depending on the same information as other stages, will not share that information, resulting in inefficiency, large amount of errors and faults, building damages, shortened lifetime and higher costs.

IFC allows, among other things, two or more unrelated applications in the building process to exchange information about the underlying model the project wishes to realize. It thus supports the move away from drawings and onto real 3D BIMs, allowing for a continously enrichment of the model with relevant information throughout the building process [Bakkmoen and Sunesen 2005, Bjørkhaug et al, 2005, Kvarsvik et al, 2005]

EXPRESS is the data modelling language in wich both the IFC and IFD[1] standard are based. International Framework for Dictionaries (IFD), or the ISO/DIS 12006-3 [ISO 1992], extends IFC to include unique and specific definitions of concepts used in the building industry. While the IFC standard describes objects, how they are connected, and how the information should be exanged and stored, the IFD standard uniquely describe what the objects are, and what properties, units and values they can have [Bjørkhaug et al 2005].

One recent development project within the IAI network is the development of IFC based links to geographical information (IFG[2]) [Wix et al, 2005]. This will be essential for example for facilitating a seamless, internet based zoning and building plan permit. Key to this development is the integration of GIS information in a central building and property registry with AEC/FM information about the individual buildings that are registered. For a building that is to be developed, information is taken from the registry to provide information about location map data including property data, utility services, demographics, zoning, risk factors.

[1] IFD - International Framwork for Dictionaries, ISO/DIS 12006-3. http://www.icis.org/tc59sc13wg6
[2] Ifc for GIS. GIS workgroup in IAI. http://www.iai.no/ifg/

The IAI developers has already defined and included concepts and properties related to a whole range of facility management aspects, such as life cycle costing, environmental impacts and service life, relying and referring to international standards, such as ISO 15686 [ISO 2000].

5 IFC COMPLIANT SUSTAINABILITY ANALYSIS OF SM

5.1 Objectives

Sustainability and sustainable housing are important goals in the development of the Snøhetta Modular concept, and is one of the important issues due to the Future House exhibition in Beijing.

The overall goal of the parallell R&D programme project is therefore to provide "up-to date" and optimum solutions for IFC compliant sustainability analysis, with the following detailed objectives

- to describe methods and input for an evaluation of the sustainability of the Snøhetta Modular concept
- use some of the described methods and tools to document some aspects of the concept and product's sustainability
- give early stage input for improvement of product based on the sustainability evaluations
- give information on critical aspects concerning sustainability and the results
- evaluation of state of the art, as regards sustainability analysis delivered on IFC based platform

5.2 Work programme

5.2.1 Background

An evaluation of the sustainability of the SM houses built in Fredrikstad, Norway and Beijing, could be described in several work packages, each of which is also evaluated in terms of IFC feasibility, and best as possible IFC based analytical tools are delivered.

An evaluation of sustainability means regarding ecology, economy and social aspects in a common context. Emissions and resource efficiency, hence use of energy and material, are important issues in the ecology part of sustainable aspect. Affordable housing is an aspect in sustainable housing due to the economy part. Social aspects may both be regarded on a building and/or an area level. On a building level important issues are health and availability, where indoor climate is an important part of the health issue.

As sustainable is not a well-defined level, sustainability has to be described in terms of comparison to defined reference levels. These levels will vary from country to country, or from region to region. The reference level should therefore be defined based on this, and decided if there should be a global reference or and more local reference, for instance one reference for Norway and one for China.

5.2.2 Reference library

"BARBi" is an acronym play on the Norwegian word for a reference library for the Norwegian Building and Construction Industry. It is a fully developed, compliant and working platform based on the IFD standard. This allows producers, engineers, architects and users to completely define products and concepts within a standardised framework. The product will be available through an open and clearly defined Application Programming Interface (API), and be free to use for the Norwegian Building Industry. The goal is to synchronize the definitions with other national IFD based dictionaries.

The BARBi database already contains 9 different languages, 999 activities or processes, 40901 names, 6600 definitions, 41 actors or roles, 1939 properties, 8757 objects and subjects, about 680 groupings, about 1300 relations between concepts, etc [Bjørkhaug et al, 2005].

Developing the Chinese "BARBi" is part of an R&D cooperative program between Norway and China, thus facilitating the industrialisation and production of the SM in China. When these reference libraries have been developed and synchronised the applications “understands” both Norwegian and Chinese and communication runs smoothly.

5.2.3 Complying with requirements according to ISO 15686

The evaluations will be based on the ISO 15686 “Buildings and Constructed Assets – Service Life Planning” series of standard. This is developed by the standardisation group ISO/TC59/SC14” Design Life of Buildings” with the scope to document steps to be taken at various stages of the building cycle to ensure that the resulting building, or other constructed facility, will last for its intended life without incurring large unexpected expenditures. [Sjöström et al 2002].The ISO 15686 is applicable to both new constructions and the refurbishment of existing structures.

Service life planning of a design object involves the estimation of service life of its components (ESL), and compliance with the requirement of estimated service life being ≥ design life. If not, other types must be considered untill the criterion is satisfied, see **Figure 2**[ISO 2000]. Similarly goes for checking compliance with LCC and environmental assessments, ISO 15686-5 and -6.

Defining the reference service life and the estimated service life for the chosen products and locations is of high importance for the work with LCC and LCA. Therefore this WP should be completed before the other WPs starts. As for the IFC compliance the the minimum set of properties an IFC based BIM must contain to estimate RSL and ESL on the relevant objects are defined and put into the
BARBi library.

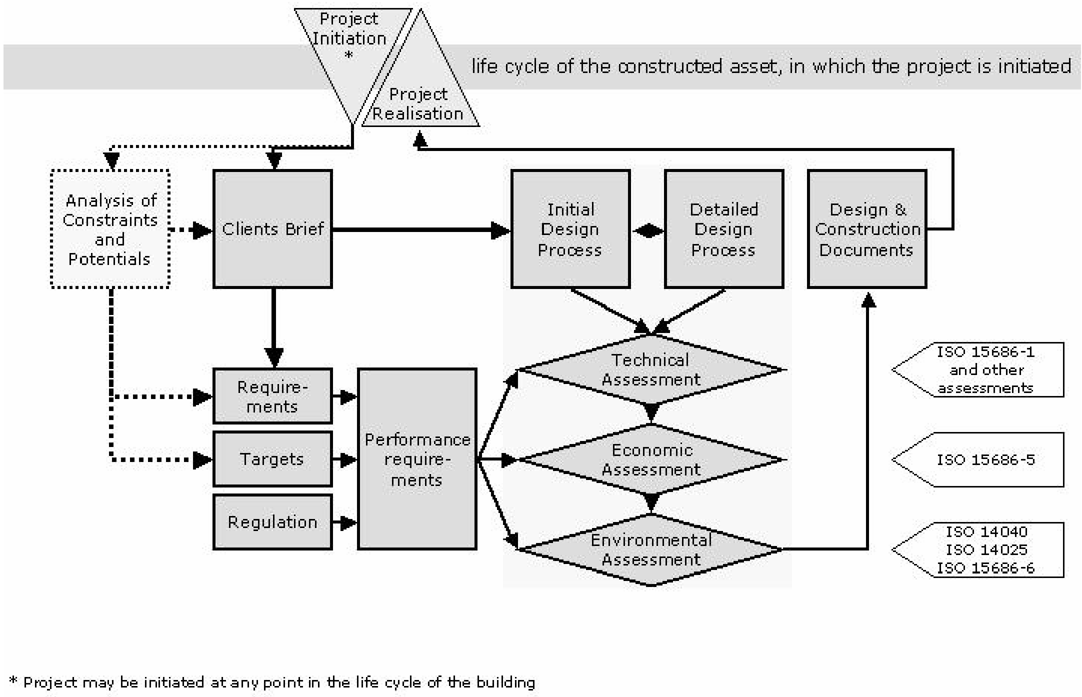

Figure 2Technical, economic and environmental assessment in the ISO 15686 Service Life Planning process

In order to achieve an appropriate estimated service life, there is a need of modifying a reference service life available by taking the differences between the object-specific in-use conditions and the reference in-use conditions into account. The Factor method [ISO 2000 and Jernberg et al, 2004] provides a systematic way of carrying out such a modification.

For the SM building elements a RSL will be provided with a fully described set of in- use conditions compliant with the value of 1.0 for each of the factors. On each building site deviation from any of these conditions will cause a resulting difference in estimated service life.

For example, the NBI knowledge base contains a range of dose-response functions or damage functions relating the degradation to the exposure environment; e.g. addressing the factor E, as well as relevant exposure data. Those have been obtained from various sources [Jernberg et al, 2004, Tidblad and Kucera 2002].

Aluminium is considered one of the most resistant structural metals under atmospheric exposure. Its degradation being, however, dependent on time of wetness (ToW), SO_2 and ozone (O_3) under free and open exposure, and, in addition, chloride (Cl^-) under sheltered exposure. The corrosion rate in unpolluted atmosphere is about 0,06µm/year and higher (0,08µm/year) under sheltered positions. The effect of SO_2 is quite pronounced, meaning that the corrosion rate will increase very much in the heavy polluted Beijing atmosphere. For the same reasons of pollution cleaning and maintenance, especially of the sheltered positions, will be an important issue here.

Such dose-response functions have been mapped for many places in Norway by use of existing environmental data [Haagenrud and Henriksen 1996], and with the use of the IFC/IFD/IFG technology these functions and the resulting service life for the product can now be directly linked to the building element in question [Bjørkhaug et al 2005].

With this data and functions durability and service life can be predicted at the building location, also when rolling out of SM at new places in China. The degradation functions will be modified, and the environmental exposure data will be provided by the Norwegian Institute for Air Research (NILU), who is responsible for several air quality assessments networks in China, and member of the co-operative R&D Norwegian-Chinese team see [NILU 2003].

5.2.4 LCA of the aluminum products

A crucial aspect in the modular project and in an environmental context is the use of aluminium because of the high embodied energy. Based on the life cycle assessments (LCA) already done, there will be an independent assessment of the aluminium-products and the modular system. Important factors for sustainability being addressed are quality, transport, distance manufacturer-market, energy form/source for electricity etc and their importance for the results. The project will be based on available information, and a discussion concerning the input quality is an important part.

This part of the project will use building parts (construction system and façade system) as cases for evaluation. The results are very dependent on construction details, and the evaluation should be done in close cooperation with the designers to ensure correct input. Critical factors have to be identified. The results have to be compared with traditional buildings. The results of these assessments will be used in a total life cycle assessment

5.2.5 Environmental assessment

Several methods are available for describing the buildings environmental performance and also to some extent the sustainable performance. Such methods gather most information regarding environmental and sustainable issues and is said to be an important tool to present and understand the environmental impact from buildings. Common for most of the methods is the need for reference level and some need for interpretation of the results. A recently established standardisation committee in Europe, CEN/BT WG 174 "Integrated environmental performance of buildings" deals with the inherent problematic of the sustainability issue.

5.2.6 Life Cycle Costing

Yearly costs and life cycle costing for the Snøhetta Modular concept with reference to standard costs can be used as documentation method due to the economy part in sustainable housing.

Life cycle costing is a well documented method for evaluation of economy in the project. Calculations have to be done on home/building level. Important inputs are investment costs, maintenance costs, running costs, and service life. The calculations have to be based on information from the manufacturer as well as other information sources. As there is little experience related to the product/production, there are several uncertainties related to input data.

5 REFERENCES

Bakkmoen, K. I., Sunesen, S. From lines to object models in building design, *10th Conference of Durability of Building Materials and Components, Lyon, 2005.*

Bell, H., Bjørkhaug, L., Rønning, J. 2004, ICT- platform for Object Oriented Knowledge in the Building and Construction Industry. B4E Conference "Building for a European Future", Maastricht, 14. Oct 2004.

Bøhler, T., Sivertsen, B. (2004) "An intergrated Air Quality Management System for sustainable development" *Dubai International Conference on Atmospheric Pollution 21-24 February* ,2004

CIB report Publication 256, 2000, *GIS and the Built Environment,* Haagenrud, S.E., Rystedt, B., Sjöström, C. (eds.), October 2000, Rotterdam.

European Organisation for Technical approvals, 1999, *Assessments of working life of products.* PT3 Durability (TB97/24/9.3.1).

ISO (International Organisation for Standardisation) (2004) Buildings and constructed assets – Service life planning – *Part 6: Procedure for considering environmental impacts*. ISO, Geneve. ISO 15686-6.

ISO (International Organisation for Standardisation, 2000, Buildings and constructed assets – Service life planning –-*Part 1* –General principles Geneve (ISO 15686-1, 2000).

Jernberg, P., Lacasse, M.A., Haagenrud, S.E. and Sjöström, Ch, 2004, *Guide to Bibliography to Service Life and Durability Research for Building Materials and Components*, CIB W080/RILEM 175–SLM Publication 295, CIB, Rotterdam.

Kvarsvik, O. K., Thi Lê, M. A., Rognlien, S., Mohus, F. 2005, Project management and the building process – A holistic view using IFC as a tool and carrier of information. *10th Conference of Durability of Building Materials and Components, Lyon, 2005.*

Ministry of Construction 2003 Policy statement on Energy efficiency and interesting areas, Beijing 2003

Sjöström, Ch., Jernberg, P., Caluwaerts, P., Kelly, S., Haagenrud, S.E., Chevalier, J.L. 2002, Implementation of the European Construction Products Directive via the ISO 15686 standards, *9th Conference of Durability of Building Materials and Components, Brisbane 2002.*

Tidblad, J. and Kucera, V., 2002, *Mapping of acid deposition effects on materials in Sweden*, Paper no 39, *15th International Corrosion Congress, Granada (Spain), 22 to 27 September 2002.*

Wix. J, Bjørkhaug, L., Rooth, Ø., Sjögren, J. 2005, Industry Foundation Classes – Facilitating a seamless zoning and building plan permission. *10th Conference of Durability of Building Materials and Components, Lyon, 2005.*

Aluminium is considered one of the most resistant structural metals under atmospheric exposure. Its degradation being, however, dependent on time of wetness (ToW), SO_2 and ozone (O_3) under free and open exposure, and, in addition, chloride (Cl^-) under sheltered exposure. The corrosion rate in unpolluted atmosphere is about 0,06μm/year and higher (0,08μm/year) under sheltered positions. The effect of SO_2 is quite pronounced, meaning that the corrosion rate will increase very much in the heavy polluted Beijing atmosphere. For the same reasons of pollution cleaning and maintenance, especially of the sheltered positions, will be an important issue here.

Project management, building processes and facilities management
- A holistic view using IFC as a tool and carrier of information

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ABSTRACT

There are a large number of different participants in a building project. These participants need lots of information from many different aspects. A systematic information structure will contribute to a common way of communication. With IFC as a common carrier of information, errors can be avoided or held to a minimum. All participants are working with their expert tools on the same product model, and with the same, updated information and datasets. This gives an assurance to the quality of the building processes and hence to the quality of the building itself. Information, knowledge and experiences can by this way be reused and improved, i.e. more dynamic data. A reduced number of construction defects and deficiencies will result in a more sustainable development and add value.

Statsbygg is going to make a gap analysis on specific capabilities required in building processes and for operation and maintenance that are currently supported by IFC. Our hypothesis is that the use of IFCs will be beneficial as a *tool* for stating user/client requirements ("requirement model"), followed by "design models" developed by the design team from the requirement model, "as built" models from the contractor developed from the detailed design model, and "FM models" that in essense are "as built models" enriched with operations and maintenance data from the building's service life.

KEYWORDS

Construction, project and information management.

1 INTRODUCTION

There is clearly a need for the participants in construction processes to communicate with a common language and in a more standardised manner, in order to reduce the friction and the shortcomings in information logistics. As much as 30% of the costs in a building project could be saved by improving logistics and the handling of information (Tollefsen 2004).

For the investor/ client to obtain better value for money, improvements must take place with respect to productivity and efficiency in the construction sector. This means that significant changes are necessary in the way the industry handles the flow of information in the projects.

1.1 Objective of the Study and limitation of this paper

Statsbygg is making a gap analysis to get a better understanding of the issues above. The main objective of this study is to consider how to share information in order to have the right and relevant information to the right person at the right time along the building processes. This paper focuses mostly on the processes in the various phases. The following issues are therefore not described in this paper: Health, Safety and Environment (HSE), risk management, uncertainty analysis, environmental impact, lifecycle management/ cost, space management

1.2 The philosophies of IFC

The IFC-format is an international standard through ISO/ PAS 16739, and is an acronym for **I**ndustry **F**oundation **C**lasses. IFC may store data in database systems, and is being developed of the International Alliance for Interoperability (IAI)[1]. The technology has for a long time been utilized in the production industry, and is now about to be established within the building sector in Norway. IFC defines a specification for data structure that can be used to organise a database, which can be accessed i.e. read/written and queried, by different software applications. This is a move towards interoperability in contrast to the building industry's present practice where "the same" information is put into different applications over and over again. The main philosophy of IFC is to have efficiency gains in the processes, from reuse of information/ data, integration of processes, interoperability etc., the participants are enabled to work much more seamless/ integrated compared to traditional approach.

2 THE GAP ANALYSIS

The course of building process will normally resemble this rough sketch:

Main project development phases	*Main project activities*	*Present IFC support - and application of these IFC mechanisms (preliminary analysis - hypothesis to be tested)*	*Gap (hypothesis)*
Investor's business development	Alternative investments (ROI) Sites of topical interest Investment project vision	*<For future study>* Actual, operational experiences from built projects entered as IFC object properties in running operation and maintenance can be invoked by query to IFC model server and assessed in investor context for new projects	Large?
Client / user / owner / tenant / government requirements	Project scope and goals Project limitations Government codes and regulations Spaces (size, qualities) Functional requirements Technical requirements Equipment and furniture	Present IFC theoretically seems to support most of the mechanisms to describe requirements. The possible need for some boolean operations, e.g. XOR, NOT, may need futher study – as well as mechanisms for stating "qualitative"/ "vague" requirements, e.g. regarding aestetics, environmental impact etc. Certain specific topics may need futher	Relatively small?

[1] www.iai-international.org

Main project development phases	*Main project activities*	*Present IFC support - and application of these IFC mechanisms (preliminary analysis - hypothesis to be tested)*	*Gap (hypothesis)*
	Project organisation Some early analyses Simple sketches and visualisations Contractual issues – client vs. design team	property sets as structural, security services, environmental issues etc. Further software implementation support and guidance for view definitions may be necessary.	
Building design	3D/ 4D object model of designed building (architect, structural, electrical, HVAC) established and continuously refined Building model (CAD) and map (GIS) interfacing Analyses (energy, cost, fire safety, acoustics, clash detection etc) Quality assurance Quantity surveys Government approval processes (permissions, waivers) Tender documents Contractual issues – client vs. design team and constructors	Present IFC seems to support most of the mechanisms needed for building design. IFG (IFC for GIS) should support data harvesting from government mapping, e.g. terrain model, zoning requirements etc. The number of IFC supporting vertical SW applications for various queries and analyses is increasing rapidly. Government codes and regulations checking is shown operational, Building and Construction Authority (BCA), Singapore; The National Office of Building Technology and Administration (BE) in Norway *ByggSøk* project progressing etc.	Small?
Building construction	Object model populated by actual vendor / system / component name / number / references according to deliveries in contract ith constructors Quantity survey referencing contractors quantities Change management at individual object level on-site Contractual/ Economic issues On-site logistics – 4D? Virtual walk-through – quality control of deliveries	The mechanisms for populating the designed objects with *real* objects according to contract are very similar and should be supported by IFC to an extent similar to design phase support. The main obstacle may be contractual – defining roles, risks, economic compensations etc to the "populating". i.e. correct data entry, job. IFG (IFC for GIS) should support data harvesting and the creation of relevant subsets of data, e.g. building footprint, facades, ductwork, from *built* construction IFC model, for export back to government maps Document extraction usage, e.g. operating and maintenance instruction manuals, may be semi-automatic at first, automatic later on.	Relatively small to medium?
Operation and Maintenance	Management of preventive maintenance Service contracts and day-to-day work orders Moving processes management Damage recordings Retrofits, refurbishings etc	Actual, operational experiences from the built project are entered as *current* IFC object properties in feedback loop from the built environment to new projects	Medium?

Table 1. A schematic overview of the gap analysis

Most of the IFC development has been focused on the design stage. However, there are developed some FM (Facilities Management) capabilities within IFC. These projects addressed specific FM issues like maintenance management, costs and financial elements of FM and portfolio and asset management and the evaluation of performance in use. A lot of specific capabilities that are required in different stages are currently supported by IFC, and Statsbygg is going to investigate these in a pilot project in 2005, e.g. approvals, asset data and descriptions, classification generally, constraints, cost information, dates time of environmental impact, equipment, FM standards, geographic elements, hazard identification, maintenance planning, move processes, participants involved, procedures processes, service life, interaction, systems, task identification, warranty and work orders.

3 USING IFC IN BUILDING PROCESSES

3.1 Early stage investor/ client demanding

Business development

The investors' main interest is to maximise the return on investment (ROI). In principle the investor does not care if this is done by IFC, pen and paper or whatever. The investors will promote whatever benefit their business/ their value chain. The transaction costs that do not add value to their investments are believed to be at least 30 % (Cain 2003). If the investors believe that IFC is the best way to attack this "waste" of money, they will most certainly promote the use of this tool. Hence it should be that IFC, including related spin-off standards IFG, IFD etc., effectively supports client requirements as outlined, through the entire life cycle of the building – from early stage ideas, through client program requirements, building design, construction, operation, refurbishment, retrofit and renovation – and eventually to demolition or re-use.

Project activities	*Important (IFC) issues*
Market analysis/ -needs	Location attributes (noise, pollution, traffic, communication)
Business idea	Potential development, gap analysis
Profit expectations	Return of investment, LCC
Location/ placing/ site	Site attributes (regulations in effect, topographical, ground attributes, installations, supply system/ capacity)
Alternatives	LCC
Uncertainty-/ sensibility-analysis	Risk calculation
Choice alternative/ investment objects	(Absolute) physical solutions

Figure 2 Some of the important issues in the stage of business development

3.1.1 Sustainable construction project

Areas, material, energy, LCC (life cycle cost) and effluent and waste are some issues that IFC will influence.

3.1.2 Hazards

Requirements for avoiding certain land areas (ex. contaminated land, landslides, high voltage proximity, high noise levels etc), certain building materials (e.g. PCB, asbestos, etc), certain safety aspects (fire, natural disasters etc) or security risks for persons or valuables (e.g. general crime rates, organised terror etc), technical service availability issues (energy supply, water and sewage, gas, telecom, etc), or losing lives of employees.

3.1.3 Early phase – ideas stage, building program

In early phase the functional descriptions take place, e.g. areas and volumes, proximity between functions, utilization etc. Then the technical requirements, e.g. at room, room group, or systems level are stated, and economic, environmental and LCC issues are evaluated. Various analyses e.g. statics, mechanics, acoustics, energy, LCC, costs estimates, risk analyses etc., can be accomplished by IFC-compatible software/ applications. Detailing can be early visualised in the process from the IFC model.

3.2 Submission processes

With the IFC format, an electronic validation of codes/ regulations is possible, as shown by the Building and Construction Authority in Singapore. The approvals, regulation plans etc. are handled more predictable and feedback of all details, governmental norms, requests, guidelines, directions etc. can be approved easily.

3.3 Design and construction, Design phase and Building phase

Following is the main project activities and the identified issues where IFC should be beneficial:

Project activities	*Important (IFC) issues*
Design and construction,	
User needs analysis	Quality assurance
Functions analysis	Situation plan
	Terrain- and ground issues
Room program	Room database
System solutions	”Proximity requirements” functions
Standard solutions	Room program
Qualities	Functional demands
Equipment	Quantitative demands
business like choices	Alternatives, simulations
Design phase	
Planning	Quality assurance (previous assumptions, rooms, space, equipment, amounts, qualities, solutions)
Sections	Foundation
Equipment	Energy needs, air quantities, etc.
Construction announcement	Building case processing
Approbation	Building case processing
Contracting / contracts	Contractual issues
Building phase	
Contract	Quantity control
Physical construction	Change management
Physical installation of all systems and products	”Technical change”
Final-settlements	Logistics, ”as-built” drawings, operation instructions

Figure 3 A schematic presentation of some important issues in building processes.

During the production phase IFC should also contribute to correct construction information on-site and ”in time”. Then the production management can record progress and logistics in “true time”. In addition it is possible to have visualization on-site, virtual surveys, continuous ”as built” updating and quality control and FM documentation.

4 THE WAY AHEAD - INFORMATION MANUAL AND CONTRACTUAL ASPECTS

Defined views should be supported to promote software implementation and to assist software implementers with implementation guidance. There are several requirements for documents in FM that could be supported from information contained within a populated IFC model. In particular, operating/ maintenance instruction manuals and health/ safety plans are relevant. Guidance on extracting information from the model and formatting it for presentation in documents would be a valuable project. It is now initiated a development project to make an information manual for IFC. The first milestone for the workgroup is in February 2005 where all involved parties will meet for co-ordination and information sharing/ exchange.

Contractual implications within the design team and versus the client and contractor when making IFC models are not obvious, and may have large impact. This may be the major pitfall with IFC models at present if not addressed properly.

5 SOME REFLECTIONS

5.1 Barriers for changes

There is without doubt a significant potential with respect to productivity improvements within the building and construction industry. Smooth and correct information flow is paramount to reducing overhead costs. Furthermore, co-ordination, co-operation, and a holistic view among all participants with the end product in focus, are absolute prerequisites for success. IFC could be the winning formula provided that the surrounding issues of people and processes are also effectively considered and organised.

It is in many ways a paradox that such obvious benefits are not exploited. One main barrier here is the absence of participants and/ or substitutes from outside. This of course curbs the construction industry's needs to innovation and continuous improvement in the building processes. According to Cain (2003) there are four ingredients of successful change. First the organisation must have a clearly defined goal for the change process. Secondly, the initiative must be committed from the Chief Executive. Thirdly the process of changing must be clearly described. Fourthly the employees must have a clear explanation of the commercial benefits of implementing the change process. Other important barriers as seen from a Norwegian perspective are few demanding customers, and the fact that the client/ project owner buys bits and pieces in a process, not an (end) product as such. Furthermore, the process has an excessive and complicated legal framework, where no client has an overall total responsibility or power for that matter, in this difficult and complicated process. This makes it difficult to determination who has responsibility for what, There is also a great number of participants in the process that only to a limited extent knows/ has knowledge about one another's work/ production systems, and at the same time the individual participants in the project team and their roles varies from project to project. All the actors are playing the market in a fierce price competition, which of course is often counterproductive as to co-operation. The industry also has a weak system-theoretic base with respect to measurement of performance and in addition there are extensive public regulations, which from our point of view limits innovation.

5.2 Holistic value chain/ -system

The information flow in a building process is complicated. There are a great number of participants sub-optimising from their own, contractually focused business interests. Holistic approach/ -actions and co-ordination with the end product/ building as main target comes in second.

To improve the industry and its end product, it is necessary to emphasise the focus on the entire value creation chain and its entire participant up until the completed building fulfilling its required functions. Value creating activities and the "expansion" of this to so-called "strategic maps", is an approach that can result in substantial economical improvements, value for money in the production of buildings. The overall value system can be considered and improved/ optimised by changes in activities, removing, moving, splitting and joining activities, moving activities to another/ others' value chain(s) and submission of activities after quality assurance.

In order to overcome the barriers listed, it is necessary to look into each individual company's information and production systems. In many ways it is an issue of an objective analysis of the respective companies; what they do in the process, with what quality and at what cost. This will naturally meet "business wise" grudges by many present participants as result could be changes to traditions roles and work division as well as responsibilities, and thereby business opportunities could be changed or moved.

5.3 Successful companies – regulating conditions/ mechanisms

Markets with strong competition makes "time-to-market" paramount to success. This leads automatically to increased expectations from the customers with respect to innovation, quality, delivery time, reliability/ security, service etc. The demands for increased productivity/ cost efficiency increases also because the competition reduces the companies' margins. This leads further on to the necessity to not only improve/ adapt processes, but to make changes to the processes. In the light of this it is not enough to look at ones own/ an independent company. Those that succeed consider the entire value creation chain and the process of "all" of the involved in the process.

5.4 Challenges for IFC

There are some real challenges for the development of IFC. First system wise; who owns the data, and who is the database manager. Secondly, this is a new way of working with new and different contractual interfaces. Then the technological aspect of it is that the applications presently are putting themselves in the centre. In order of the cultural aspect of this, the suppliers are locking the customers to them through proprietary solutions – IFC means a development "from handcuffs to silk thread". Besides these the ICT competence level and the willingness for sharing information and knowledge are vital. In addition, the economic aspect, as to who pays and who reaps the benefits as well as what is the right price for the data. In the end, the government bodies must stimulate innovation and increased life-span-focus. By this way, the market may create demand for IFC, push towards sustainability in the construction sector and facilitate massive use of ICT in AEC. Then, information and knowledge will give AEC industry a competitive advantage.

6 CONCLUDING REMARKS

In short, every stage of a building process will gain advantage by using IFC. First client/ owners will get updated information faster and earlier during the crucial decision making process. This will, simplify quality assurance, better decision simulations, simplify digital information storage, lower building costs and hence increase the investor's return on money.

In "early phase" - program phase, the participants will get a more accurate data fundament, faster and "safer" design, "new products", better knowledge management. The client/ user(s) can specify demands and external framework conditions that can be directly used in later stages. This includes also a contract document for what the design team should achieve, even before the architect has started modelling. This reduces risk as to cost, time, and quality and gives us a benchmark to use as a reference later.

The government regulators will have a more effective submission and approval process, better regulation and zoning administration/ development, better certainty as to higher service level. IFC can perform data harvesting from the "basis map" of the municipality with all regulation and zoning conditions etc. and the applicant will then get predictability as well as timesavings in the submission processing.

In building design, the use of the IFC-model in all subject fields gives a full consistence and traceability. Any IFC-object has a unique ID that cannot be manipulated and it follows the object through its entire lifecycle. This gives everyone the possibility to use applications as desired, e.g. "clash detection" – to discover where the ventilation shafts would potentially collide with the load-carrying wall, lighting fixtures and sprinkler nozzles. This gives an increased possibility for atomised quality control of the building and building services design, and the resources can be directly used for handling those discrepancies that the control of the IFC-model points out.

In the construction phase, IFC gives the possibility for running exact calculations of all objects, thus leading to improved cost control in all phases of the project. If the model is kept current and updated on an ongoing basis, it automatically also provides documentation "in real time" up till the end product. This will greatly improve the ability to handle all the alterations in the building phase in a much more predictable and cost effective way. The suppliers will have a wider assortment, better product development, improved predictability, increased proximity presence for customer and focus on end customers. Then the trade will gain an improved logistics optimising, "new products". This will result in a building process with better planning, better handling of alterations, better cost control, better quality in production, improved logistics etc.

In the operation, maintenance and development phase, with the use of the IFC-platform, information can/ will be transferred directly to a maintenance system, i.e. the property manager/ administrator will benefit from cost reduction, access to digital product information, better documentation regarding energy signatures, faster alterations/ rebuilding procedures, simpler digital information access and lower operational costs. A database of technical experiences can update the steering documents e.g. routines, procedures, description etc. as put forth in paper Model of experience transfer (Lê 2004). In order to succeed, the organisation needs an expert group that work continually for quality control of the experiences and route them to the right places in the database for experience transferring.

Finally, the society in general will have more effective use of resources, increased competitiveness as a nation, new services and increased export possibilities.

Phase	*Project-activities*	*Important (IFC) issues*	*IFC activities*
Business evelopment	market, idea, alternative placing, needs, function, analysis, choices, ROI, potential	site, regulations, settlement-prospects, alternatives, economy	Analysis of experiences from the operative and maintenance stage for similar type of construction, and also experiences from the previous similar project performance
Programming	space, qualities, solutions, illustration, economy, business choices, organising, contracting	site, regulation, function-demands, standard solutions, illustrations, economy	Map, calculation, comparison to norms, guidelines and experiences for previous cases, application approval
Designing	Sketches, plans sections, facades, equipment, construction announcement, tender documentation, contracting	quality assurance, approbations, enterprise budget, contractual issues	Comparison to norms, requirements, guidelines, regulations and experiences from previous similar projects. Descriptions of system and product in IFC
Building	quantity control, changes, ”certificate”,	amounts, progression and financial management, change-handling	Calculation and delivery of drawings and operative and maintenance documentations
Operative and maintenance	hand-over, trial period, operation, changes, development, service, potential	as-built drawings, operation instructions, labelling, changes, development	Work orders, historic registration of systems and products, experiences transfer to steering documents and database for analysis and QA.

Figure 4 A summary of project development – activities

In General, the effects by using IFC will be improved accessibility to information, more precise and “up-to-date” data. There will be easier and more precise reuse of information, more automation as to processes, faster and more dynamic development and exchange of data and quicker and more precise learning and knowledge development. In addition, the participants will have new and better decision processes, availability of new information earlier in the process, possibility to simulate decisions and see the consequences earlier. They also get ownership to the project data earlier and may ensure sustainable development by improving the use of resources and best practice solutions.

6.1 Future challenges

Future challenges will be new ways of working together, different roles for today’s participants, new and different contractual interfaces, placing of uncertainty/ risk, technology (the application is no longer vital), cultural – sharing competence, economy – determination of who is the owner of the data, what the right price for the data is, public body interaction – and politics.

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8 REFERENCES

TT9-239, *Project management, building process and facilities management*
- A holistic view using IFC as a tool and carrier of information, Lê, Rognlien, Wix, Mohus, Kvarsvik

Cain, T. C., (2003) *Building down Barriers, A Guide to Construction Best Practice,* Publisher Routledge, ISBN: 0415289653

Lê, Mai Anh Thi (2002), *Conditions That Promote or Inhibit the Transfer of Experience at a Public Construction and Property Company – a Case Study at Statsbygg*, NTNU, paper in conference EuroFM Mai 2003, Rotterdam

Lê, Mai Anh Thi (2004), *Methodology for Correcting the Quality of Decisions In Building Business*, paper in conference ICSFF August 2004

Mohus, F., (2004), *IFC - Byggherrens krav, IAI Informasjonsseminar om IFC, 15. Juni 2004*, Conference proceeding in Oslo

Rognlien, S., (2002), lectures' notes at Business School at Oslo and Norwegian University of Science and Technology

Tollefsen, T., (2004), *Digital fm; building information modelling used to improve the performance of buildings throughout the life-cycle*, paper in conference EuroFM, June 2004, Denmark

Wix, J. (2004), IFC Support for Early Design Stage & FM, Phase 1 – Vision/ gap analysis on early design + IFC status in FM for Statsbygg, Jeffrey Wix Consulting Ltd, Thatcham, UK