TC 140

Guide and Bibliography to

SERVICE LIFE AND DURABILITY RESEARCH FOR BUILDINGS AND COMPONENTS

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W080

Joint CIB W80 / RILEM TC 140 – TSL *Committee on*
Service Life of Building Materials and Components

Guide and Bibliography

to

Service Life and Durability Research for Building Materials and Components

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Preface

This publication 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 – a primer –providing fundamental information related to methods of service life prediction, information on environmental characterisation, and relevant information on the performance and durability of construction materials. Each part is self-contained - pagination is unique to the part and Tables or Figures in the text have been enumerated using a prefix that relates to the section in which they are presented. References are exclusive to the Part in which they are used – these have not been cross-referenced to the other parts nor the bibliography.

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, aluminum, 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 is yet to be completed. Given the significance of the task this work must be considered on going. The format for reporting has been set and it is hoped that future editions of this work will help fill in the missing pieces of information. 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 at the National Research Council Canada in 1993.

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. It is hoped that this initial contribution will spur others 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. The task to compile the information is considerable and continued support of these activities is essential if these worthwhile objectives are to be fulfilled.

P. Jernberg, M.A. Lacasse, S.E. Haagenrud, and C. Sjöström
March 2004

Introduction

P. Jernberg[1], C. Sjöström[2] and M.A. Lacasse[3]

1 Background

The most recent decades have seen rapid development in the area of durability and service life prediction of buildings materials and components. What earlier were ideas, ambitions or visions among a fairly few number of researchers, are today realities or within the reach for engineering applications. There are several reasons to this positive development: improvements in testing procedures, better analytical tools and methods, and a common computerisation that significantly has facilitated the ability to handle and process data, may be singled out as examples of factors that greatly have contributed. Another important influence and certainly a driving force to continued research in this area has been the demand for reliable service life data from building asset/property managers, management firms and management consultants as well as building owners.

In 1982 a joint CIB and RILEM activity was established on the Prediction of Service Life of Building Materials and Components, denoted W080 and 71-PSL (Prediction of Service Life) within CIB and RILEM, respectively. Mr. Larry Masters, National Bureau of Standards, USA chaired this Committee. The Committee focused on describing the state of the art of the research area, and proposed a generic methodology for the prediction of service life; work was concluded in 1986.

During the period 1987-1990 the RILEM co-operation with CIB W080 continued with the formation of TC 100-TSL. The work during this period centred on developing methodologies for generating data from long-term ageing studies of materials and components in actual, 'in-use', conditions. Within this area, special consideration was given to obtaining durability data from the inspection of existing buildings. The combined work resulted in the publication of a number of reports and papers.

The work of both these earlier technical committees was in general concerned with generic methodologies for service life prediction. It was a common opinion among those engaged in the earlier work, and also among many interested parties outside the Committee, that there was a great need for additional work on the development of a generic methodology. Hence, activities in the area of service life prediction continued in the same vein as the previous joint CIB/RILEM committees following the termination of RILEM TC 100-TSL in 1990.

This publication 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 third period between 1991 and 1996 and as well, additional information subsequentrly provided in the period between 1997 and 2002.

2 Objectives

Described in a simplified way, service life analysis aims at establishing the performance of a given material, component or element over time, i.e. how measured values of some chosen

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performance indicator varies with time. Loss in performance over time is clearly indicative of degradation taking place. Hence, service life prediction requires knowledge of the degradation environment and the resistance of materials to factors causing degradation. This simplistic view of the research area may be regarded as self evident, but it nevertheless describes the domain in which the development of both generic methodologies and material specific methodologies are needed. Further insights in regard to service life and long-term performance are provided in Part 1 section A.3.

3 Scope

3.1 Purpose

The work programme of W080/140-TSL was set up in 1990 and aimed at further detailing of the generic methodology established by W080/71-PSL. Prof. Christer Sjöström, KTH, Sweden, was elected as chairman of the joint committees. The programme covered five subject areas and was likewise organised into five sub-groups each having their respective sub-group chair to co-ordinate activities within the group. The five groups included:

1. *Data from field exposure testing, in-use testing and experimental buildings in service life prediction*

The chairman of this sub-group has been Mr. Erik Brandt, SBI, Denmark. The group aimed at completing efforts made within committee W080/100-TSL in which further descriptions of methodologies and development of approaches required to generate long-term ageing data were developed. The primary focus was on in-use testing, experimental buildings and field exposure testing.

2. *Environmental characterisation including equipment for monitoring*

Dr. Svein Haagenrud, NILU, NORWAY, and Dr. Jonathan Martin, NIST, USA chaired this subgroup. The work planned within this sub-group is to result in a contribution presenting the state of the art and research needs regarding characterisation of the degradation environment, including methods and equipment for microclimate monitoring.

3. *Materials characterisation including monitoring of degradation processed*

The chair in this sub-group has been Dr. Per Jernberg, KTH, Sweden. The sub-group task was to produce a report on modern experimental analytical methods for materials characterisation and the monitoring of degradation processes. The report is problem orientated, i.e. focusing on degradation processes, and is structured according to material classes (i.e., polymers, metals, brick, etc.).

4. *Mathematical modelling*

Mr Ton Siemes, TNO, The Netherlands, and Dr Jonathan Martin, NIST, USA chaired the subgroup. The group was to produce a report on the modelling of degradation processes and on prediction models.

5. *Design of short-term test methods*

Ms. Lesley Jacques from South Florida Test Service, USA has been the chair for this sub-group. The aim of the group was to prepare a state of the art report on the design of short-term test methods.

3.2 Significance — the liaison with standardisation

Twenty years ago, predicting the service life of building materials and components was only a distant vision. Today, the possibility of standardising methodologies and incorporating predictions of the service lives of materials and components into the design process for whole buildings is being given serious attention. The change in perspective is due to the sustained efforts of a small group of researches to advances in building material science and to developments in computerised knowledge systems.

The generic methodology established by W080/71-PSL for the prediction of service life became a RILEM technical recommendation in 1990. Hence, practitioners have increasingly accepted the methodology and service life concepts embodied within this recommendation. Indeed, this document is being promoted as the guide to use in dealing with service life problems. The need for standardisation in this area has been mentioned by several practitioners in the domain and consequently, this has been one of the key arguments for the work performed by W080/TC 140-TSL. In 1990 Eurocare, an umbrella project within the European research programme EUREKA, in co-operation with W080/TC 140-TSL approached CEN to help support standardisation of the methodology. The discussion that followed, involving ISO, CEN, W080/TC 140-TSL and a number of other interested parties, helped resolve the issues and resulted in the establishment, in 1993, of an ISO working group on the design life of buildings (ISO TC 59/SC 3/WG9 Design Life of Buildings). The ISO working group decided from the outset, to utilise the contributions provided by the W80/TC 140-TSL. In fact, the committee was asked to undertake the role of producing pre-normative work in the area ‘design life of buildings’ for the consideration of the ISO working group in the standardisation process.

The set of standards to be developed will be structured in a hierarchical way with, at the highest level, a generic standard presenting a methodology to be followed in predicting the service life of a material or component. An obvious candidate to this standard is the RILEM Technical Recommendation on Prediction of Service Life (the generic methodology). Other important documents to be mentioned are:

- British Standard BS 7543 “A Guide to Durability of Building Elements, Products and Components”;
- Architectural Institute of Japan, “A Principal Guide for Service Life Planning of Buildings”, (1993); and
- Canadian Standards Association CSA S-478, “Guideline on Durability in Buildings”.

On the second and lower level, a set of standards will be produced giving guidance on how the principles of the generic methodology should be incorporated into material specific service life prediction. As an example, the set of standards might be:

- Establishment of performance requirements and criteria;
- Characterisation of service environment;
- Characterisation of a material or component;
- Identification or degradation mechanisms of a material or component;
- Data from ageing under service conditions;
- Short term test methods;
- Predictive models and modelling of degradative processes; and
- Form of report on results of the service life prediction.

In view of this brief outline of the planned standardisation work it is evident that the current publication will establish an important input to this standardisation.

4 Future work

The work of the present committee will be continued in a new RILEM TC joint with CIB W80. The new work programme will be undertaken over a three-year period. The direction of work in the proposed committee is similar to the present one and will continue to refine existing prediction and service life techniques, tools and methods. However, the new committee will make efforts towards further development of service life prediction methods in the context of emerging information technologies (IT). Hence, the focus of a joint committee will then be to integrate existing prediction and service life techniques, using information technologies being developed for the construction industry. Furthermore, stochastic and reliability methods, typically used in the aerospace and automotive industries, will be reviewed as potential means of addressing particular aspects of service life prediction.

The proposed work is based on the elaboration of key areas of knowledge in the service life domain, which together provide practitioners with the necessary tools to guide their decisions. The five areas under the title, ”Service Life Methodology” includes:

I. Information technologies in construction: an integration tool in SLP

II. Service life and durability
- Service life and long-term performance; performance concept in buildings; integrated approach
- Establishing requirements for service life prediction, including factors causing degradation and performance over time functions: review of work undertaken by previous TCs with additional input from other committees contributing, based on proposed workshop.
- Data formatting

III. Selection of service life prediction methods
- Long-term studies, e.g. inspection of buildings (surveys) and controlled studies, e.g. in experimental buildings, field exposure studies and in-use testing
- Short term tests (accelerated test methods)
- Modelling of prediction methods: guide to the use of deterministic and stochastic methods

IV. Understanding factors causing degradation
- *Characterisation* of factors: e.g. weathering, biological, stress, compatibility
- *Modelling* of degradation factors: e.g. climatic modelling (macroclimate, meso-climate, and microclimate), weathering maps, climatic indices (e.g. solar radiation, time-of-wetness, driving rain, pollution).

V. Characterisation of degradation of building materials and components

Expected achievements are:

- Harmonisation of test methods;
- Internet access to various databases;
- Development of database of environmental test sites;
- Environmental modelling (micro and meso climate);
- Recommendations of accelerated and short term test methods using expert systems;
- Organisation of the 8th International Conference on the Durability of Building Materials and Components (8DBMC) and workshops on issues common to other committees within CIB and RILEM; and
- Support the development of ISO draft standard.

5 Terminology

Accelerated short term exposure	Short term ageing exposure in which the intensity of the agents is raised above the levels expected in service
Adapt	Intervention required to make an item suitable for a new use
Ageing	Degradation due to long term influence of agents related to use
Ageing exposure	Procedure in which building products are exposed to agents believed or known to cause degradation for the purpose of service life prediction (or comparison of relative performance)
Ageing test	Combination of ageing exposure and performance evaluation used to assess changes in critical properties for the purpose of service life prediction
Agent (degradation agent)	Whatever acts on a building or its parts (to reduce its performance), e.g. Person, water, load, heat
Agent intensity	The momentary magnitude of an agent, for instance the relative humidity or the SO_2 concentration at a certain moment (the term "agent intensity" refers either to intensity in the strict sense, or to concentration level, frequency or, maximum or minimum values. Momentary values of the agent in question)
Assembly (building assembly)	Set of components used together (draft amendment to ISO 6707/1)
Biological degradation factor	Degradation factor that is directly associated to living organisms, including micro-organisms fungi and bacteria
Biological growth	Growth of organisms on the surface or in the body of a material. These are generally fungi (moulds) or algae but other life forms are not excluded.
Bleeding	Diffusion of a soluble coloured substance into or through a coat from beneath that produces an undesirable staining or discoloration
Blister	Dome-shaped defect in a coat that arises from loss of adhesion
Bloom	Deposit on a coat of paint or varnish that causes loss of gloss and dulling of colour that can be removed by wiping with a damp cloth
Blooming	A visible exudation or efflorescence on the surface of a sample
Brief	Statement of the requirements for a building project
Building	Construction works that has the provision of shelter for its occupants or contents as one of its main purposes and is usually enclosed and designed to stand permanently in one place (draft amendment to ISO 6707/1)
Building context	A description of a building and its parts in terms of influences from design, service environment and usage
Chalking	Loose, removable powder resulting from breakdown of the surface of a material
Client	Person or organisation that requires a building to be provided, altered or extended and is responsible for initiating and approving the brief
Colour change	Change in colour other than that due to chalking, dirt collection or biological growth
Commissioned specialist	Person or organisation capable of conducting a service life prediction study
Commissioning client	Person or organisation that orders the service life prediction study

Terminology

Component (building component)	Product manufactured as a distinct unit to serve a specific function or functions or, specifically in this International Standard for editorial convenience, an item such as an assembly, a sub-component, a product or a material (based on ISO 6707/1)
Condition	All levels of critical properties of a building or a building part, determining its ability to perform
Conservation	Maintenance to preserve the appearance and performance of buildings and other structures, particularly historic buildings and structures
Constructor	Person or organisation that undertakes construction work
Control samples	Samples retained in an environment that is believed or known not to induce degradation for the purpose of comparison between exposed and non-exposed samples
Cost in use	Total cost including purchase, installation, running costs, removal, disposal and reinstatement
Cracking	Pronounced breaks that may extend well below the surface. The breaks may be "irregular" forming no definite pattern; "line" if running in parallel lines; or "sigmoid" if running in relatively large curves that meet or intersect.
Critical property	Property that must be maintained above a certain level if the building or the building part is to retain its ability to perform its intended functions
Damage	All the consequences of failure of a building component. This concerns the direct and the indirect effects.
Defect	Fault or deviation in aimed condition of a building or a building part
Degradation	Reduction over time in performance of a building or a building part
Degradation indicator	The complement of a performance characteristic, for instance when gloss is a performance characteristic, then gloss loss is the corresponding degradation indicator; when mass (or thickness) is a performance characteristic, then mass loss is the corresponding degradation indicator.
Degradation mechanism	Chemical, mechanical, physical or biological changes that lead to changes in a critical property(ies) of a building or a building part when exposed to degradation agent(s)
Delamination	Separation of layers of material
Design life	Service life intended for a building or a building part as stated by the designer, being in accordance with specifications set by the client
Designer	Person or organisation responsible for stating the form and specification of a building or a building part
Dimensioning critical property	The critical property of a building or a building part on which, for a certain set of performance requirements and a certain service environment, is imposed the performance requirement that first in time will fall short
Discoloration	Change in colour, including the effects of chalking, dirt collection and biological growth
Dose (agent dose)	The value of a definite time integral of the agent intensity function
Dose-response function	Function that relates the dose(s) of a degradation agent to a degradation indicator

Terminology

Durability	Capability of a building or a building part to perform its required function over a specified period of time under the influence of the agents anticipated in service
Effect	Result of an action of an agent, e.g. Wear, dampness, deformation, expansion
Environment	Aggregate of all external and internal conditions either natural, man made or self induced that can influence performance and use of a building and its parts (based on MIL-STD-721 C and draft ISO 6707/1)
Environmental condition	State of a characteristic of the environment
Estimated service life	Reference service life multiplied by factors related to specific circumstances, e.g. Materials, design, environment, use and maintenance (factor approach)
Exposure in experimental buildings	Long term ageing exposure in special buildings where the conditions may be monitored and in some cases controlled
Exudation	Emergence on the surface of a coat of one or more of its liquid constituents or of liquids from a substrate
Factor method	Modification of reference service life by factors related to the specific in use conditions
Failure	Termination of the ability of a building or a building part to perform a specified function
Failure probability	Probability that a given building component will fail during a given period
Feed back from practice (inspection of buildings)	Performance evaluation or assessment of residual service life of building parts used in actual buildings
Field exposure	Long term ageing exposure at special locations with known environmental conditions (agents)
Flaking	Lifting of coating from underlying surface in form of flakes or scales
Gloss	The shine or lustre of the surface of a material
Hair crack	Crack that is just visible to the naked eye
Incompatibility	Detrimental chemical and/or physical interactions between components which lead to degradation
Inspection	Activities that are necessary to register the present situation and performances
In-use condition	Environmental conditions under normal use
Life cycle	Successive periods of a building component, starting with the design, the construction, the use, the maintenance, the demolition and reuse
Limit state	Situation in which the product (materiel, component etc.) Is exactly equal to resist the influence of a degradation factor
Long term exposure	Ageing exposure with a duration of the same order as the service life
Long term in-situ exposure	Long term ageing exposure of building parts deliberately incorporated in actual buildings
Maintenance	Combination of all technical and associated administrative activities during the service period that are meant to retain an item in a state in which it can perform its required function

Terminology

Maintenance strategy	Maintenance policy
Manufacturer	Person or organisation that manufactures
Material (building material)	Substance that can be used to form products or construction works (draft for ISO 6707/1, pren 1745)
Obsolescence	Inability of a building or a building part to satisfy changing performance requirements
Peeling	The lifting of pieces from the surface without their becoming completely detached
Performance (performance in use)	Ability of a building or a building part to fulfil its functions under the intended use conditions (adapted from CIB W60 but ”a building or a building part” used rather than ”material, component, assembly or building”)
Performance characteristic	A quantity being a measure of a critical property, or an actual value of said quantity, i.e. A performance characteristic can be the same as the critical property, for instance gloss, or if the critical property is for example strength, thickness or mass may be utilised in certain cases as a performance characteristic
Performance criterion	A level of a performance characteristic, below which the corresponding critical property or properties of a component no longer are maintained
Performance evaluation	Evaluation of critical properties on basis of measurement or inspection
Performance over time	Description of how a critical property varies with time under the influence of degradation agents
Performance requirement	Range of acceptable performance within which a critical property is maintained (use set of requirements to denote the set of performance ranges within which the corresponding critical properties are maintained)
Predicted service life	Service life predicted from recorded performance over time as obtained, for instance, in ageing tests
Predicted service life distribution	The probability distribution function of the predicted service life
Preservation	Activities that are meant to maintain the present capacity of a building component (conservation, protection)
Preventive maintenance	Maintenance activities performed to avoid failure
Product (building product)	Item manufactured or processed for incorporation in construction works (draft ISO 6707/1, pren 1745)
Propagation time	Time in which the performance is degrading
Property measurement test	Test to determine a (performance) property
Reference sample	Samples of known performance which are exposed simultaneously and under identical conditions as the samples under study to provide comparative data
Reference service life	Service life for a building or a building part for use as a basis for estimating service life
Refurbishment	Modification and improvements to an existing plant, a building or a civil engineering construction to bring it up to an acceptable condition

Terminology

Rehabilitation	Extensive work to bring a plant, a building or a civil engineering construction back to an acceptable functional condition, often involving improvements
Reinstatement	Restoring and making good the surface structure of roads and land, replacement of fences, clearing of ditches and watercourses, and all similar operations following wok of repair or construction work
Repair	To replace or correct damaged or faulty components or subsystems of a building to maintain operating capability
Restoration	Actions to bring back an item to its original appearance or state
Retrofit	To add new materials or equipment not provided at the time of the original construction
Risk	The product of the probability of failure and the amount of damage
Service life	Period of time after installation during which all conditions of a building or a building part meet or exceed the performance requirements
Service life planning	Preparation of the brief and design for a building or a building part to achieve the desired design life in order to, for instance, reduce the costs of building ownership and facilitate maintenance and refurbishment
Service life prediction	A generic methodology which, for a certain or any reasonable performance requirement, facilitates a prediction on the service life distribution of a building or its parts for the use in a certain or in any reasonable environment
Short term exposure	Agei [illegible] on considerably shorter than the service life a [illegible] mes used and related to this type of expo [illegible] ictive service life test", which is a speci [illegible] m exposure and performance evaluation proce [illegible]
Short term in-use exposure	Short [illegible] which the intensity of agents are at level [illegible]
Specimen	Repr [illegible] quantity of material
Staining	Formation of discoloured areas
Sub-component (building sub-component)	Manufactured product forming part of a component
Supplier	Person or organisation that supplies buildings or building parts (the supplier can also be the manufacturer)

International Council for research and Innovation in Building and Constrcution (CIB)

Joint CIB W080 / RILEM TC 140 – Prediction of Service Life of Building Materials and Components

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to
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PART III – Building Materials and Components: Characterisation of Degradation

P. Jernberg, V. Kucera, K. Ödeen, A. Lewry, H-J. Schwarz and T. Yates

PART III — BUILDING MATERIALS AND COMPONENTS CHARACTERISATION OF DEGRADATION

P. Jernberg[1], *V. Kucera*[2], *K. Ödeen*[3], *A. Lewry*[4], *H-J. Schwarz*[5], *T. Yates*[6]

A. METALS (*V. Kucera and P. Jernberg*)

A.1 COPPER

A.1.1 Physical properties

Structural copper sheet is available in two main qualities: annealed, soft with a mat surface, and half-hard, rolled to a specular surface. Mechanical data are given in table A.1:1

Table A.1:1 Mechanical data for structural copper

	General	**Annealed**	**Half-hard**
Density (kg/m^3)	8940		
Thermal exp. coeff. (10^{-6} K^{-1})	17		
Modulus of elasticity (GPa)	11,8		
Yield strength (MPa)		≤140	≥180
Ultimate tensile strength (MPa)		220-260	240-300
Hardness (Hv)		40-65	65-95
Elongation (A_{50}%)		≥33	≥8

The annealed quality is mainly used for lapping works, where the softness facilitates the shaping. For other purposes the half-hard quality is preferred because of the higher stiffness.

Copper may be alloyed to brass, where zinc is the main alloying element. As brass is not used as a structural material in the normal case, this chapter focuses on "pure" copper.

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[6] Senior researcher, Building Research Establishment, Garston, United Kingdom

A.1.2 Chemical properties

Chemically, the most usual type of "pure" copper sheet is phosphorous oxidised, "oxygen free" copper, with a very low content of oxygen. A typical list of abundance of trace elements in such copper is shown in table A.1:2.

Table A.1:2 Example of trace elements abundance in phosphorous oxidised copper

	P	As	Ni	Ag	O	Se	Pb	Sn	S	Zn	Sb	Fe	Te	Tot.
ppm	240	85	62	50	38	34	32	25	17	16	13	9	1	**622**

The resistance of copper against various chemicals can be summarised as follows:

- *Acids.* Copper is rather resistant against oxygen-free solutions of non-oxidising acids like acetic, sulphuric and phosphoric acid. In oxidising acids, like nitric and chromic acids, or in acid solutions containing oxygen, the corrosion rate becomes appreciable. The rate is greater in concentrated and warm than in diluted and cold acids. Organic acids usually are less aggressive than mineral ones.
- *Alkali compounds.* At room temperature sodium and potassium hydroxides are fairly harmless to copper, while at higher temperatures only diluted solutions can be accepted. Ammonium and cyanic alkali compounds are quite aggressive to copper. An exception is dry ammonium hydrate, which does not react with copper. However, if water is added the reaction will be violent.
- *Salts.* Neutral salts, like nitrates and sulphates and sodium and potassium chlorides, are fairly harmless to copper. Alkali salts, like sodium carbonate, phosphate and silicate, reacts as the corresponding hydroxides, although weaker. Acid and hydrolysing metal salts, especially iron (III) chloride and sulphate, are quite corrosive. Chromates are not aggressive in neutral and alkalic solutions, while in acids solutions are very aggressive. Sulphides should be avoided, as the corrosivity is fairly strong.
- *Organic compounds.* Copper is resistant to esters, glycols, ethers, ketones, alcoholes, aldehydes and most of (other) organic solvents.

A.1.3 Usage aspects

Because of its low corrosion rate and the aesthetic so-called patina formation, copper has long been used for building structures such as roofs, façades and gutters. Many copper roofs on castles and other monumental buildings have retained for several centuries. Thus, large amounts of copper materials are used under atmospheric exposure in different types of climates all over the world

If conditions are favourable, copper may after some years in outdoor atmospheres develop a characteristic blue-green patina, consisting of basic copper salts and oxides. Sometimes a user is at first disappointed and complains when his newly laid copper roof shows a less attractive, mottled appearance [1]. After 6 to 12 months, however, the surface usually has acquired a uniform dark-brown colour. In general, the surface does not develop beyond this stage for a number of years. After 4-6 and 7-15 years a green patina usually begins to appear on sloping surfaces under marine and urban conditions, respectively. Vertical surfaces generally stay black much longer, as their time of wetness is shorter. In a marine atmosphere the surfaces facing the sea acquire a green patina sooner than the other surfaces. This is due to the greater supply of chlorides from the sea winds. In rural environment the formation of patina normally is initiated after 20-50 years. If the atmosphere only is slightly polluted, the patina may take an extremely long time to form (hundreds of years) due to the scarce supply of anions for the formation of basic copper salt. It may even fail to form at all if the temperature is low or the exposure conditions very dry. As shown by the potential-pH diagram in fig.A.1:1, green patina will also fail to form under acidic conditions, e.g. near chimneys, where acid smoke strikes the roof. Under such conditions, the corrosion products are soluble. Bitumen may also cause similar but more severe problems (see section A.1.5).

Prepatinated copper sheets are available at the market, to be used where the patina will not or take too a long time to develop. Also, quick patination coating systems for moderate areas are available, to be used at repair or extension of existing constructions.

Rainwater, running at copper-bearing surfaces, generally picks up traces of dissolved copper. The fraction of corrosion products that leave the surface and dissolve in the rainwater depends on the acidity of the micro environment. Accordingly, the dissolution is greatest in urban and industrial atmospheres, highly polluted with SO_x [3]. Such water in contact may cause blue staining on masonry, stonework, etc. It may also cause micro-galvanic corrosion in contact with metals less noble than copper. Consequently, the rainwater from copper surfaces should be properly collected and drawn off through gutters and spouts.

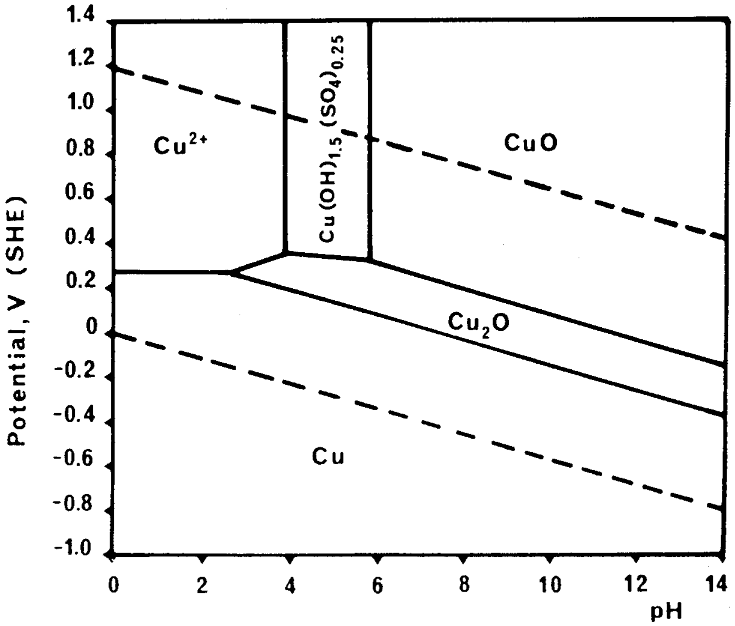

Fig.A.1:1 Potential-pH diagram, Cu - SO_4^{2-}- H_2O; 10^{-1} M Cu, 10^{-3} M SO_4^{2-} [2]

As copper in too high concentrations is poisonous, concern regarding copper leakage to the environment due to corrosion of copper roofs has been put forward. The problem should not be neglected, although investigations has shown that copper roofs contribute to a minor part, 1,5-10 % [4,5], of the total copper load in sewage sludge, while, for example, the traffic contributes by 20-25 % [5].

As copper is more noble than most other types of construction metals, measures to avoid ordinary galvanic corrosion have to be taken:

- Avoid electric contact between different metals; apply electric insulating material between them.
- Protect the metals from direct contact of humidity and water; apply a corrosive protective coating at the surfaces, even the copper
- Otherwise, fasteners, hangers and other details should be made also of copper or of stainless steel, compatible to copper

Copper in a soil and water environment, mainly found in the case of installations (which is beyond the scope of this book), is just mentioned briefly below.

A.1.4 Typical micro environments and loads

Agents at outdoor exposures affecting the atmospheric corrosion process of copper are mainly precipitation, time of wetness, temperature (directly by governing the instantaneous corrosion rate and indirectly by the effect on time of wetness), oxygen and air pollution, in the form of gases, molecules dissolved in rainwater and particles. As in all corrosion processes the concentration of oxidative compounds, normally oxygen, in the electrolytic film determines the corrosion rate.

Except from installations, copper in soil or water environments is not very common in the case of constructions and buildings. In the soil, usually the decisive parameters are the concentrations of humidity and oxygen, and the electrical resistivity of the soil. In water the pH value of the water and the concentration of dissolved ionic compounds like chlorides and sulphides are the main parameters governing the corrosion process. In the case of running water, e.g. in pipes, the risk of erosion corrosion has to be considered.

A.1.5 Typical degradation and failure modes

We may first consider the thermodynamic possibilities for the formation of copper compounds related to the various constituents in outdoor atmospheres [2]. The potential-pH diagram in fig.A.1:1 represents the system Cu - SO_4^{2-} - H_2O at 25°C with a Cu^{2+} concentration of 10^{-1} mole/litre and an SO_4^{2-} concentration of 10^{-3} mole/litre. As can be seen, copper metal is stable in a substantial part of the stability region of water. This is consistent with copper being a noble metal. The diagram also shows a stability domain for basic copper sulphate, $Cu(OH)_{1,5}(SO_4)_{0,25}$. The width of this domain depends on the SO_4^{2-} concentration; at decreasing concentration the domain becomes narrower, see fig.A.1:2a. The stability domains for $Cu(OH)_{1,5}Cl_{0,5}$, $CuOH(CO_3)_{0,5}$ and $Cu(OH)_{1,5}(NO_3)_{0,5}$ versus corresponding anion concentrations can also be seen in fig.A.1:2. The coating of corrosion products forming the patina has a rather complex composition, varying from place to place (table A.1:3) [6]. The main components are generally copper oxide and one or more basic copper salts [6,7]:

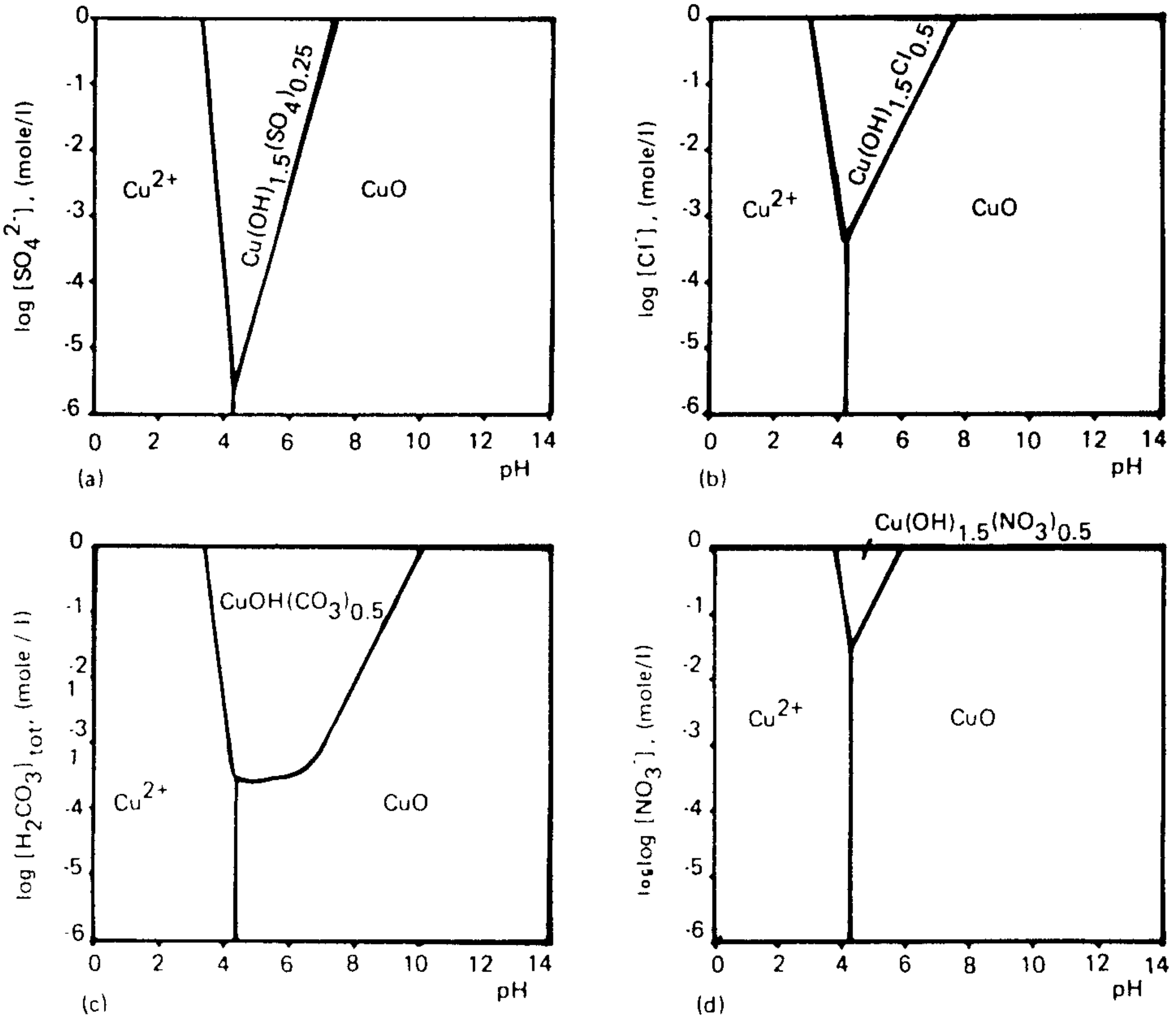

Fig.A.1:2 Stability domains of basic copper salts in aerated aqueous solutions with varying anion concentration and pH value; 25°C, 10^{-1} M Cu [2]. (a) sulphate; (b) chloride; (c) carbonate; (d) nitrate

Basic sulphate: $Cu(OH)_{1,5}(SO_4)_{0,25}$, $Cu(OH)_{1,33}(SO_4)_{0,33}$
Basic chloride: $Cu(OH)_{1,5}Cl_{0,5}$
Basic carbonate: $CuOH(CO_3)_{0,5}$, $Cu(OH)_{1,33}(CO_3)_{0,33}$, $Cu(OH)_{0,67}(CO_3)_{0,67}$
Basic nitrate: $Cu(OH)_{1,5}(NO_3)_{0,5}$

Table A.1:3 Basic copper salts in green patina from various atmospheres, defined by anions ranked with respect to concentration [6]

				Type of atmosphere			
Reporter	**Object**	**Time of exposure (years)**	**Country**	Rural	Urban or industrial	Marine	Mixed urban - marine
Vernon & Whitby	Copper roofs; copper conductor in marine atmosphere	12-300; 13	UK	1. SO_4^{2-} 2. CO_3^{2-}	1. SO_4^{2-} 2. CO_3^{2-} 3. Cl^-	1. Cl^- 2. CO_3^{2-} 3. SO_4^{2-}	1. SO_4^{2-} 2. CO_3^{2-}, Cl^-
Vernon	Copper roof on church spire on the isle of Guernsey	33	UK			1. Cl^- 2. CO_3^{2-} 3. SO_4^{2-}	
Freeman, Jr.	Copper roofs	16-78	USA				1. SO_4^{2-} 2. CO_3^{2-}, Cl^-
Thompson, Tracy & Freeman, Jr.	Copper panels from field test	20	USA	1. SO_4^{2-}, Cl^- 3. CO_3^{2-}	1. CO_3^{2-} 2. SO_4^{2-}	1. Cl^- 2. SO_4^{2-} 3. CO_3^{2-}	
Aoyama	Copper conductor for railway	-	Japan	1. NO_3^- 2. CO_3^{2-}	1. SO_4^{2-} 2. CO_3^{2-}	1. Cl^- 2. CO_3^{2-}	
Mattsson & Holm	Copper-base materials from field test	7	Sweden	1. SO_4^{2-} 2. NO_3^- 3. CO_3^{2-}	1. SO_4^{2-} 2. CO_3^{2-}	1. Cl^- 2. SO_4^{2-}	
Scholes & Jacob	Copper-base materials from field test	16	UK		1. SO_4^{2-}, 2. Cl^-, CO_3^{2-}	1. Cl^- 2. SO_4^{2-}	

In urban and rural atmospheres basic sulphate is predominant, while in marine atmospheres basic chloride usually is the main component. This is in good agreement with the thermodynamics described. Unexpectedly, however, basic carbonate is sometimes found in practice. Fig.A.1:2c indicates that the conditions in the atmosphere would not favour the formation of this type of patina, as the H_2CO_3 concentration in the water film would only be about 10^{-5} mole/litre at equilibrium with the air. The presence of basic copper nitrate, also found in some locations, indicates that the water film on the metal surface may contain an appreciable amount of nitrate (fig.A.1:2d).

Generally the corrosion products first formed are Cu_2O and CuO. The copper oxides react rather slowly with airborne molecules and ions, such as SO_x, Cl^-, CO_2 and NO_x, with the formation of basic salts, e.g.

$$4Cu_2O + 2SO_2 + 6H_2O + 3O_2 \rightarrow 8Cu(OH)_{1,5}(SO_4)_{0,25}$$

$$4Cu_2O + 4Cl^- + 6H_2O + O_2 \rightarrow 8Cu(OH)_{1,5}Cl_{0,5}$$

provided the pH value of the surface moisture is sufficiently high. The corrosion products give a non-insignificant corrosion protection to the surface.

Estimated ranges of atmospheric corrosion rates at intermediate exposure periods (10-50 years) for copper are given in table A.1:4

Table A.1:4 Corrosion rate ranges for copper in various types of atmospheres [6,8]

Atmosphere	Corrosion rate (µm/year)
Rural	< 1
Urban and industrial	1 - 3
Marine	1 - 2

For various types of copper alloys, the atmospheric corrosion rates at three different long-term tests are shown in fig.A.1:3. The weight loss is approximately the same for low-alloyed coppers, brasses, nickel silvers and tin bronzes.

The corrosion rate of copper decreases slowly with time, appearing to follow an exponential decay function as shown in fig.A.1:4 [3]. This behaviour indicates the formation of a partially protective layer of corrosion products. The protective action is probably due mainly to the presence of basic copper salts. In a study in UK [9], six contractors were asked to estimate the average service lives of copper roofs. The answers ranged from 60 to 100 years, and from 60 to 120 years, for harsh and mild environment, respectively. Mean values in the two cases were 83 and 92 years. However, with a sheet thickness of 0,6-0,7 mm and say, a corrosion rate of 1 µm/year that further decreases by time, service lives of hundreds of years could be expected. Functioning copper roofs at historical buildings of more than 300 years may also be found at many sites in Europe.

Typical corrosion rates in soil, fresh water and sea-water are 1, 10 and 50 µm/year, respectively. An exception is soils with extremely low pH values and high salt concentrations, where the corrosion rate can reach as much as 50 µm/year.

True corrosion defects of copper roofs are not very usual. However, bitumen may cause drastic corrosion [10]. Unprotected bitumen oxidises in contact to oxygen, at which acid reaction products are formed, being very aggressive to most types of metals. If bitumen is applied at a construction part above a copper roof, rainwater acidified by the bitumen may run down and put the copper material at risk. Such water — pH values about 1,5 have been detected — will hinder the normal formation of protective corrosion product layers, i.e. patina. In practice, 50 % diminishing of the thickness of a copper sheet after only 15 years has been found.

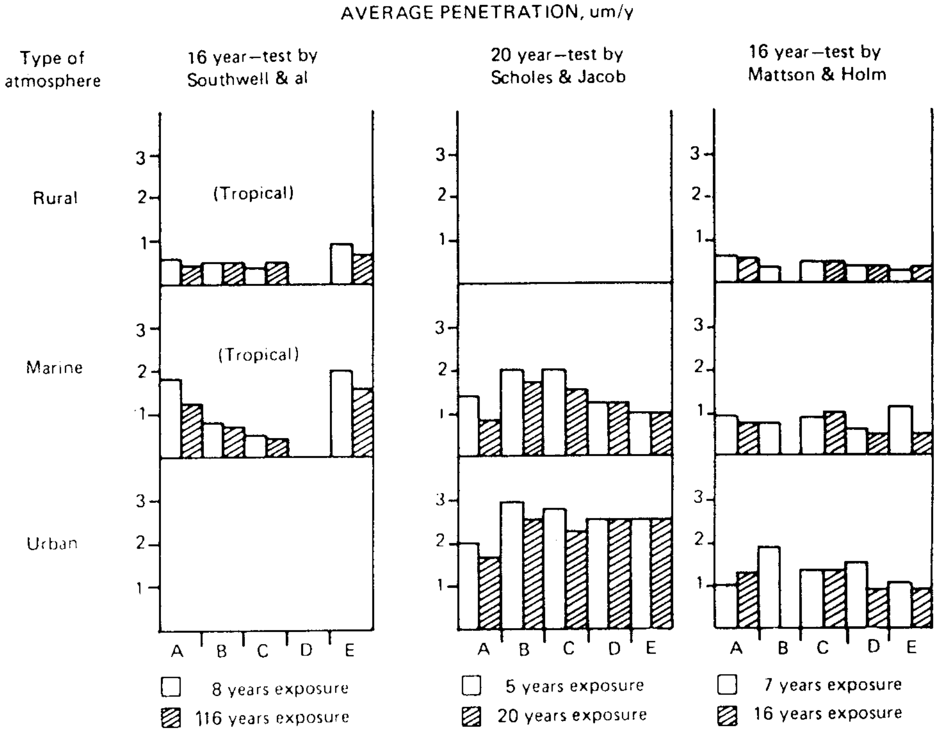

Fig.A.1:3 General corrosion rates for copper and copper alloys in the atmosphere, sloping surfaces. A = low-alloyed Cu; B = low-Zn brass; C = high-Zn brass; D = Ni silver; E = Sn bronze [3].

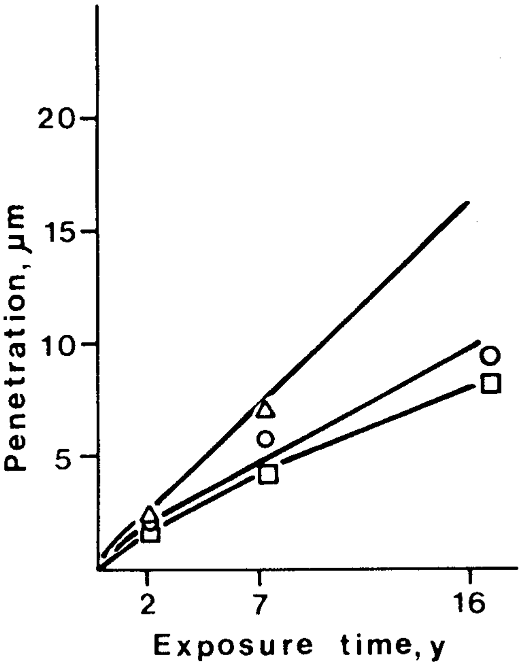

Fig.A.1:4 Penetration depth vs. exposure time for copper. Δ = urban, ❐ = marine, O = rural.

The reduction in mechanical properties due to general corrosion is usually small, in most cases less than 5% in ultimate tensile strength and less than 10 % in rupture strain [6]. For high-zinc brasses, however, greater changes occur because of dezincification [3]. Zinc is dissolved selectively from the alloy, which results in portions of porous copper with poor mechanical properties. The diagram in fig.A.1:5 shows that (α+β)-brasses are somewhat more prone to dezincification than pure α- and β-brasses, probably because of local cell interaction when α- and β-phases occur together in the structure. Small additions of As — about 0.02 - 0.04 % — inhibit dezincification in α-brass.

High-zinc brasses, if under tensile stress, are also susceptible to stress corrosion cracking on atmospheric exposure outdoors [6]. The risk of stress corrosion in brasses is greatest in industrial and urban atmospheres characterised by high concentrations of sulphur dioxide and ammonia. The stress corrosion susceptibility is markedly lower in marine atmospheres.

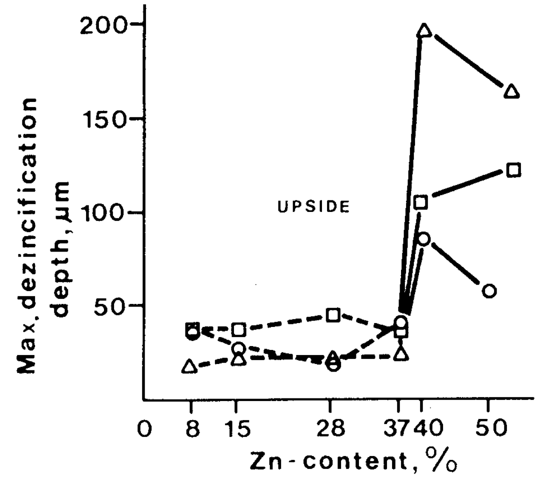

Fig.A.1:5 Maximum dezincification depth vs. proportion of zinc in binary brasses after 16 years of exposure in different atmospheres [1]. Δ = urban, ❒ = marine, O = rural.

A.1.6 References

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B Natural Stone (*T. Yates and A. Lewry*)

B.1 INTRODUCTION TO NATURAL STONE

This chapter is primarily concerned with the characterisation of natural stone used for buildings and associated hard landscaped areas. It does not cover the design or installation of buildings or the use of natural stone as aggregates or as road stone.

Stone has been used in buildings both as a structural and decorative material for a long time. Traditionally, building stone has been extracted and used within the immediate local region - which resulted in the evolution of building design to take into account the materials available and their qualities. The exceptions were major buildings where stone was imported for architectural or aesthetic reasons or because of the absence of suitable local materials. Changes in transport now allow materials to be moved across great distances and imported from almost any country in the world - leading to their use in regions or countries which are significantly different from their traditional areas. In some cases the initial driving force for the use of a particular stone is the cost, availability or colour - and only then are its qualities questioned. Some balance is needed in which the design takes account of the stone's properties or that only stones of suitable quality are used if the design is fixed. But this leaves open the question of how can quality be assessed and which qualities are important for a particular stone or design.

Natural building stones are obtained from the three basic types of rock: igneous, metamorphic and sedimentary.

Igneous rocks (e.g. granites) are those which have crystallised from molten rock or "magma''. The grain size of the igneous rocks is determined by the rate at which the molten rock cools - a slow cooling rate producing a coarse-grained rock, and a rapid cooling rate producing fine-grained rocks. Granite is the main igneous rock used for buildings but the name covers a wide range of rocks of various origins and mineral compositions. Granites contain between 50 and 75% feldspars (K, Ma, Ca, or Ba aluminous silicates) [1] that mainly determines the colour and between 10 and 35% quartz (SiO_2). These rocks are usually dense and range in grain size from fine to coarsely granular. The porosity of igneous rocks is usually very low and this, together with their mineral composition, makes them resistant to weathering. In the building industry the term `granite' is sometimes used to describe a number of igneous rocks that are not technically true granites.

Metamorphic rocks (e.g. slates and marbles) are formed by recrystallisation of the parent rock as a result of being subjected to high temperature and/or pressure. These conditions are usually caused by deep burial or movements of the earth's crust.

Sedimentary rocks (e.g. limestone and sandstone) are the main source of building stones in many countries. The formation of these rocks is a two stage process. First a sediment is deposited; this can originate from a number of sources such as fragmentation of earlier rock, the accumulation skeletons of aquatic animals, or by chemical deposition in lakes or seas. The second stage is the cementing of the sediment to form a hardrock; this is assisted by compaction and pressures generated from movements of the earth's surface.

Sandstones are formed by the fragmentation of earlier rocks, such as metamorphic gneisses and igneous rocks. The particles of parent rock were transported mainly by the action of water, leading to deposition in layers on the floors of seas, lakes and estuaries or by wind action, leading to deposition in deserts. During transport the sediments were sorted in size; in general the finer particles would have carried further than coarser ones, and the longer the sediment took to deposit, the greater the degree of sorting. The degree and method of sorting determine the texture of the sandstone.

All sandstones contain quartz (silica) along with a variety of other minerals such as mica and feldspar. Once the sediment has been laid down it can be bound together by a number of different cements:

1. siliceous — containing silica
2. calcareous — containing calcium carbonate
3. dolomitic — containing dolomite (calcium magnesium carbonate)
4. ferruginous — containing iron oxide
5. argillaceous — containing clay

The nature of the cementing material has a profound influence on the durability and the physical properties of sandstones with the siliceous sandstones generally being the most resistant to weathering.

Limestone sediments are usually formed from the skeletons and shells of aquatic animals, from chemically formed grains such as ooliths or a combination of these. Ooliths are formed by the crystallisation of calcium carbonate from solution around a nucleus that could be a fragment of shell or a grain of sand. Oolithic deposits are formed in seas where there is a tidal action that is essential for their formation and grow. Limestones whose sediments originate from aquatic organisms are usually formed under marine conditions but can be produced in freshwater.

Limestones are all cemented with calcium carbonate (calcite) and their durability and physical properties are determined more by the structure of the rock (for example the porosity and pore size) than by its chemical nature. In some cases, limestones are converted to magnesium limestones by a process called ‘dolomitisation’. In this process the calcite is gradually replaced by dolomite, a double carbonate of calcium and magnesium.

The characterisation of natural building stone is important as it allows the suitability of a stone to be assessed. Characterisation tests applicable to building stone can be divided into four groups:

1. Physical properties of the stone (for example compressive or flexural strength)
2. Durability of the stone (for example will it be frost resistant)
3. Safety of the stone in use (for example the slipperiness of paving)
4. Petrographical and microscopical properties (for example mineralogy or porosity)

Characterisation can be based on both direct measurements of some properties, where the stone is subjected to the same conditions that it would encounter in use, and also indirect measurements, where the internal structure of the stone is used to determine a different property. Indirect tests are particularly common in assessments of durability. In addition, the petrographic and microscopic properties are determined by analysis of samples that are independent of the proposed final use.

B.2 PHYSICAL PROPERTIES

The physical properties of natural stone vary widely, not only between the three major group of stones but even within a single stone type. Table B.2:1 provides a summary of published data based on [2] but with additions from others including [3] and the German Standard DIN 52 100 [4]. Table B.2:1 is divided into three groups - aggregation properties, thermal properties and mechanical properties - and these are used as the basis for the outline of each the range of appropriate properties.

B.2.1 Aggregation properties

Porosity, water absorption and permeability are all important physical properties because they characterise the internal structure and the way water can reach the interior of the stone. Water ingress is the key to the weathering of natural stones as it is involved in all the major dissolution and expansive processes that result in the decay of the stone. However, thermal and mechanical properties must also be considered in assessing decay and decay mechanisms.

Building stones are polycrystalline mineral aggregates and therefore intergranular bonding, cementing matrix, pore shape and pore size are often more important than the properties of individual grains. Density, porosity (including pore size distribution) and permeability are all determined by the relationship of the grains and their surrounding cements, but in the denser igneous and metamorphic rocks the pore space may well be in the form of fractures or cleavages along the grain boundaries.

B.2.1.1 Density

The density of a stone can be described in a number of ways. The first is the bulk density (r_b), also termed the apparent density. This is defined as the mass divided by volume (including the pore volume) and the units are g cm^{-3} or tonnes m^{-3}. The second description is the real density, which is the mass divided by the volume excluding the pore volume, for example by measuring the stone after crushing. The third term used is the bulk specific gravity, a dimensionless measure. Traditionally, stone with densities of between 1,7 to 2,2 tonnes m^{-3} have been considered as workable stone but now stones of much greater densities, often associated with better weathering properties, can be worked with modern equipment.

B.2.1.2 Porosity

Porosity (f) is the ratio of pore volume to bulk volume. In general the porosities of igneous and metamorphic rocks are low (<5%) compared to that of sedimentary rocks (up to 40%). With respect to stone decay, pores are important in that they are fluid receptacles and sites of weakness for internal stresses. Limestones, which have a predominance of pores of larger diameters, appear to be more durable than those with a very small mean pore size. Microporosity is defined as the percentage volume of pore with a diameter equal to or less than 5 mm and this appears to be a critical value for durability and frost sensitivity.

Porosity is described as 'open', i.e., pores that form an open system and are accessible by air and water, or 'closed', i.e., pore spaces that are isolated from the environment. The 'open' pores can also be divided into two groups: those that will readily absorb water at atmospheric pressure, and those that are only accessible when the sample is subjected to a vacuum or prolonged immersion.

B.2.1.3 Permeability

The ease of fluid flow through a rock is defined empirically by Darcy's law:

$$Q = \frac{mPA}{vL}$$

where:
v = fluid viscosity (poise)
Q = discharge (cm^3/s)
m = permeability (darcy)
P = pressure difference (bars)
L = flow distance (cm)
A = cross-sectional area (cm^2)

This is important because the absorption of fluids in rocks depends on the connected, effective (i.e. permeable) porosity (that is the open pores). Rock permeability can be decreased by a factor of 10 by sealing of pores due to dissolution and reprecipitation of soluble minerals. Permeability can be applied to liquids in the vapour phase as well as those present as liquids.

B.2.2 Thermal properties

The term thermal properties can be used to describe a range of parameters that describe both the movement of heat within and through a stone and the resultant effect of this movement on the mechanical properties of the stone.

B.2.2.1 Thermal expansion

Thermal expansion is usually expressed in mm per metre per degree centigrade and table B.2:1 shows that most stones have values of between 4 and 15 which for a 60°C temperature rise would result in an expansion of between 0,24 mm and 0,90 mm per linear metre.

In the most stones expansion (or contraction if the temperature falls) is fully reversible when the stone returns to its original temperature. However, there are some stones, particularly some fine to medium grained marbles that can retain a residual expansion when cooled which leads to a disruption of the crystal structure and a resultant loss of flexural strength.

Thermal stress due to differential mineral expansion and thermal gradients within a piece of stone can lead to micro-cracking between and in the mineral grains. Damage from this process is probably minimal when compared to frost damage though this can also be considered a form of thermal stress. A number of mechanical or physical properties can be used to reveal the extent of change due to thermal and frost action, these include changes in elasticity or flexural strength. Non-destructive methods can also be used, for example, ultrasonic or acoustic methods could measure the decrease in elasticity and continuity of the structure and as a result the extent of weathering on building stones.

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Table B.2:1 Physical properties for different stone types

Rock Type	Aggregation properties			Thermal properties			Mechanical properties			
	Bulk density tonnes m^{-3}	**Porosity** (%)	**Permeability** m (-logd)	**Thermal expansion** a (10^{-6} °C)	**Thermal conductivity** K (mcal/cm S °C)	**Diffusivity** k (10^{-2} cm^2s^{-1})	**Moh's Hardness** H	**Young's Modulus** E x 10^3 (Mpa)	**Compressive Strength** S (MPa)	**Modulus of rupture** R (MPa)
Granite	2,5 - 2,7	0,1 - 4,0	9 - 6	5 - 11	3 -10	0,5 - 3,0	5 ,0 - 7,0	0 - 60	80 - 330	10 - 70
Gabbro	2,8 - 3,1	0,3 - 3,0	7 - 5	4 - 7	4 - 6	1,2	5,0 - 6,5	50 - 110	110 - 300	10 - 70
Rhyolite-Andesite	2,2 - 2,5	4,0 - 15,0	8 - 2	5- 9	2 - 9	0,4 - 3,0	5,0 - 6,5	60 - 70	60 - 220	1 - 70
Basalt	2,7 - 3,1	0,1 - 5,0	5 - 1	4 - 6	2 - 5	0,4 - 1,5	4,0 - 6,5	50 - 100	50 - 290	10 - 90
Quartzite	2,5 - 2,7	0,3 - 3,0	7 - 4	10 - 12	8 - 16	2,8	4,0 - 7,0	60 - 100	110 - 360	10 - 100
Marble	2,4 - 2,8	0,4 - 5,0	6 - 3	5 - 9	3 - 7	0,5 - 1,5	2,0 - 4,0	20 - 70	40 - 190	4 - 30
Slate	2,6 - 2,9	0,1 - 5,0	11 - 8	8 - 10	3 - 9	0,5 - 3,0	3,0 - 5,0	30 - 90	50 - 310	5 - 100
Sandstone	2,0 - 2,6	1,0 - 30,0	3 - 0	8 - 12	2 - 12	0,4 - 5,0	2,0 - 7,0	3 - 80	20 - 250	1 - 40
Limestone	1,8 - 2,7	0,3 - 30,0	9 - 2	4 - 12	2 - 6	0,4 - 1,5	2,0 - 3,0	10 - 70	20 - 240	10 - 50
Shale	2,0 - 2,5	2,0 - 30,0	9 - 5	9 - 15	1 - 8	0,3 - 2,0	2,0 - 3,0	10 - 40	30 - 130	2 - 50
Soapstone	2,5 - 2,8	0,5 - 5,0	6 - 4	8 - 12	2 - 7	0,4 - 1,5	1,0	1 - 10	10 - 40	1 - 10
Travertine	2,0 - 2,7	0,5 - 5,0	5 - 2	6 - 10	2 - 5	0,4 - 1,0	2,0 - 3,0	1 - 60	10 - 150	2 - 10
Serpentinite	2,2 - 2,7	1,0 - 15,0	7 - 2	5 - 12	3 - 9	0,5 - 3,0	2,0 - 5,0	10 - 50	70 - 190	5 - 10

B.2.2.2 Thermal conductivity

This is a measure of the insulating capacity of a stone and it is a property that is becoming of greater importance in constructions where thermal efficiency is important. Thermal conductivity can be calculated using the formula:

$$K = \frac{qL}{A(t_1 - t_2)}$$

where:

K = thermal conductivity
q = rate of heat flow
L = thickness of material
A = area of isothermal surface
t_1 = temperature of hot surface

t_2 = temperature of the cold surface

In general, dense rocks have a higher thermal conductivity than porous stones.

B.2.2.3 Diffusivity

This is a measure of the rate of heat transfer across the body of a stone. From this the extent of temperature changes (how deep) and the magnitude across a stone body can be calculated.

B.2.3 Mechanical properties

This section describes a number of important mechanical and physical properties of stone that can be used a basis for its characterisation [5-7].

B.2.3.1 Hardness

Engineering hardness tests measure the resistance of the surface of a material to indentation or scratching by a hard object; the object varies from a steel sphere to a diamond pyramid depending on the test.

Mineralogists generally determine the hardness of a mineral by reference to an empirical scale of standard minerals. Mineral hardness is a crude scratch test and Mohs devised the most common scale.

B.2.3.2 Tensile properties

The tensile test measures the resistance of a material to a static or slowly applied force. From this test a stress-strain plot can be obtained where:

engineering stress: $\sigma = F/A_0$ (A_0 = original cross-sectional area)
engineering strain: $\varepsilon = (l - l_0)/l_0$ (l_0 = original distance between the gage marks)

For most materials the stress-strain curve is linear until a yield point beyond which the material does not undergo further plastic deformation until breaking point is reached. The stress at the yield point is defined as the yield strength and the maximum stress as the tensile strength. The modulus of elasticity (Young's modulus, E) is the slope of the stress-strain curve in the elastic region. This relationship is Hooke's law:

$$E = \sigma/\varepsilon$$

B.2.3.3 Compressive strength

Brittle materials are generally tested in compression since this is the mode of stressing in which they are most often used and is also the easiest mode to test. The advantage of this test is that, unlike the tensile test where necking will occur, the true stress-strain curve can be estimated for larger strains. As a rule of thumb compressive strength is a magnitude greater than bending strength, which in turn is an order of magnitude greater than tensile strength.

B.2.3.4 Modulus of rupture

This a material dependant quantity calculated from the bending strength measured by three or four-point loading. The specimen fails in tension and the failure load is dependant on the critical crack size on the specimen's surface. This can mean that the result is very dependant on the presence of fractures or flaws on the material's surface. Introducing a critical flaw, such as in a single notch bend specimen, and measuring the fracture toughness of the material can alleviate this problem.

B.3 CHEMICAL PROPERTIES

B.3.1 Igneous rock

These rocks contain mainly silicate minerals that consist of crystal lattices in which metal cations, such as Ca, Na, K, Mg, etc., are incorporated. Cations exposed at the surface may become hydrated due to unsaturated valencies in the structure. Hydrogen ions will penetrate the mineral surface and subsequently break down the silicate structure by a process of ion exchange. This will "free" ions, which can be transported away, leaving gaps in the crystal lattice and a 'weathered structure'.

B.3.2 Metamorphic rocks

Quartzites are sandstones that have recrystallised as a result of being subjected to pressure and high temperatures. Chemically they are largely silica with the result is that their chemical properties are similar to those of igneous rocks.

Slates, whose parent rock is clay-based (e.g. siltstone or mudstone), are susceptible to the same forms of ion leaching as igneous rocks. However, if impurities such as calcium carbonate and iron pyrite are present they are also susceptible to attack by acidic gases or rainwater. These reactions are expansive and the reactivity of these mineral impurities is dependent not only on the amount present but the location of the mineral within the slate's microstructure.

Marbles are derived from limestone and dolomite limestone. Their main chemical constituent is calcium carbonate or calcium/magnesium carbonate. These minerals are readily attacked by acids and acidic gases resulting in dissolution and the possible formation of expansive products particularly gypsum.

B.3.3 Sedimentary rocks

Sandstones are bound together by a number of different cements (see section B.1). In general the nature of this cement will determine the chemical properties of the stone. However, the presence of other minerals, for example clays, can lead to rapid deterioration if the stone is subjected to wetting and drying cycles.

Limestones are similar to marbles but they tend to have an increased reactivity if they are porous with a more open structure.

B.4 WHAT IS STONE USED FOR?

Traditionally, the main use of stone, such as limestone, sandstone and granite, has been in structural masonry units, however, in recent times many stone types have been used for cladding applications. Stones are also be used for flooring if they are sufficiently resistant to abrasion.

Traditional uses include roofing where slate, due to its good cleavage and weathering properties, has been exploited.

B.5 FORMS OF DEGRADATION AND FAILURE MODES OF NATURAL STONE

B.5.1 Physical weathering

B.5.1.2 Effect of moisture/temperature

The degree of moisture absorption of a stone will depend on its porosity and permeability. Due to the heterogeneous nature of stone, a temperature gradient will lead to internal stresses that may eventually result in degradation of the stone.

Freezing and thawing cycles will occur if the temperature change is around 0ºC and includes significant periods below 0ºC. The resulting decay may be caused by:

1. volume increase of water below 0ºC
2. volume increase of water to ice
3. displacement of water from frost front
4. conversion of ‘pore water’ to ordered water’

Water can be considered a corrosive element with the main damage caused during the ‘drying’ phase; this could be due to the swelling and contraction of certain minerals or drying gradients within the stone inducing mechanical stresses.

B.5.1.3 Salt crystallisation

The crystallisation of water-soluble salts, derived from the atmosphere or the soil, can cause stresses which may lead to the degradation of stone. Deposition near the stone's surface can be a major form of decay and results in surface spalling and the generation of a new fresh surface for further physical and chemical decay.

B.5.1.4 Erosion

This is normally due to wind-borne dust or sand. It can cause damage to the surface by impact and abrasion but except in arid areas is only a minor contributor to decay.

B.5.2 Chemical weathering

Carbon dioxide (CO_2) is the only naturally abundant acidic gas, producing a pH of around 5,6 in rainfalls in urban areas this may increase to 5,1 due to increased CO_2 concentrations. Industrialisation has lead to the more acidic sulphur oxides, nitrogen oxides and their derived compounds being a major contributor to acid attack.

Calcareous stones can be attacked by acidic precipitation that causes dissolution of the calcite. In polluted atmospheres calcium sulphate and calcium nitrate may form and, if exposed to rainfall, also dissolve. If calcium sulphate recrystallises gypsum will be formed which can cause subsequent salt damage as a result of the formation of expansive hydrate salts. Problems can occur if sandstone is in contact with a limestone and the gypsum is 'washed' into the sandstone where it subsequently re-crystallises.

Silicate rocks weather by ion exchange that results in discolouring and softening of the surface followed by pitting and crumbling.

Slates, which contain calcium carbonate and iron pyrite, are attacked by sulphuric acid produced by the reaction between the two impurities. Calcium sulphate is produced as a by-product and this expansive reaction results in splitting of the slate along the mica layers.

B.5.3 Biological attack

The action of higher species, such as lichens, is to degrade the stone by mechanical as well as chemical microns. Micro organisms may deteriorate stone due to the formation of by-products such as acid (inorganic and organic), sulphates and nitrates.

B.6 CHARACTERISATION OF NATURAL STONE

B.6.1 Microscopic structure

B.6.1.1 Petrography

This would involve visual examination of the rock from the quarry, either of the block or a hand specimen. The petrographic observations could include the homogeneity of the rock, e.g. mineral veins, flaws and bedding that could affect the performance in use. Sections of the rock are then normally taken and these can be polished or prepared so that there are optically transparent ("thin"). These specimens can then be examined by optical microscopy for the gross mineral composition and microstructure of the rock.

B.6.1.2 Scanning Electron Microscopy (SEM)

The utilisation of an electron beam focused on a small cross-sectional area of a sample allows spatial probing of composition and the imaging of topography. The introduction of primary electrons with a sample produces signals, for example X-ray, cathodo-luminescence, back-scattered electrons, Auger electrons, and transmitted or absorbed electrons, the energies of which are related to elemental composition. There are also signals related to the topography of the surface, such as secondary electrons and, to a lesser extent, back-scattered electrons. SEM is best used to analyse rough fracture surfaces and can be coupled with semi-quantitative or qualitative elemental analysis by the use of an X-ray detector.

B.6.1.3 Porosimetry

Examination of specimens by optical or electron microscopy coupled to an image analyser can give information about the pore structure of a rock. However, due to the nature of these techniques the sample area examined is very small.

Larger specimens can be examined by mercury intrusion porosimetry. Here mercury is pumped into the sample under pressure, the volume of mercury pumped in at different pressures can be measured and a pore size versus pore volume curve produced. These techniques are quantitative however it can result in misleading data if the mercury intrusion process damages the sample or the 'porosity' is in the form of microfractures.

B.6.2 Chemical and mineralogical properties

B.6.2.1 Petrography

Examination of polished sections by reflective and thin section by polarised/non polarised transmitted light gives information on the mineralogical composition of the rock specimen.

B.6.2.2 Electron microprobe

This works on the same principle as SEM (section B.6.1.2) except to achieve quantitative elemental analysis polished samples and a high resolution X-ray detector are used.

B.6.2.3 X-Ray Diffraction (XRD)

The interaction of a monochromatised x-ray beam with a crystalline powder will produce a characteristic pattern. Such patterns are used for qualitative analysis, leading to the identification of the phase or compound. A mixture of crystalline solids can be analysed using this technique, however, there are problems with superimposition and background noise which normally leads to a detection limit of between 1-5 weight percent.

B.6.2.4 Fourier Transform Infra-red (FTIR) Spectroscopy

Chemical bonds absorb infrared radiation by stretching or bending of the bond. Every type of chemical bond has a distinct frequency range to its bending and stretching modes. The position of the absorption peak within the frequency range is dependent on the chemical environment. Normal IR spectroscopy is carried out in the transmittance mode on ground samples suspended in gel between potassium bromide/iodide disks. However, FTIR coupled with optical microscopy allows examination of the rock specimens' surface in the reflective mode.

B.6.2.5 X-Ray Fluorescence (XRF)

Bombardment of a target with X-rays will expel an electron from an atom by the photoelectric process if the energy of the X-ray photon exceeds the binding energy of the electron. This produces a vacancy that can then be filled by outer-shell electrons resulting the emission of a characteristic X-ray. Detection of these characteristic X-rays leads to quantitative elemental analysis of the specimen. This process is the basis of the electron microprobe except electrons are the bombarding media. The probe is a surface technique whilst XRF is associated with bulk analysis. Not every ionisation will result in the emission of a characteristic X-ray. This process competes with another process in which the atom returns to its ground state by the emission of an electron known as the Auger electron. The probability of this type of event increases markedly as the atomic number decreases.

Auger electrons also have characteristic energies and give chemical as well as elemental information (B.6.2.6); the maximum energy is equal to the difference between the energy of the exciting X-rays and the binding energy.

B.6.2.6 Electron spectroscopy

This is the study of the energy distribution among the electrons ejected from a target material being irradiated by X-rays, ultraviolet radiation or electrons. The convention for distinguishing between electron spectroscopy methods is the mode of excitation. The two principal forms are X-ray Photoelectron Spectroscopy (XPS) and Ultra-violet Photoelectron Spectroscopy (UPS). A common notation is Electron Spectroscopy for Chemical Analysis (ESCA). ESCA is probably the most used technique for chemical studies due to its sensitivity to surface chemistry.

B.6.2.7 Neutron-gamma techniques

Neutron-gamma techniques involve the measurement of gamma rays that result from the interactions of neutrons with the material under analysis. Neutrons can interact by inelastic scattering from a nucleus, producing a nucleus in an excited state. Subsequent de-excitation results in the emission of characteristic gamma rays. The constraints of the method are that it only provides elemental data on a bulk sample but will provide information on low atomic number elements down to and including hydrogen.

B.6.3 Physical properties

B.6.3.1 Strength measurements

Strength measurements are important engineering properties and the mode of testing will depend on the final usage of stone. If the block is to be in compression the compressive strength is measured normally by crushing of a block and a failure load plus the stress-strain curve are measured. Flexural or bending strength is normally measured by three point loading. From the failure load a material dependent quantity, the Modules of Rupture (MOR) can be calculated by the use of the following equation:

$$\mathrm{MOR} = \frac{2Fl}{3bd^2}$$

where:

F = failure load
l = length of specimen under the two top loading points
b = breadth of the specimen
d = depth of the specimen

Care should be taken to test sufficient samples (typically 20 or more) so that the sample is statistically representative.

B.6.3.2 Abrasion resistance

When stone is used as flooring or exposed to an environment with high wind speeds and particulate matter (e.g. deserts) abrasion of the surface will occur. Typical tests to quantify this effect look at the degree of material removed when the sample is subjected to abrasion with a metal disc or blasted with sand carried by a high-speed air jet.

B.6.3.3 Co-efficient of friction

An important safety aspect of flooring is that it should have sufficient grip or friction when it is being walked on. Various tests have been devised to quantify this property. The two most commonly used types are static tests, where a weight is moved across the surface (e.g. the tortoise test), and the dynamic test, where a rubber tipped pendulum swings across the surface of the stone.

B.7 ASSESSMENT OF DURABILITY

When considering how to test for durability, an approach must be selected. Currently, durability tests can be divided into four approaches:

1. benchmark tests
2. reference materials/comparative tests
3. environmental/stress testing
4. site testing

These approaches are not mutually exclusive in that tests have been or can be designed to incorporate one or more of these approaches.

B.7.1 'Benchmark' tests

These are usually accelerated tests designed with a pass/fail criterion defined using historical data on a material's performance within a certain environment. An example of this is the British Standard for roofing slate; a slate is subjected to three accelerated tests: water absorption, wetting/drying and acid immersion. If the slate passes all three, it is deemed to be durable in the United Kingdom environment. The American Society for Testing and Materials (ASTM) test method for slate carries this approach a step further by setting a series of levels for two accelerated test (see table B.7:1); these levels are then used for service-like prediction on the basis of historical knowledge of the material in use. Benchmark tests are relatively inexpensive and quick but are limited to a single material and climate.

Table B.7:1 Service-life prediction for slate using ASTM C 406

Classification	Modulus of rupture across grain (MPa)	Water absorption (max, %)	Depth of softening (max, mm)	Service life (years)
Grade S_1	62	0,25	0,05	>75
Grade S_2	62	0,36	0,20	40-75
Grade S_3	62	0,45	0,36	20-40

B.7.2 Reference materials/comparative tests

The Building Research Establishment salt crystallisation test simulates the effects of salt change on limestone; reference materials are included in the test against which the unknown is assessed. The limestone are then placed into durability classes A to F (see table B.7:2); by experience the result is taken a stage further by defining the exposure zones of a building in which the stone can be safely used (see fig B.7:1).

Table B.7:2 Effect of environment on the suitability of limestone for the four exposure zones[a] of a building

Limestone durability class	Crystallisation loss (%)	Suitability zones for various limestone in a range of climatic conditions							
		Inland				Exposed coastal			
		Low pollution		High pollution		Low pollution		High pollution	
		No Frost	Frost	No Frost	Frost	No Frost	Frost	No Frost	Frost
		Zones[a]	Zones	Zones	Zones	Zones	Zones	Zones	Zones
A	<1	1-4	1-4	1-4	1-4	1-4	1-4	1-4	1-4
B	1 to 5	2-4	2-4	2-4	2-4	2-4	2-4	2^b-4	2^b-4
C	>5 to 10	2-4	2-4	3-4	3-4	3^b-4	4	-	-
D	>15 to 35	3-4	4	3-4	4	-	-	-	-
E	>35	4	4	4^b	-	-	-	-	-
F	Shatters early in test	4	4	-	-	-	-	-	-

[a] = The exposure zones are illustrated in fig B.7:1

[b] = Probably limited to 50 years' life

Comparative tests are likewise inexpensive and quick, with knowledge of the material, climate and in-service performance needed. Precision will be limited by the consistency of any equipment used and the reference standards.

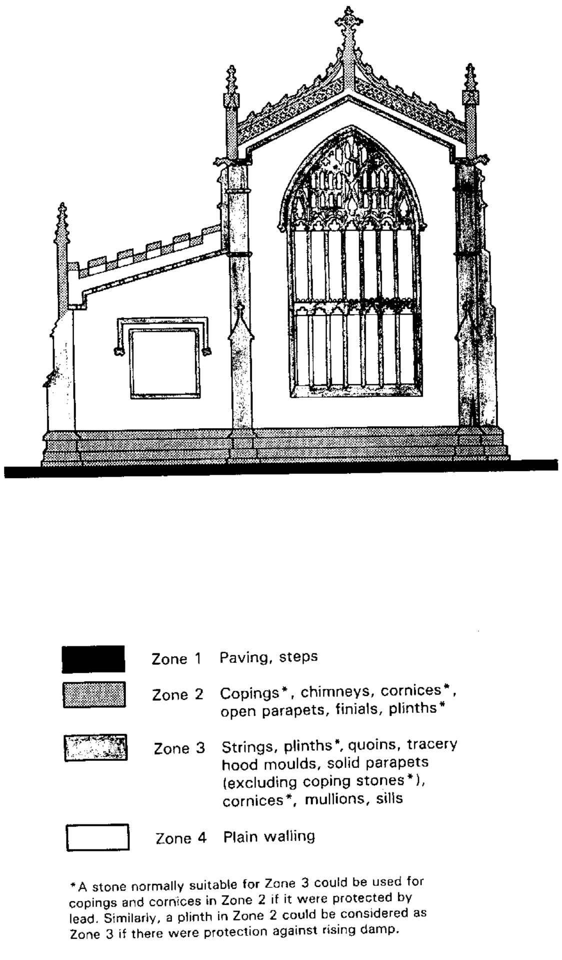

Fig.B.7:1 Exposure zones of a building in which a stone can be used

B.7.3 Environmental and stress testing

These tests try to simulate/accelerate the service environment or stress the materials by intensifying the environmental factors it is susceptible to.

Climatic chambers such as VENUS attempt to accelerate all environmental conditions; rainfall, sunlight, pollutants, temperature etc. Simpler chambers can be used to look at the effects of wet/dry surfaces, pollutants and temperature on stone degradation.

These tests do not need reference materials but are slower and more expensive, requiring confirmation of the results with on-site measurements.

B.7.4 Site testing

The building's can be monitored 'in-situ' or materials can be exposed to a range of environments. An example of large site testing project is the International Materials Exposure Programme (IMEP) which was set up to investigate the long-term impact of acid deposition on building and building materials (including stone). This programme provided a range of pollutant and meteorological climates.

Site testing is site specific with a high level of confidence but a meaningful range of environments should be selected.

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C Masonry (*T. Yates and A. Lewry*)

C.1 INTRODUCTION TO MASONRY

Masonry is a composite material made from units and mortar joints; the units can be made from a variety of materials. The European Standards Committee CEN/TC125, which is responsible for masonry, has considered seven materials for units:

- clay
- AAC
- concrete
- gypsum
- natural stone
- artificial stone

This chapter will deal mainly with clay bricks and mortar; the majority of the other materials are covered in other chapters.

C.2 CLAY BRICK MANUFACTURE AND COMPOSITION

C.2.1 Brick manufacture

Bricks are ceramic components that are generally manufactured from claystones, shales or siltstone, moulded and then fired. Bricks are formed by three distinct methods [1]:

1. Soft mud method -soft mud are poured into moulds and dried before firing. This is not a commonly used practice in modern brickmaking.
2. Stiff mud method - plastic clay is extruded through a die and then the bricks are cut to size.
3. Semi-dry or dry press method - clay is pressed into a mould.

During the above ‘forming’ processes additives are commonly used to colour the brick or inhibit efflorescence.

Apart from the composition of the starting materials, the firing temperature, the kiln type and the time kept at the temperature are probably the most important factors in determining the nature and quality of the brick produced [2].

Three principal types of kiln are used:

1. Scone kiln — constructed of unfired bricks that are plastered on the exterior. The bricks are stacked so that a channel allows heat to travel by convection. This kiln type is not commonly used;
2. Periodic kiln — the shape may be beehive, rectangular or square. These kilns are dome-roofed with fire bases around the outside of the wall base;
3. Tunnel kiln - commonly used because they are efficient and fire uniformly resulting in a chapter, reproducible product.

The firing process generally controls the physical properties of bricks; differences of 50-100°C can radically alter properties. Firing is carried out by slowly raising the temperature and allowing volatiles to escape and oxidation to occur before raising the temperature further. Generally, water is removed by heating from 100-400°C that results in a material with a porosity of 30-40 vol. % [3]. This is followed by the removal of clay hydroxyl water and oxidation in the range 400°C to 800°C. Above 800°C to 1400°C sintering or densification occurs via chemical reactions, grain growth and development of a liquid phase. The porosity decreases as a result of sintering and leads to an increase in strength of the final product.

Generally, the ideal point to stop firing results in a brick that is 'steel hard' with a low-water absorption. Overfiring wastes energy and underfiring generally results in poor quality bricks.

C.2.2 Brick composition

Bricks are normally produced from raw materials consisting of at least 50% kaolins and clays that together with water give the necessary plasticity to the material when forming it to components.

The liquid phase, produced during firing, transforms to glass without crystallisation on cooling. The result in a brick is usually composed of a crystalline silicate phase, Mullite ($3Al_2O_3.2SiO_2$), the remaining quartz from the raw materials and some minor phases, which are all bonded together by a glassy matrix. The porosity of the brick could still be as high as 20% and the glassy phase could constitute 60% of the brick. This means that the brick is really a composite with the crystalline phases embedded in the glassy matrix whose typical composition is given in Table C.2:1.

Table C.2:1 The approximate composition of the glass in a ceramic such as brick [4]

Component	Wt %
SiO_2	71-77
TiO_2	1-4
Al_2O_3	7-16
Fe_2O_3	1-5
MgO	0-2
CaO	0-1
Na_2O	0-3
K_2O	4-8

C.3 MORTAR COMPOSITION AND MANUFACTURE

Mortar is a cementitious material made of a binding agent, such as Portland cement and/or hydrated lime, mixed with water and sand as filler. The sand is usually screened in order to avoid too coarse or fine particles.

Cement manufacturing processes can be wet (ground wet and fed to the kiln in a slurry), semi-dry (ground dry and moistened before feeding to the kiln) or dry (ground dry and added as a dry powder). The dry processes are more recent developments that consume less energy. Cement is highly alkaline, having a pH of 12-13. Cement powder is stored and distributed in either bulk or bag form.

Cement is derived from mixing limestone/chalk and clay or shale, together with other minor materials that impart different properties to the resulting product. There are four stages in the manufacture of cement:

1. Crushing and grinding the raw materials;
2. Blending the materials in the correct proportions;
3. Burning the prepared mix in a kiln to produce cement clinker, in the past, bottle or chamber, later continuous shaft kilns were used; modern works use rotary kilns, fired by coal, oil or gas;
4. Grinding the cement clinker together with about 5% gypsum/anhydrite in a ball mill to produce a fine powder; the gypsum can be used to control the setting time of the cement.

The constituents of different cements are as follows:

C.3.1 Portland cement

Ordinary Portland Cement (OPC) comprises approximately 60-67% lime, 19-25% silica and 3-8% alumina, with varying small amounts of iron oxide, sulphur trioxide, magnesia, titanium oxide and manganese oxide.

OPC in its unhydrated form consists mainly of four minerals:

1. Tricalcium silicate, $3Ca0SiO_2$ (abbreviated to C_3S);
2. Dicalcium silicate, $2CaOSiO_2$ (C_3S);
3. Tricalcium aluminate, $3CaAl_2O_3$ (C_3A);
4. Tetracalcium aluminate-ferrite, $4CaOAl_2O_3FeO_2$.

Modified Portland cements include:

• Sulphate resistant	These contain less than 3% tricalcium aluminate
• Ultra-high early strength	Extra gypsum is added to the mix
• White	Obtained by careful selection of raw materials
• Coloured	5-10% pigment is added to the mix
• Air entrained	A plasticiser is added to improve frost resistance and workability
• Waterproofing	Small amounts of calcium and aluminium compounds are added to the mix in the final grinding stage
• Rapid Hardening	The OPC is finer

When mixed with water, cement undergoes a sequence of hydration reactions that slowly transform the cement paste to a hardened matrix of hydrated products. The most important hydration reactions are those involving C_3S and C_2S that leads to the formation of calcium silicate hydrate (C-S-H) gel and calcium hydroxide (Portlandite).

C.3.2 High alumina cement

A mixture of bauxite and lime containing the following:

- 35-40% lime
- 40-50% alumina
- up to 25% iron oxide
- no more than 5% silica.

C.3.3 Slag cements

• Portland blast-furnace slag cement	A mixture of Portland cement and 50-90% blast-furnace slag
• Supersulphated blast-furnace slag	A mixture of blast-furnace slag, calcium sulphate and a small amount of Portland cement
• Portland pulverised fuel ash cement	A mixture of Portland cement and pulverised fuel ash (PFA).

C.3.4 Lime mortar

Lime is produced by burning crushed, high purity limestone/chalk (96-98% calcium carbonate) in kilns, at temperatures of between 800°C and 1500°C. At these temperatures carbon dioxide gas is liberated and the oxide is produced. The lime is cooled, stored and used as the burnt lime (quick lime), or mixed with water and used as the hydrated lime (slaked lime).

Although the native of mortar has changed considerably over time, modern masonry mortar is usually a mixture of ordinary Portland cement (OPC), sand, lime and additives (for example plasticizers or air-entrapment agents).

C.4 PHYSICAL PROPERTIES

The physical properties of brick and mortar vary widely, however, they can still be described by those laid out in the stone chapter (B.2) — aggregation, thermal and mechanical.

Brick properties will depend on the firing temperature composition of the raw materials, forming methods and the firing conditions. Firing of a brick does not alter the pore size distribution but will lower total porosity, thus increasing final strength. However, insufficient firing will result in a differential pore structure i.e. the surface has a much lower porosity than the interior. The raw materials control the pore size distribution of the final product; clay have small pores but the addition of sand and crushed brick shifts the distribution towards larger pore sizes. Forming processes such as hand moulded and pressing led to bricks with large pore sizes whilst extrusion produces a product with small pore sizes. Although porosity is important other factors such as water permeability and mechanical strength must be taken in account.

Mortars have two distinct, important sets of properties, which are those of fresh mortars and of hardened mortars. Mortar serves to cushion the units of masonry, giving them full mutually bearing, in spite of their surface irregularities. Furthermore, the mortar seals the space between the units and/or the wall surface, preventing water, damp and wind from penetrating. Therefore not only the properties of hardened mortar, but also those of fresh mortar relate to the properties of masonry. The physical properties of a hardened mortar, determining the structural performance, will be dependent on the composition of the mix and the setting conditions. These properties include bond strength, compressive strength, elasticity, permeability and durability. Types of masonry mortars, classified according to compressive strength, are given in table C.4:1 [5].

Table C.4:1 Types of mortar

Mortar type	**Construction stability**	**Typical average compressive strength (MN/m^2) at 28 days**
High-strength mortar	Masonry subjected to high lateral or compressive loads, or severe frost action; masonry below ground	17,3
Medium high-strength mortar	Masonry requiring high flexural bond strength, but subjected only to normal compressive loads	12,4
Medium strength mortar	General use above ground	5,2
Medium low-strength mortar	Non-loadbearing interior walls and partitions	2,4

The physical properties of a masonry wall depend on both the mortar and the masonry units, as well as on the compatibility between the two. The relatively small part of mortar in masonry significantly influences the total performance. Compressive strength of mortar increases with increasing cement content and decreases with increasing lime, sand, water and air content. It is useless, however, to use a very strong mortar the masonry units themselves are weak. Although compressive strength is a prime consideration in concrete, it is only one of several important factors in mortar. Since strong mortars often leads to shrinkage cracking, suitable selection of the type of mortar needs to be done for satisfying a variety of diverse requirements.

Adding epoxies or latexes to the mix can create mortars with exceptionally high strengths and adhesive qualities.

C.5 CHEMICAL PROPERTIES

C.5.1 Brick

The brick's glassy matrix is similar to acidic silicate glasses and as a result is probably the most susceptible component to dissolution by neutral, acid or alkaline solutions. Water will leach alkali ions from the surface resulting in an alkali-ion depleted layer, which can subsequently be removed by dissolution. Bricks can contain N_2SO_4 if the fuels during firing are sulphur rich or the temperature is insufficient to decompose the salt.

C.5.2 Mortar

Non-hydraulic lime mortar depends on exposure to air to harden:

$$Ca(OH)_2\ (s) + CO_2\ (g) \rightarrow CaCO_3(s) + H_2O\ (l)$$

On hardening, carbonation and drying occur simultaneously resulting in a volume contraction. In the case of dolomitic mortars magnesium phases are also present.

Hydraulic mortars contain Portland cement whose main constituents are Portlandite ($Ca(OH)_2$) and calcium silicate hydrate (C-S-H). Naturally acidic rainfall will leach $CaCO_3$ from the surface, however, while being detrimental to any steel reinforcement due, carbonation has been observed to double the strength of OPC mortars and only usually changes the appearance, i.e. staining and efflorescence.

Accelerated carbonation tests were carried out on mortars [6]. OPC was mixed with river sand whose fineness modulus was 3,01. Two types of mortars were produced, whose sets of water-cement ratio and sand cement ratio were 0,6 and 3,22, and 0,7 and 3,75, respectively. Flow of both mortars was about 205 mm. Fig C.5:1 shows results of the accelerated carbonation test, which was performed in the following condition: temperature 30°C; relative humidity 50 %; and CO_2 gas concentration 10 %.

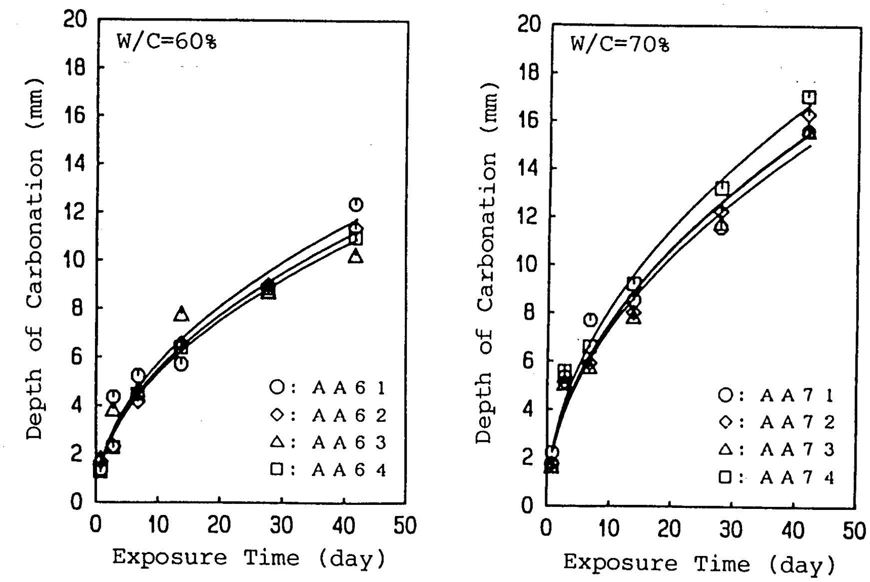

Fig.C.5:1 Accelerated carbonation test results

Solid lines in fig C.5:1 were drawn for different specimens using

$$d = a\sqrt{t}$$

where d is the depth of carbonation, a is a constant and t is the exposure time (days). Fig C.5:2 shows that the carbonation depth after exposure in the test environment for about 50 days is influenced mostly by coarse pores larger than 75 nm.

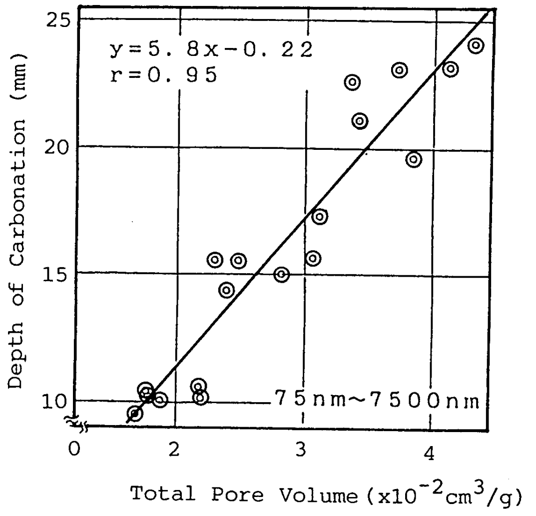

Fig C.5:2 Relation between total pore volume larger than 75 nm and the depth of carbonation after 50 days exposure [6]

When considering brick masonry as a whole it has been shown that the relative porosity of the brick to the mortar is important, with the most porous material being the site of salt-induced attack.

Mortars contain Portlandite that can react with sulphur dioxide to form gypsum. The resulting result could further degrade the mortar by reacting with calcium aluminate (C_3A) to produce ettringite ($3CaO.Al_2O_3.3CaSO_4.31H_2O$). Gypsum and ettringite formation are both expansive reactions and can result in cracking.

C.5.3 Incompatibility

The porosity of the brick is also important when considering the setting of the mortar. If the brick contains large pore sizes, the suction effect is fairly low and has been shown to produce a stronger mortar/brick bond. However, this also produces a mortar with a lower strength that is less frost resistant. This situation will also depend on other factors, for example the water saturation level of the brick, the possibility of water retaining additives in the mortar, etc.

C.6 TYPICAL MICRO ENVIRONMENT AND LOADS

The micro environment and loads affecting mortar are movement of water, including moisture, and tensile stress due to shrinkage. Rain penetration is likely to occur mainly through shrinkage cracks rather than directly through masonry units or the mortar itself. Rain and melting snow penetrate the surface, and the mortar is subjected to freeze/thaw cycles during winter in cold regions. Some shrinkage of mortar is inevitable; the amount depends to some extent on the width of mortar joints and the mixture of mortar.

Mechanical loads are of compression under the weight of subsequent courses of masonry, and of shear and tension under wind and seismic actions. Differential settlements of foundations also cause shear stress in the mortar.

Porosity and permeability are the most important properties when considering the accessibility of water to the interior of the brick. It has been shown that the pore size distribution has a critical pore size, in the range of 1-3 μm. In general large pores and low porosity result in a durable product.

C.7 FORMS OF DEGRADATION AND FAILURE MODES OF BRICK MASONRY

C.7.1 Physical failure

The most typical degradation and failure mode of mortar is cracking. Drying shrinkage and moisture movement are the main sources of cracking, besides mechanical loads.

In larger elements, such as load bearing walls, which are restrained from movement, the type of failure is usually attributed to cracking of the mortar. Because of tensile induced stress due to shrinkage, cracking tends to be in the vertical direction. In walls built with small masonry units, the cracking may take place along vertical and horizontal joints, thus forming a step pattern.

Degradation of mortar due to frost action is mainly associated with fine cracks. However, these cracks normally are of less significance in comparison to cracks that may result from thermal and moisture movement. The use of finely graded sands in the mortar mix has been shown to increase strength and frost resistance. Differential pore structures, between the surface and interior, can cause spalling in brick due to prevention of water migration by the lower porosity surface.

C.7.2 Chemical weathering

Water-soluble salts, derived from the atmosphere or the materials themselves (e.g. Na_2SO_4 from brick), will degrade brickwork; the result is spalling of the surface.

Acids will attack the glassy matrix of brick and it has been estimated that weathering due to pollutants in industrial area will increase the dissolution of brick 10-fold. However, the maximum dissolution of brick is approximately 1%.

C.8 CHARACTERISATION OF BRICK MASONRY

In general bricks can be characterised, after production, using the techniques used in the stone chapter (B.6). However, mortar is an in-situ material and therefore mixes with known properties have to be prepared or in-situ testing has to be done on the setting material before application.

The most typical degradation monitoring method for mortar may be to observe the cracking properties, i.e. pattern, density and width distribution. From the cracking pattern the reason for the degradation can be judged. Information on crack density and width distribution may help to estimate the service life and suggest repair methods.

While the traditionally method to observe cracking by eye and sketching is still used, advanced systems have been developed in which crack properties recorded with camera are assessed using an image analyser.

C.9 REFERENCES

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D Clay (*H-J. Schwarz*)

D.1 INTRODUCTION

One third of mankind lives in earth buildings. Such dwellings are especially found in the dry and hot tropical countries, where the manpower is cheap and modern building materials, requiring a huge amount of energy to produce, are very expensive. But one might also find such dwellings in central Europe, mainly for the construction of half-timbered homes, and in these instances, clay is used as a building material.

The service life of clay constructions can be much more than 100 years if carefully protected against water erosion. All other environmental attack is insignificant.

Clay as a building material is very easy to use and to repair, and no specialist is necessary for the construction work. Everybody is able to do it and in addition, the costs are very low in most countries. Only in the industrialised countries is clay a bit more expensive than some other building materials.

Clay is an ecological product with a very good eco-balance. It is usually used close to the excavation site and the transport costs are low. Only a little energy is needed to manufacture clay building materials and in most cases no energy demanding additives are necessary.

D.2 CLAY AND ITS COMPOSITION

Clay as a building material is a mixture of gravel, sand and pure clay. The latter is formed by the weathering of consolidated rocks into fine particles. Clay as a primary weathering product at the place of the formation of the original rock should be distinguished from secondary clay deposits, to which the clay is moved after the weathering. Secondary clays often show a finer grading curve.

Clay as a building material is a tenacious mineral rock from which the smallest fraction of clay minerals is responsible for the tenacious character. Depending on the composition and the origin of the clay, many types of clay with different characteristics could be distinguished, for example morainic clay, slope wash, meadow loam, loess clay.

The classification of constituents of the clay by their grain size means that also in the clay fraction minerals could be found, which do not contribute to the tenacious character. Only the fraction of the clay minerals itself has to be considered if the tenacious character is the question.

Clay minerals are mainly water-bearing alumosilicates. They are formed essentially at weathering of silicates. In addition, iron minerals like hematite and goethite, titanium dioxide as anatas, and in the tropics aluminium hydroxides — mainly gibbsite, are frequently found.

Clays also contain weathering residuals and other newly formed minerals. Weathering residuals are mainly resistant minerals as quartz, muscovite and feldspars, sometimes biotite and rarely chlorite.

The accessory minerals of the original rocks, as zircon and rutile and to some extent apatite, are enriched in the sediments. Newly formed minerals frequently found in clays are the silicate glauconite, the carbonates calcite, dolomite, siderite, the iron sulphides pyrite, markasite and chalcopyrite and galenite. The oxidation of the sulphides gives secondary sulphates like gypsum, jarosite and alunite [1].

Responsible for the typical properties of clays — the soapy consistency, the water binding power, the swelling ability, high adsorption capacity for many inorganic and organic materials, the thixotropy, the plasticity — are the siliceous clay minerals.

According to the main mineral constituents, clays are divided in kaolinite rich and smectite rich (bentonites) clays. The common clays mainly contain illite and some chlorite, kaolinite, smectites and mixed layer minerals (smectite/illite and kaolinite/illite). The last ones are the usual clays for building materials, while the other, more pure clays are rather used in the ceramic, paper and food industry.

The different properties of clays with the same amount of clay-sized particles are based on the mineralogy of the clay minerals. Characteristic is the small mean grain size, which for the smectites falls far below 2 µm. Kaolinic clays are much more coarse and contain in many cases nearly no fraction < 0,2 µm. The mineralogical composition of a clay alters with the grain size of the particles, that means a finer clay has a different composition than a coarser clay.

Kaolinite often forms more or less well-shaped six-cornered plates, which are put together book or money roll like. Illites are mostly lath-like. Montmorrillonites form thin crystals that look like pieces of foil with irregular edges, bent, sometimes folded or rolled at the edges.

A characteristic property of the clay minerals is their cation exchange ability; the anion exchange ability is rather low and is found only in acid environments. The kaolinites have exchangeable ions only at the surfaces. The exchange ability of the smectites and vermiculites is determined by the inter layer cations and their cation exchange capacity is much higher compared to the kaolinites. In the micas the potassium ions are strong bonded between the silicate layers and the ion exchange requires very strong reaction conditions.

The crystal structure of the clay minerals is not very complicated. Clay minerals are made up of $[SiO_4]$ tetrahedrons and $[Me(O,OH)_6]$ octahedrons (Me — metal cation). The tetrahedrons and octahedrons are linked together by common atoms.

The so-called two layer clay minerals, like kaolinite, are composed of layers of one tetrahedral and one octahedral layer. They have aluminium in the octahedral layer and the formula is $\{Al_2[Si_2O_5(OH)_4]\}$. In the so-called three layer minerals the octahedral layer lies between two tetrahedral layers. The group has a very great variety in the composition. In the three layer minerals the layers are negatively charged. To compensate this negative charge cations are bonded between the layers. In addition, water molecules can be present between the silicate layers, which leads to complex formulas. For this reason no formula is given here.

D.3 PLASTICITY OF CLAYS

The tenacity is decisively influenced by the exchangeable cations. For example, with calcium ions house-of-cards and ribbon-like structures are formed and kaolinite is able to bond more than 50% water by volume.

In moist conditions the mineral plates could be moved against each other. After moving, the plates always find new positions where they are fixed by the calcium ions. The clay is plastic.

The cavities survive the drying to a great extent, i.e. the mineral plate structure resists the shrinkage to a dense mass.

Clays always contain a certain amount of quartz that can be bond by the clay minerals. The clay mineral plates surround the quartz grains and create large contact surfaces, which are preserved on drying and give the high strength. The fewer cavities the mass has after drying, the stronger is it. The more thin and flexible the plates are, the less clay is necessary for a strong bonding. Therefore the grading curve of the clay is an important parameter. The smectites have very good properties in this respect.

If sodium ions are solved in the water present, the structure of the kaolinite plates brakes down; a ceramic mass could become a slurry, which can be poured and later on formed to a mass with very high density and dry strength. This mass is no longer plastic.

D.4 CLASSIFICATION OF THE CLAY BUILDING MATERIALS (TECHNICAL PARAMETERS)

The clay building materials can be divided according to their clay content and their technical parameters.

In general, building clays are divided in fat clays with a high content of clay minerals, lean clays with a low content of clay minerals and medium fat clays in between.

Some important characteristic parameters are collected in table D.4:1. The technical parameters are explained in more detail below.

Table D.4:1 Characteristic parameters of clays

Parameters	**Lean**	**Medium fat**	**Fat**
Clay content	low	middle	high
Uptake of water	high	middle	low
Swelling up/shrinkage	weak	middle	high
Tenacity	low	middle	high

D.4.1 Sieve and sedimentation analysis

The constituents of clay are classified according to its grain size. The percentage of each fraction is determined by a sieve analyses which gives the grading curve, represented graphically in fig D.4:1.

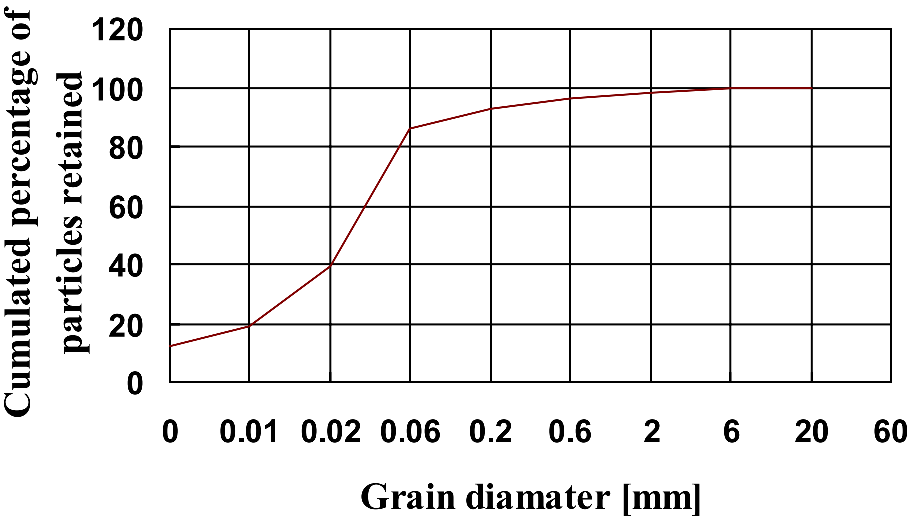

Fig.D.4:1 Grading curve of a silty clay

D.4.2 Atterberg consistency limits

The different states of tenacious soils, referring to the consistency, are divided according to Atterberg [2] in fluid, plastic, semi-solid and solid.

The "liquid limit" w_L is determined by the water content at the transition point from the liquid to the plastic state. In the laboratory the liquid limit is measured by an apparatus for plasticity test after Cassagrande [3].

The "plastic (rolling-out) limit" w_P is determined by the water content at the transition point from plastic to the semi-solid state. It is measured by the rolling-out test [3].

The "plasticity index" I_P is the difference between the liquid limit and the plastic limit, w_L-w_P, and characterises the plastic behaviour of a tenacious soil like a building clay.

D.4.3 Compressive strength

The compressive strength of dry building clay is determined by its cohesion (depending on the clay content) and the grain friction. For the fine building clay materials the compressive strength increases with the clay content. For the coarse building clay material the clay fraction can be lower to get the required compressive strength (due to grain friction).

The compressive strength can be increased by suitable treatment of the building clay (ramming, kneading, suspending, compressing). An increase in compressive strength can also be obtained by additives. It should be kept in mind that additives not only have positive effects on the properties of a building clay but also negative ones.

Usually the compressive strength as estimated by the simple compressive experiment is sufficient reliable. In the simple compressive experiment the "cross" tensile strength of the material is dominant and is measured simultaneously. Therefore it is not necessary to measure separately the tensile strength, the tensile bending strength or the shear strength. Only for the testing of clay plaster and to assess the stability of the edges of clay building stones the tensile bending strength is important. The bonding strength plays a part when testing clay plasters.

The compressive strength of dry clay building units varies generally between 0,5 to 5,0 N/mm^2. Load bearing clay building units should always be tested for their compressive strength. It should be at minimum 2 N/mm^2.

Closely connected to the strength parameters is the modulus of elasticity, in the range 6000 to 7000 N/mm^2.

D.4.4 Proctor density and the best water content

Proctor density and the best water content are the result of the Proctor test [4].

The water content of a building clay mixture is optimised when the maximum dry-density is reached with minimum compression energy. If the water content is too low, the compression energy will be higher. If the water content is too high, the compression energy cannot be absorbed and the dry-density will be reduced.

In the Proctor test a building clay sample is compressed under well-defined boundary conditions. The water content of the sample is varied several times during the experiment. After each run the water content and the dry-density of the sample are measured. At the end the dry-densities could be represented as a function of the water content.

The peak of the Proctor curve gives the maximum dry-density under the defined compressive conditions, the so-called Proctor density ρ_{Pr}. This is estimated for:

- "fat" clay (more than 50% clay) as $\rho_{Pr} \approx 2000$ kg/m^3
- sandy clay (more than 50% sand) as $\rho_{Pr} \approx 2200$ kg/m^3
- gravely clay (more than 50% gravel) as $\rho_{Pr} \approx 2500$ kg/m^3

The corresponding water content is the best water content w_{Pr}. If the water content is lower or higher compared to w_{Pr}, the dry-densities obtained will in both cases be lower than ρ_{Pr}.

The best water content w_{Pr} is important especially for rammed earth constructions. According to the Proctor curves this water content is reached at about 12,5%. Is the clay rammed at a higher water content, in addition to a lower dry-density, more shrinkage cracks and settlement movements occur. At a water content of more than 2% below the best value, the required compressive strength cannot be reached by ramming.

These best values obtained with the compressing experiment are the basis for the compression that should be reached during the realisation of the construction. The Proctor test is suitable to building clays for rammed earth constructions and pressed adobes.

D.4.5 Capillary uptake of water

Basis for the method to determine the uptake of water coefficient is the DIN 52617 [5], which has to be modified in the application to water-sensitive clays.

According to DIN 52617 the uptake-of-water coefficient w of a wet sample is related to the water uptake per unit area and time according to:

$$W = w \cdot \sqrt{t}$$

where:

W [kg/m^2]	uptake of water per unit area
w [kg/m^{2}h^{½}]	uptake-of-water coefficient
t [h]	time

For the determination of the uptake of water, a sample, watertight at the side faces, is initially dried to mass equilibrium, and then dipped in water to wet the suction area. By weighting the sample before and after the wetting in distinct time intervals, the amount of water taken up can be found.

During the examination of several heavyweight clays uptake of water coefficients (w) between 1,1 to 5,5 kg/m^{2}h^{½} have been found [6]. The capillary water capacity (Φ_k) of these clays was between 0,18 to 0,32 m^3/m^3.

Bentonite-sand mixtures and fat clays with a high content of swelling clay minerals have low w-values, see fig D.4:2.

The experimental results concerning the investigations of the influence of the compression on the capillary uptake of water are not clear. Considering fat and silty clays, the w-values for stamped samples with a high raw density are lower than those of hand-made samples.

Sandy clay samples behave in the opposite way. Here the w-value of the stamped samples — with a higher raw density as well — lies clearly higher than the values of the hand made samples.

Obviously several phenomena overlap, such as the mineralogical composition and the pore distribution. Uncompressed very sandy clay has such great pores that the capillary force is substantially diminished.

Compared with other building materials with similar pore structure, the capillary uptake of water is a very slow process. This reduced absorption rate is caused by the swelling characteristic of the clay minerals and the associated sealing effect.

Stabilising building clays by adding about 2% cement alters the pore structure to such an extent that the uptake-of-water coefficient increases up to 22,5 kg/m^{2}h^{½}, some 16 times higher than for non-stabilised clays, and thus in parity with the absorption rate of a weak burned clay brick.

The absorption rates of light-weight and heavy-weight clays are of the same order, see fig D.4:2-3. In the region of low bulk densities < 500 kg/m^3, a marked difference between straw light-weight clays and mineral light-weight clays exists. For straw lightweight clays the capillary absorption capacity of the plant fibres and the higher portion of capillary pores become effective. The expanded clay aggregates, on the other hand, show a very high degree of closed pores. Even more obvious is the difference of the water capacity.

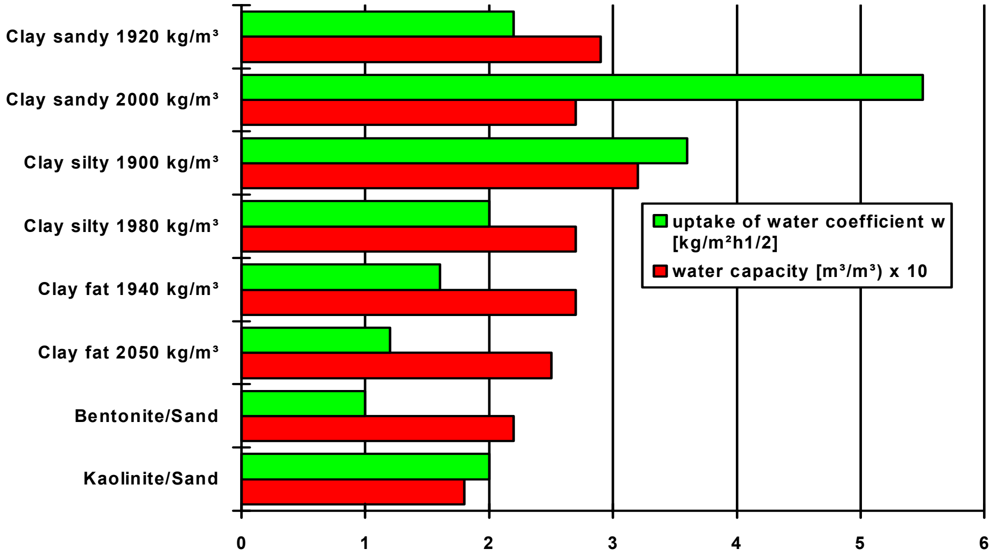

Fig.D.4:2 Uptake-of-water coefficient and capillary water capacity of heavy-weight clays [6]

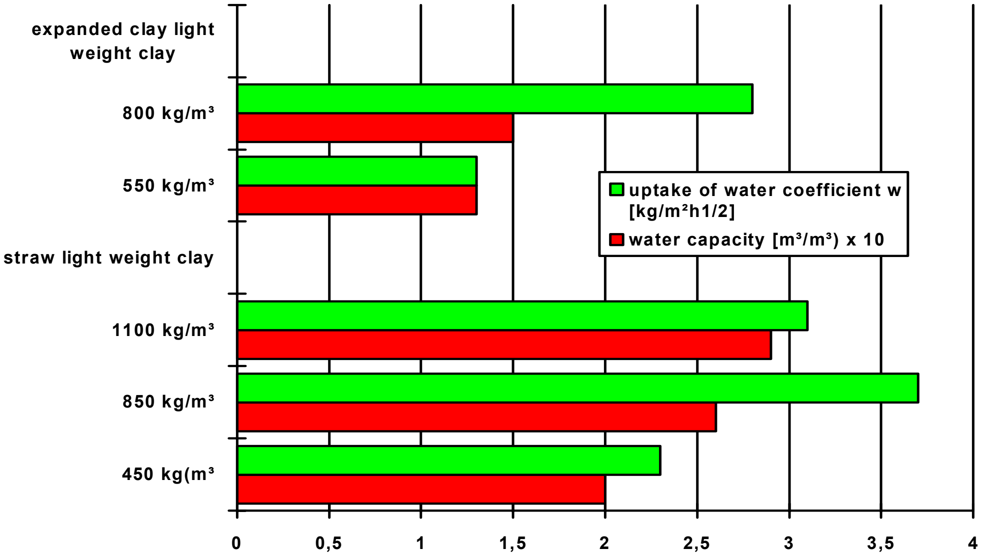

Fig.D.4:3 Uptake-of-water coefficient and capillary water capacity of light-weight clays [6]

D.4.6 Water vapour diffusion

Basis of the method is according to DIN 52615 [7].

The relative water vapour resistivity μ represents the capability of a building material to hinder diffusion of air humidity. The μ-value is the ratio of the thickness of an air layer to the thickness of a layer of the building material considered, both layers yielding the same water vapour resistance. For air, by definition $\mu \equiv 1$, and for solid materials $\mu > 1$, see table D.4:2.

There are two different methods to measure the relative water vapour resistivity: the wet cup and the dry cup method. The results differ between the methods, see table D.4:3. For example, for heavyweight clay with the wet cup method the μ values vary between 6 to 8, whereas with the dry cup method the μ values are higher, varying between 15 to 23.

Table D.4:2 Bulk density and relative water vapour resistivity according to [8]

Material	**Bulk density [kg/m^3]**	**μ- mean at 20°C / 60% RH**
Heavy-weight clay	2000	10 - 11
Light-weight clay	1200	8 - 10
”	900	6 - 8
”	600	5 - 6
”	300	4 - 5
Brick	1100	4 - 6
Lime sandstone	1600	10 - 25
Gas concrete stones	400	3 - 5
Wood (spruce, fir)	450 - 500	20 - 40
Wood wool building slabs	350 - 500	2 - 5

Table D.4:3 Bulk density and relative water vapour resistivity according to [6]

Material	**Bulk density [kg/m^3]**	**μ- mean at 20°C / 60% RH**
		dry cup method
Heavy-weight clay, fat	2050	23,0
”	1960	19,5
Heavy-weight clay, silty	1980	15,5
”	1890	15,1
Heavy-weight clay, sandy	2060	22,5
”	1880	18,2
		wet cup method
Heavy-weight clay, fat	2050	7,0
”	1960	6,8
Heavy-weight clay, silty	1980	6,0
”	1890	5,9
Heavy-weight clay, sandy	2060	7,5
”	1880	7,2
Expanded clay, light-weight clay	650	6,8
”	800	8,1
Straw, light-weight clay	450	2,2
”	950	3,1
”	1250	4,4
Clay plaster, silty		9,8
Clay plaster, fat		8,0

D.4.7 Uptake of water — hygroscopicity

Building clays have very favourable hygroscopic properties. They can exchange great amounts of humidity with the environment and have better humidity regulating properties than all other building materials.

To facilitate good interaction with water vapour, materials with poor hygroscopic or diffusion hindering properties should be avoided, e.g. tight coatings.

Usually climatic conditions are not stationary, and then materials are not in equilibrium with their environment. In this case, the limiting factor for the rate of vapour exchanged between the environment and a hygroscopic component is normally the transport rate at the surface rather than any interior gradient of the material moisture concentration.

To determine the equilibrium humidity, small samples should be put above saturated salt solutions. The difference between the mass of the sample in equilibrium with the saturated salt solution and the dry mass measured afterwards gives the moisture content.

The time dependency of water absorption is determined by measuring the mass increase as a function of time of a sample, put in higher relative humidity after drying at about 50% RH. Comparing unburned bricks with bricks of the same shape, the unburned material takes up about 30 times the amount of water than the burned material in two days [9], rising the relative humidity from 50% to 80 % (see also fig. D.4.4.).

Absorption and desorption take place first at the surface of the building component. Subsequently deeper zones will be affected.

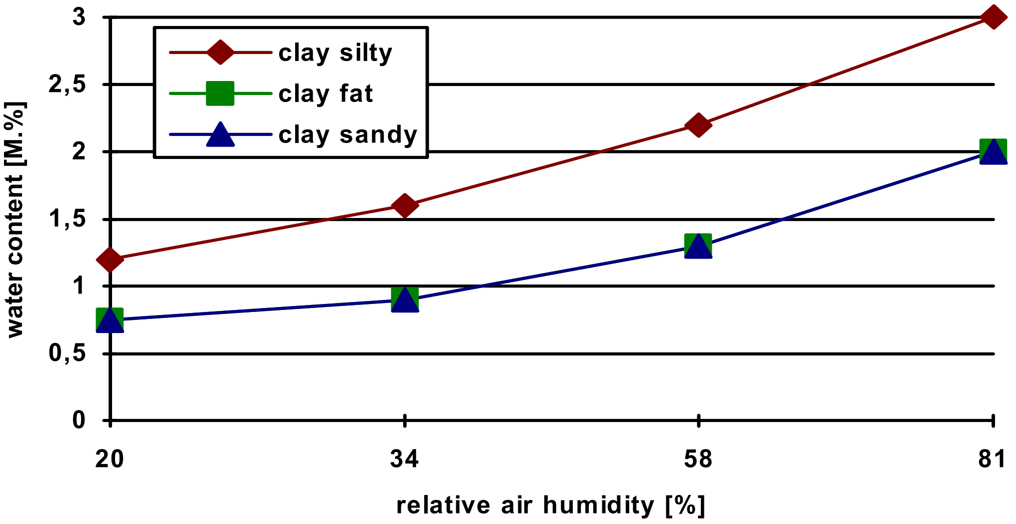

Fig.D.4:4 Equilibrium humidity of heavy-weight clays [6]

Straw light-weight clay shows increasing equilibrium humidity with decreasing bulk density, as can be seen in fig.D.4:5. This can be explained by the increasing straw content. Expanded clay lightweight clay shows decreasing equilibrium humidity with decreasing bulk densities, caused by the closed pores of the mineral aggregates.

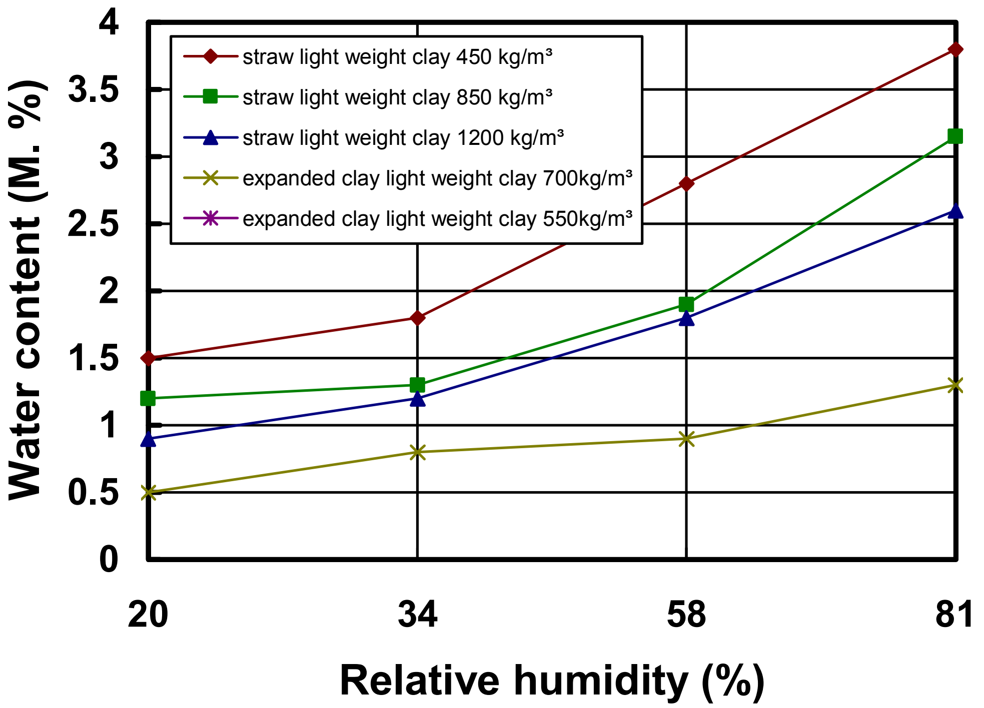

Fig.D.4:5 Equilibrium humidity of light-weight clays [6]

The type of the clay minerals has also a great influence. Thus the equilibrium humidity of a mixture of montmorillonite and illite (60% RH, 14,8 M% water content) lies by the factor 17 higher than the equilibrium humidity of kaolinite (60% RH, 0,8 M% water content) and reaches nearly the same value as loose rye straw (60% RH, 12,8 M% water content) [9].

The absorption rate of all clay samples is greatest just after the change in air humidity and decreases continuously. The water amount absorbed is approximately proportional to the square root of the absorption time $\sqrt{t}$.

D.4.8 Shrinkage — drying

Corresponding to the uptake-of-water propensity of clays, during the drying of clay building materials a corresponding shrinkage takes place. Fat clay with the same consistency as a lean clay takes up more water because of its higher clay content.

In case of a very fast and intense drying, the shrinkage leads to tensions in the clay that result in fissures and cracks. This fact has to be considered especially with clay rendering. Fat and very fat clays with a high shrinkage potential are not suitable for rammed earth constructions. To keep the shrinkage within limits, clays with a high portion of clay minerals have to be shortened by aggregates.

Another possibility to prevent the disadvantage of the shrinkage is to work with small prefabricated building units where the shrinkage has taken place before the construction.

The linear dry measure of shrinkage is determined with test samples (prisms) with a length of at least 20 cm, starting with a water content near the rolling out limit.

The dry measure of shrinkage results from the mean value of at least three test samples of the same clay. This measure could also be used to classify building clays, confer table D.4:4.

Table D.4:4 Dry measure of shrinkage of clays [8]

Clay	Dry measure of shrinkage
Lean	< 2%
Medium fat	2 - 4%
Fat	4 - 6%

Beside the dry measure of shrinkage also the time dependent change of drying is of great importance. As can be seen in table D.4:5, usually 50% of the absorbed amount of water is released again in the first five days. Only with bricks and vertical coring bricks this period is exceeded considerably, somewhat with normal concrete. Further drying occurs only with samples of aerated concrete and expanded clay lightweight clay in a comparable short period of time. These samples dry completely in 28 days at the most. Heavyweight clays and straw lightweight clays lose their rest humidity very slowly, so that the time to the complete dryness reaches within 40-80 days. With brick samples, normal concrete and lime-sand stone the drying curve is linear and flat after 2-5 days, and equilibrium humidity is not reached after 110 days. Wetting periods of one and 24 hours are distinguished only by the amount of loss of water and not by the time factor.

Table D.4:5 Drying time at 23°C, 50% RH after one hour wetting [6]

Material	**Bulk density [kg/m^3]**	**Drying time [days]**	
Drying level		*50%*	*100%*
Clay stone	1900	3	60
Clay stone silty	1950	5	80
Straw light-weight clay	1200	4	60
Straw light-weight clay	550	2	40
Straw light-weight clay	450	3	40
Expanded clay, light-weight clay	750	4	28
Expanded clay, light-weight clay	700	3	28
Brick	1850	13	>110
Vertical coring brick	1200	14	90
Lime-sand brick	1800	5	>110
Aerated concrete Hebel	600	2	21
Aerated concrete Ytong	450	4	28
Normal concrete B25	2200	7	>110

D.4.9 Heat conductivity and heat capacity

The heat conductivity λ and the heat capacity c are dependent on the dry-density ρ. Clay without light aggregates shows good heat accumulation properties. The heat conductivity is in this case relatively high. The heat resistance can be improved by adding lightweight aggregates, lowering the λ-value of clay from 0,9 down to 0,1. The heat capacity of massive clay building parts is of the same magnitude as those of bricks and concrete (see table D.4:6).

Table D.4:6 The relation between the bulk density and the heat parameters of clays [9]; comparison to other building materials [8]

Material	ρ [kg/m³]	λ [W/mK]	c [kJ/kgK]
Heavy-weight clay	2000	0,95	1,0
Light-weight clay	1200	0,47	1,0
”	900	0,26	1,1
”	600	0,15	1,2
”	300	0,09	1,3
Brick	1100	0,37	0,9
Insulating brick	1200	0,47	0,9
Lime-sand brick	1600	0,80	0,9
Gas-concrete stone	600	0,18	1,1
Wood (spruce, fir)	450-500	0,14	2,0-2,4
Wood-wool board	350-500	0,09	1,6

ρ — bulk density, λ — heat conductivity, c — specific heat capacity

D.4.10 Thermal expansion

The temperature dependent changes in length are of importance in clay renderings. To avoid damages the coefficients of expansion of the rendering and the support should be the same if possible. For clay building parts and other materials the values in table D.4:7 were found experimentally [9].

Table D.4:7 Thermal expansion coefficient for building materials

Material	Thermal expansion [mm/mK]
Heavy-weight clay	0,0043 - 0,0052
Clay masonry	0,0062
Lean clay mortar	0,007
Lime mortar	0,005
Cement mortar	0,01
Concrete, gas concrete	0,01
Lime-sand stone	0,007
Resin plaster	0,13 - 0,3

D.4.11 Sound insulation

The sound insulation property of clay building materials depends on their dry-density (air-borne sound) and their composition, especially their elastic additives (structure-borne or impact sound), respectively.

Good air-borne sound insulation values have clays with a bulk density between 1000-2000 kg/m^3. The impact sound insulation value of clay without any additions is much better than of other harder building materials of the same bulk density, because of the elasticity of the clay minerals.

D.4.12 Fire resistance

Clay is fire resistant even in the case of minor addition of plant fibres like straw as long as a density of 1700 kg/m^3 is not exceeded.

D.5 FIELD TESTS FOR BUILDING CLAYS

D.5.1 Appearance, smell

In principle building clay has to be free of humus, roots and other organic constituents. The ionic balance of the clay minerals could be influenced negatively by processes of fermentation, which develop from the decay of organic constituents.

The colour gives a hint on the kind of clay minerals. The smell, particularly when freshly dug, helps to recognise organic compounds.

D.5.2 Sedimentation test

By shaking clay in a glass with much water (1:3), the gravel and the coarse sand fraction settle down quickly at the bottom, while the silt- and clay particles need much more time. From the thickness of the sedimentation layers the fine and coarse fraction of the clay can be estimated roughly. Usually several layers are visible. The organic parts float at the water surface.

D.5.3 Nibble test

This is an easy way of assessing the presence of sand, silt or clay: take a pitch of earth and chew it lightly between the teeth:

- Sandy soil: Hard sand particles feel disagreeably gritty between the teeth, even if the sand is very fine.
- Silty soil: The silt particles are much smaller than those of sand and although they still feel gritty, the sensation is not disagreeable. Silt is a lot less gritty than sand.
- Clay soil: the clay particles are not at all gritty. On the contrary, clay feels smooth and floury between the teeth. A small lump of dry earth containing a lot of clay will stick to the tongue and be difficult to work free.

D.5.4 Sticking test — rubbing and wash test

The clay sample is wetted with water and kneaded. Then the wetted clay is placed in the palm of one's hand and left to dry, after which the clay is scraped off. Depending on the behaviour of the clay during this process, sandy, fat, or silty building clay may be distinguished.

If the clay only sticks somewhat to the hand (easy to get rid of the dried clay by slight rubbing and the rest with a small amount of water), the clay has a high content of silt. If the clay remains firmly attached to the hand (impossible to get rid of by rubbing and with water a lot of time is needed), this clay has a high content of clay minerals.

D.5.5 Shaking test

Like the sticking test this test is used to distinguish between building clays that are sandy, silty and fat in texture. The sample should be earth moist. The clay sample is shaken to and from in the palm of the hand and pressed between the balls of the thumb. Depending on the composition the surface of the clay thereafter looks more or less shiny. A dull surface is characteristic of fat clay.

D.5.6 Kneading test / consistency test

This method is used to distinguish between fat, medium fat and lean building clays. The sample, which should be earth moist, is kneaded and rolled on a porous support to a small spindle of about 3 cm in diameter. By kneading and rolling over and over again, more and more water is extracted from the sample. Eventually the clay spindle crumbles. At this state the stiffness of the sample is proved by pressing together the sample between the fingers. The greater the stiffness, the greater is the content of the clay fraction.

D.5.7 Ball test

This is another method to distinguish between fat, medium fat and lean building clays. The sample should be plastic. A ball, about 5 cm in diameter, is formed and dried. The dry ball is brought to fall down on a hard base from a height of about one meter. If the building clay is fat the ball stays complete.

D.5.8 Cutting test

A clay ball made of wet building clay is cut with a knife in two parts. If the cross section surface looks dull to moderate shiny the building clay is silty, while a very shiny surface indicates a clay with a high content of clay minerals.

D.5.9 Hydrochloric acid test

Lime containing clays usually have a whitish appearance, have a low tenacity and are for this reasons unsuitable for clay constructions.

To determine the lime content, a droplet of 20% hydrochloric acid is placed on the sample. The development of carbon dioxide leads to foaming and bubbling. If no foaming is visible the lime content is lower than 1%. If weak and quickly decaying foaming is evident the lime content is about 1-2%; whereas the lime content is 3-4%, if clear and lasting foaming is present, and 5% with evidence of an intense and lengthy foaming action. Also, dark lime-free clays rich in humus can show a carbon dioxide development.

D.6 REQUIREMENTS ON CLAY BUILDING TYPES

D.6.1 Construction with sun dried earth blocks, adobes

An adobe construction is a solid wall made by sun dried bricks or blocks of clay. The material used is a medium fat to fat building clay with at most a small portion of light-weight aggregates, by which high compressive strength and heat capacity are reached. The shape of adobes is in principle free of choice. The form of the adobes has to be 3-5% greater than the intended final size because of the great dry shrinkage.

There are three methods to produce sun dried earth blocks:

Adobes extruded:
This material stems usually from brickworks as the preliminary stage to the burned brick. Here the plastic clay passes an extrusion press and is cut in blocks on leaving it.

The composition of the material and the manufacturing method are usually those chosen for making a burned product and not an unburned adobe. However, when using such adobes for an exterior wall caution is advised. To produce adobes by extrusion, manufacturing methods that are specially designed for producing unburned adobes is instead recommended, e.g. no compression in vacuum (a greater amount of pores which gives a stronger frost resistance and a better heat insulation).

Adobes pressed:
This kind of clay building material is formed in small hand or motor operated presses using earth moist clay as raw material. Gravels greater than 10 mm are sorted out. The building stone produced in this way are nearly non-plastic and can immediately be stacked to dry out.

Adobes filled in forms:
The plastic building clay is put in a form manually and smoothened, and then immediately taken out. The formed clay has to start drying before it is stacked to dry out completely.

The fat or medium fat building clay for the adobes is shortened with sand. Often organic additions are added in small amounts (chopped straw, chaff, saw dust, etc.). An addition of a greater amount of mineral or organic lightweight building materials leads to a lightweight adobe that cannot be used for supporting constructions.

Additives like cement or bitumen can be mixed in if an increased strength or reduced water sensitivity is required. These measures alter as mentioned also other physical properties related to construction, like heat and humidity transport, and therefore are not always advisable.

For hand made adobes the following specific requirements should be fulfilled [8]:

- Clays medium fat to fat
- No particles greater than 5 mm
- Test bricks of the desired size, prepared with the suitable mixture, should not warp after drying or show shrinkage cracks greater than 50 mm and/or 3 mm width
- Compressive strength not less than 2 N/mm^2.

During manufacturing the produced adobes have to be controlled regularly.

D.6.2 Rammed earth construction

”Rammed earth” constructions are made by ramming clay into shuttering. The building clay used for this work is normally rather lean and earth moist. With this construction type, large building units could be build up which form a monolithic structure after drying, with a high heat capacity and a sufficient compressive strength for supporting walls.

The natural occurring lean clay needs no shortening for rammed earth constructions. Mineral additions are merely added to fat building clays or if the coarse grain fraction is missing. Then the dry shrinkage rate is diminished to a tolerable amount and the stability of the building unit is increased. Non-mineral additions are not tested in praxis of the rammed earth constructions and are not recommended.

For rammed earth constructions the following specific requirements should be fulfilled [8]:

- Clays lean to medium fat
- Grading curve with a good mixture of different grain sizes until 1/8 of the wall thickness or 50 mm
- Prepared clay mixture earth moisturised
- Preparation of cubes of $20\times20\times20$ cm^3 of the specific mixture for testing (see below)
- Minimum compressive strength 2 N/mm^2; testing of the samples has to be in the direction of ramming
- Shrinkage at drying not greater than 2%.

The clay building material has to be controlled regularly during the construction.

D.6.3 Lightweight building clay as fillings

In this technique the lightweight clay is used as a non-supporting filling material. The loads have to be born by an independent bearing structure.

Lightweight building clay is a mixture of clay and lightweight additions that lower the density of the clay, improve the heat insulation properties and reduce the measure of shrinkage. For this reason exterior walls made of lightweight clays could be erected without any additional heat insulation. The rough surface of the dried clay mixture is a non-slipping support, suitable for rendering.

As organic additions are suitable, e.g. straw, reed, wood chaff, wood-wool and cork-meal. As mineral additions expanded clay, perlite and similar materials are possible. If straw is used as addition, attention should be paid to the length of the straw that should correspond to the width of the building unit. The straw should be stable and tear-resistant. The best choice is rye and wheat straw.

For lightweight clay constructions the following requirements [8] should be fulfilled:

- The get a sufficient tenacity to bind all additives the clay has to be medium fat to fat
- Tenacity proved by hand tests
- Stability, drying behaviour, compressive strength and bulk density tested on suitable specimens, corresponding to the intended construction unit
- Bulk density below 400 kg/m^3 is not recommended for construction technique reasons
- Compressive strength, not less than 0,5 N/mm^2, proved by test samples
- Gravel and sand with grain sizes greater than 2 mm sorted out to get a low bulk density.

The clay portion of a lightweight clay mixture is governed by the tenacity and the bulk density required, in its turn depending on the target value of the heat conductivity of the construction.

D.6.4 Clay mortar and clay plaster

To get a mortar or a plaster the building clay is shortened with sand or other aggregates. Clay plasters can be reinforced by suitable additions. For clay mortars and clay plasters the following specific requirements should be fulfilled [8]:

- Fat or medium fat building clay
- Balanced grading curve, i.e. all grain sizes should be present; no grains greater than 5 mm.
- Not too fat mortar to get good working properties and to avoid shrinkage cracks; very lean mixtures have low tenacity
- To examine the clay mortar or plaster, test mixtures should be applied to wall elements
- Clay mortars shall have a strong connection to the adobes
- Clay plasters have to be abrasive resistant, crack formation is not tolerable.

In addition a great number of possibilities exist to work with clay as a building material, not presented in this paper. Please refer to the specialised literature [9, 10].

D.7 IMPROVEMENT OF MATERIAL PROPERTIES

The improvements of the material properties of building clays are necessary only in special cases. In many cases some clay properties can be improved by suitable additions and additives, but possible, negative side effects has to be checked always.

D.7.1 Reduction of crack formation on drying

The formation of cracks during the drying of a clay plaster should not be accepted. The most important factors in this respect are the water content, the grading curve, the clay content and the type of clay used. Reducing the water content is the easiest way to reduce the tendency to crack. Because there is a lower limit in the water content — the minimum processing water content — other possibilities have to be considered.

A further possibility is the increase of the non-clay part, the shortening of the building clay. The higher this content, the less is the shrinkage. Especially the addition of fibrous materials leads to a reduction of the linear dry measure of shrinkage. The reason for this is the lower clay content as well as the fibre property to fix a certain amount of the mixing water.

The addition of very small fibres increases the tensile strength of the clay in the plastic state. The crack formation is reduced in general. The formation of large cracks is blocked at the expense of the formation of more small cracks at the same place.

Constructive measures could be used all the time by choosing the right drying conditions for the clay, e.g. to allow the raw adobes a slow and constant contraction. Raw adobes shall not be put in the sun and have just very little contact to the support.

D.7.2 Improving the water resistance

Only clay building parts and clay plasters that are exposed immediately to weathering have to be protected against the influence of water. In many cases a waterproof coating is enough. But if the surface is damaged, especially with clay plasters, the water input leads to severe damage by swelling and frost action. Waterproof clay building parts allow construction works in humid environmental conditions.

A rule is that cement and bitumen are suitable for rather sandy building clays while lime is suitable for a building clay rich in clay minerals. Furthermore, it is to consider that kaolinite rich clays do not behave exactly as montmorillonite rich clays do.

According to [9], the additives for stabilisation enclose the constituents of a clay, thus hindering the water to penetrate and consequently also the swelling and softening of the clay.

In addition to that, and for many centuries, animal products as blood, urine, excrement, casein, bone glue and other stabilising additives have been used to increase the weather resistance of a clay surface. If applied in the right manner, this effect is confirmed [9].

D.7.3 Improving the tenacity

The higher the tenacity of a building clay, the greater is the compressive and abrasive strength in the dry state. No special requirements concerning the tenacity are made on building clays. If the tenacity is insufficient it can be raised by the addition of fat clay or pure clay minerals but also by a better processing, e.g. by kneading and souring. Just a prolonged stirring and kneading of a clay, say ten instead of one minute, leads to an improved tenacity by 50%.

Also the mineral, animal and plant additions, which were added to increase the weather resistance, change the tenacity — in most cases positively, but in some cases negatively. The suitability and necessity of such additions and additives have to be proved for every single case.

D.7.4 Improving the dry-compressive strength

The compressive strength of clay depends on the grading curve, the water content and the static or dynamic compression during the processing of the clay building part, as well as on the type of the clay minerals. If the sand has a grading curve corresponding to a highly dense packing with a minimum of pore space, that is the clay and silt content is just enough to fill the pore space between the sand grains, then the greatest density is reached which in most cases coincides with the greatest compressive strength.

A higher compressive strength for adobes is needed only to increase the edge strength, which is important looking to the transport, stacking and the walling up. Damages of the edges of the adobes occur not so easy.

The optimum grading curve corresponds nearly the so called "Fuller-parabolic curves", as used in the determination of the additions to concrete. They should be applied to clay only for the grain fraction greater than 0,002 mm.

During the processing of building clay the hand-made adobes often have the same strength as the ones compressed with mechanical presses, because the latter not always end up with the best orientation of the clay particles.

When compressing clay, ramming by beating or vibrating is more effective than a static pressure.

As for clay constructions in general, not the maximum dry-density and thus the optimum water content according to the "Procter curve" but the processibility and the tenacity are decisive. Therefore in most cases a higher amount of water should be used. For the manufacturing of greater adobe blocks a value of the "best value" after the Procter test plus 10% has approved.

The kind of clay minerals has the effect that, e.g. for kaolinitic adobes, a higher compressive pressure leads to a 50% higher compressive strength, while at the same conditions a montmorillonitic clay result in a 100% increase. An addition of montmorillonite to lean clay made its compressive strength higher. Also the mineral addition like lime and cement mentioned above, which contribute to an increase in the water resistance, make the compressive strength higher.

D.7.5 Improving the abrasion resistance

The abrasive resistance of clay surfaces can be improved by many additives like sodium water glass, bone glue, cottage cheese, lime, paraffin, floor polishing and linseed oil varnish. Furthermore, after traditional recipes a clay floor can get a very durable, abrasive resistant surface by painting it with bull's blood, strew Fe_2O_3 over it and hammer this cover in the clay. In the past also other materials as tar and bull's gall were used [9].

D.7.6 Increasing the heat insulation

The heat insulation properties of building clay are improved by addition of porous materials. In use are plant parts as straw or coarse cork and natural and artificial expanded mineral products as pumice, expanded clay, expanded glass, etc. The greater the amount of pores, the lighter the mixture and the higher is the heat insulation effect.

It is much more difficult to build up a wall with a high insulation factor using straw light-weight clay than with mineral products, because the plant parts have higher equilibrium humidity content, and furthermore, they tend to mould. Good heat insulation and a high material strength can only be reached by mineral products.

D.8 WEATHER PROTECTION OF CLAY SURFACES

D.8.1 Constructive measures

As being the most durable and difficult to subsequently modify, first of all and from the very beginning constructive solutions should be sought before taking further measures against the weather. To the constructive measures all constructional actions are counted which prevent (rain)-water from reaching the external surface of a building, i.e. roof overhang.

D.8.2 Compacting the surface

By compacting and smoothing the surface it will be enriched by clay particles, leading to a surface which is much more resistant to rain than otherwise.

D.8.3 Coatings

External coatings should be water repellent (see table D.8:1) and have an open porous structure at the same time, i.e. the water vapour diffusion should scarcely be influenced. This is required to facilitate moisture, soaked by the wall during driving rain, to escape outwards. Appropriate products can be used but not in half-timbered houses, because compartment fillings made of clay usually have a lime plaster.

For internal walls any kind of coating could be used. Since building clay surfaces are very absorptive, water-soluble colours stick especially good.

Many historic coatings are made at a lime base. This type can be used as pure lime coating or together with casein, borax and other organic additives and shows good properties.

The common glue-water colours and limewater colours are suitable only for interior surfaces.

Table D.8:1 Uptake of water coefficients w of clay plasters with surface treatment [9]

Coating	Amount used [g/m^2]	w-value [kg/m^{2}h^{½}]
None	0	9,5
Boiled linseed oil	400	0,0
Lime-casein 1:1	420/350	0,6/1,5
Lime-casein 1:8	300/300	0,7
Silin-paint (van Baerle)	700/250/310	0,3
Hydrophobic (Herbol)	390/390	0,0
Baysoline LD (Bayer)	400/290	0,2
Syltrit (Metropark)	350/320	0,0
BS 15 (Wacker)	450/430	0,1
Steinfestiger H (Wacker)	290/290	0,0

D.8.4 Hydrophobing

Hydrophobing agents are colourless, in organic solvents or water dissolved products, which penetrate a building material in the sense of an impregnation medium to give water-repellent properties. The hydrophobing agents reduce strongly the capillary absorbency while the water vapour diffusion is only slightly diminished. In most cases they are silanes, siloxanes and siliconates. Hydrophobing agents in common organic solvents usually yield a good result with a sufficient penetration depth. The penetration depth could be a problem with very fat clays.

The application of this product can be recommended only in very special cases, because they do not allow capillary water transport. Like coatings they should not be used with half-timbered houses. In addition hydrophobing agents are very expensive.

D.8.5 Plasters

For external plastering on unshielded windward sides no clay plaster but a lime plaster should be used. It should be applied on a key reinforced with fibres (straw, etc.) [11]. Cement plasters show usually a too high strength and are not suitable. Plasters should have a low relative water vapour resistivity μ (see table D.8:3) to allow the possible entering moisture to dry out. According to [12] plasters should be up to the standards as given in table D.8:2.

Table D.8:2 Requirements of plasters on clay masonry [9]

Parameter	Limit value
Consistency/slump	17 ± 0,5 cm
Water binding ability	> 90%
Bonding strength β_{HZ}	$\geq$ 0,05 N/mm^2
Relative water vapour resistivity μ	$\leq$ 12
Uptake of water coefficient w	$\leq$ 0,5 kg/m^{2}h^{½}
Compressive strength β_D	3 - 5 N/mm^2
Tensile bending strength β_{BZ}	1 - 1,5 N/mm^2
Tensile strength β_Z	$\geq$ 0,5 N/mm^2 resp. according to the E-modulus
Modulus of elasticity E_{dyn}	$\leq$ 8000 N/mm^2
Shrinkage ε_s	$\leq$ 0,3 mm/m
Density	$\leq$ 2,0 g/cm^3

Table D.8:3 Relative water vapour resistivity μ of lime plasters with additives [9] (plaster composition in parts by volume)

Lime powder	Trass	Screed sand	Skimmed cottage cheese	Boiled linseed oil	Fat clay	Cow dung	μ-value
1		3					11,2
	1	3					10,8
1		6	0,5				6.2
1		15	0,5		3		9,7
1		3		0,05			15,2
1		3	0,25	0,05			28,5
1,5		10			2	6	8,0

D.8.6 Facing, cladding

In addition to coatings and plasters as weather protection for clay wall constructions, linings and covers made of boards, shingles and faced masonry made of burned bricks are used. These solutions are obvious when additional heat insulation is required.

D.9 DURABILITY AND SERVICE LIFE

Clay and clay products for construction work are very sensitive to water, particularly running water. For this reason all the measures which prevent direct contact of a clay construction to water, as described in the previous chapters, are very important to prolong the service life and to enlarge the durability of a clay construction. The service life of clay constructions can be hundreds of years if carefully protected against the attack of water. All other environmental attack is insignificant.

If clay constructions are maintained in the best way, the service life is of the same order magnitude as for other inorganic building materials. In contrary to these building materials like concrete, the strength and the durability against erosion and wetting increases with the age of the clay building parts. Although no scientific investigations concerning this topic are available at the moment, this statement is based on the observations of old clay building parts and the reuse and use of earth stones tens of years old [13]. The assumption can be made that a considerable after-strengthening occurs. Old clay stones show hardness and properties like a fine conglomerate that could hardly be separated by tools.

The consolidation of clay parts under pressure (clay petrification) could explain the existence of buildings like the clay pyramids, the Great wall if China and the Ziggurat of Babylon, should not exist at all on the basis of strength calculations of single clay stones.

More research is needed in these fields to better understand the strength of clay building materials and the development with time, to use this knowledge for construction purposes.

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E. Wood (*K. Ődeen*)

E.1 PHYSICAL PROPERTIES

E.1.1 Main parts of the wood log

In a cut of a wood log a number of main areas can be identified as shown in fig E.1:1. Inside the bark is the cambium layer where the main growth takes place. The rest of the cross section contains the sapwood and the central heartwood. In the very centre is the pith and sometimes the so called juvenile wood.

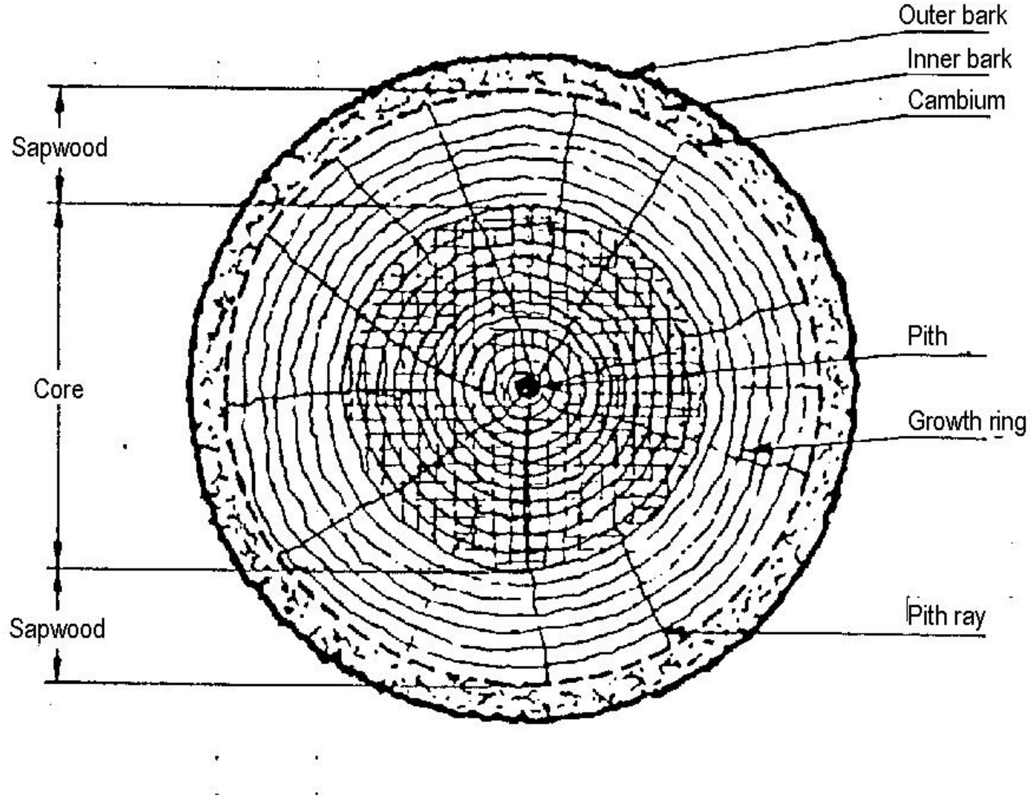

Fig E.1:1 Cross section and main parts of a log

The border between sapwood and heartwood can be clearly seen in some species, e.g. Scotch pine, whereas it can hardly be visually identified in others, e.g. Norwegian spruce. The sapwood contains both living and dead cells, and serves as nutriment store and for fluid transport. The heartwood contains only dead cells and a lot of extractives and resins that decrease the moisture absorption. This in turn influences many important properties, e.g. moisture deformations and durability. In the figure can also be seen the pith radius through which nutriment is transported in the radial direction.

E.1.2 Sawn timber

After felling, the tree is cut into logs of suitable lengths and transported to a saw mill where it is further fragmented. This process can be made according to different principles. A typical cutting pattern is shown in fig E.1:2.

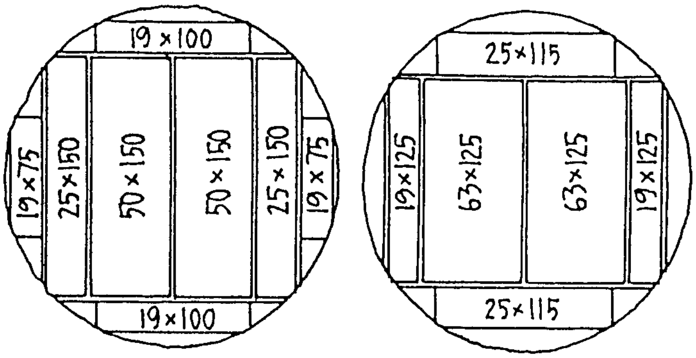

Fig E.1:2 Cutting pattern [1]

E.1.3 Drying

The sawn timber has high moisture content and must be dried before further use. The drying is a very crucial process and there is a high risk of damages of various types, such as cracks and deformations. Mostly timber is dried in industrial processes under controlled conditions. A typical drying program for conifer wood (pine or spruce) takes 3 to 6 days depending on dimensions. Drying of hardwood takes longer time. After drying, the wood is conditioned under rain cover outdoor or in special chambers, allowing the moisture content to be equalised and the stresses relaxed. This part of the process takes 2 to 3 days.

E.1.4 The wood structure

The wood structure varies significantly between wood species. In particular there are large differences between softwood and hardwood.

E.1.4.1 Cell types

The following description refers to conifer wood. Softwood has a slightly different cell structure.

E.1.4.1.1 Tracheids

Tracheids are dead cells and they form about 95% of the wood volume. They are shaped as long tubes with closed ends. The length is 3-5 mm and the thickness 25-45 mm. The strength of the wood is basically constituted by the tracheids, which also are responsible for the water transport in the tree.

E.1.4.1.2 Parenchyma cells

The living cells in the wood are called parenchyma cells. They contain nutritient agents and form 1-2% of the total volume. They are shorter and have thinner walls then the tracheids.

E.1.4.1.3 Resin ducts

The resin ducts are inter-cellular spaces surrounded by specialised parenchyma cells (epithelial cells) producing resin.

E.1.4.1.4 Rays

The radial transport of water and nutritious substances goes from the periphery into the pith via the rays. They are formed by both tracheids and parenchyma cells and may also contain resin ducts. The cavities in the different cells are called *lumen.* An overview of the cell types is given in fig E.1:3.

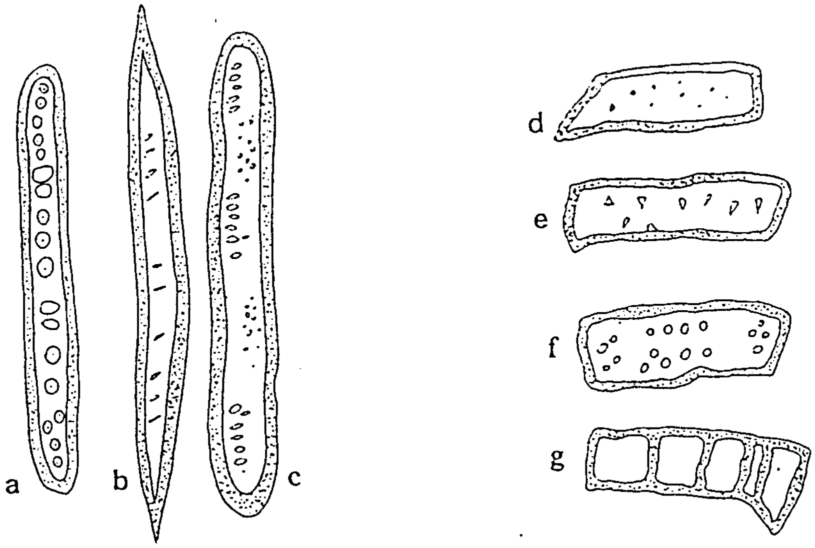

Fig E.1:3 Cell types in softwood: a) pine, earlywood; b) pine, latewood; c) Norwegian spruce, earlywood; d) and e) ray tracheids, spruce and pine, respectively; and f) and g) parenchyma cells, spruce and pine, respectively [2]

E.1.4.2 Pit system

The wood contains pits of different types. The two main types are *simple pits* and *bordered pits*. Simple pits basically exist between parenchyma cells. They are covered by a partly permeable membrane, formed by the middle lamella between the cells.

The bordered pits connect the tracheids and are a lot more complicated, both structural and regarding their function. They are covered by a membrane containing a central, non-permeable part (torus) attached to the surrounding cell wall with an open, fibrous structure (margo). There are also combinations of the two basic pit types (half-bordered pits).

An overview of the pit types is given in fig E.1:4. In the status shown in the figure, the bordered pits are permeable (via margo) and water can be transported between the pits without obstruction. When the wood is drying the capillary water meniscus will draw the torus towards the pit wall and the pit is closed. In this position the torus is fixed by chemical bonds, which makes the process irreversible. The mechanism is called *aspiration*. One effect is that dried wood has other permeability properties then green wood. However, all bordered pits do not aspire and there are large variations between species. For example, in sapwood of *pinus sylvestris* a large amount of the bordered pits are not aspired, whereas in *picea abies* they are. A practical effect of this is that it is a lot more difficult to impregnate the latter wood specie.

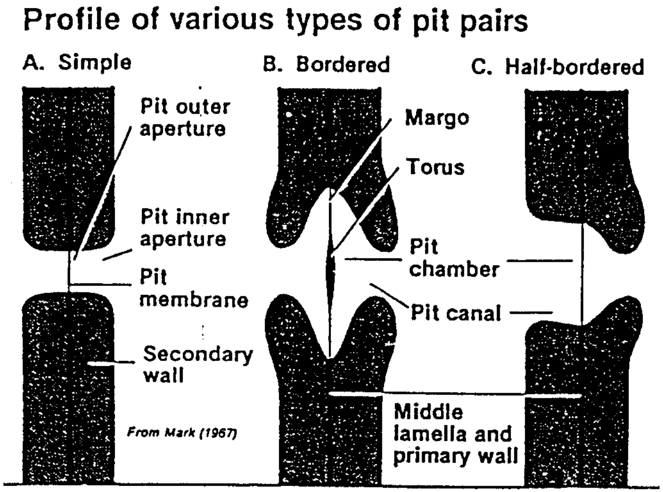

Fig E.1.4 Pit types

E.1.5 Wood properties

E.1.5.1 Density and porosity

The gross density of the wood can be expressed in different ways. As both mass and volume depends on the moisture content, this must be mentioned together with the density. A common method is to give the density at a moisture content of 12% by dry weight. The gross density of Scandinavian species of softwood varies within a wide range both between trees and within a single tree. Approximate values are 400-500 kg/m^3 for *pinus sylvestris* and 370-440 kg/m^3 for *picea abies*.

The compact density for all wood species is approximately 1500 kg/m^3. The density and its variation has large impact on most properties relevant to, for example, the end-use. These are:

- strength
- modulus of elasticity
- moisture deformations
- cracking
- surface treatability

impregnatability

E.1.5.2 Moisture properties

Wood is a highly hygroscopic material and most essential properties vary strongly with the moisture content. Water may be bound in the cell walls, or appears as free water in the cell cavities or as water vapour.

The moisture content is usually expressed relative the dry weight of the wood – the moisture ratio. It can be determined directly by weighing and drying at 103-105°C or by some indirect method. A well established method of this type is to measure the moisture dependent electrical resistance between two nails driven into the wood.

The properties of wood are mainly influenced by the water bound in the cell walls up to the so called fibre saturation point. For Scandinavian softwood this is in the region of 25-30%.

E.1.5.2.1 Moisture fixation

The relation between surrounding climate and moisture ratio at equilibrium is given by sorption isotherms. A typical example is shown in fig E.1:5. The curves have been determined at 20°C. The influence of the temperature may be significant.

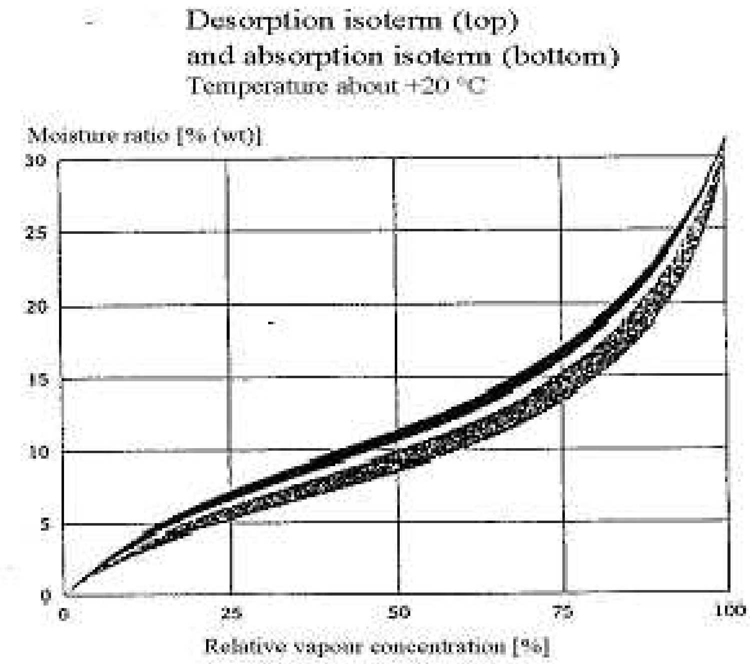

Fig E.1:5 Sorption curves for wood [3]

E.1.5.2.2 Moisture transport

The moisture is transported in wood both as liquid and as vapour (diffusion). The capillary transport in liquid phase takes place in the cell cavities (lumen) and through the pits, whereas the diffusion also penetrates the cell walls. The diffusion is a much slower process than the capillary transport. It takes about one week for a 25 mm thick panel to reach equilibrium with the surrounding climate. The dependence on thickness is approximately quadratic so the corresponding time for a 50 mm thick panel is one month.

The rate of capillary transport in the longitudinal direction is 10-20 times higher than in the transverse direction. Therefore, it is very important that the ends of wood panels or other components are protected from contact with liquid water by constructive measures or by suitable treatment.

The moisture transport by diffusion can be characterised by the moisture diffusivity. It is strongly dependent on the moisture content.

E.1.5.2.3 Moisture induced deformations

Above fibre saturation, hardly no deformations occur when the moisture content is changed. However, below fibre saturation the wood swells and shrinks when the moisture content is varied. The deformations are highly different in different directions. For Scandinavian softwood the following deformations values for fibre saturated wood relative completely dried wood are typical:

Direction	%
Tangential	8,0
Radial	4,0
Longitudinal	0,3
Volume	12,0

The difference between the tangential and radial deformations causes a panel to deform according to fig E.1:6 when its moisture ratio is changed.

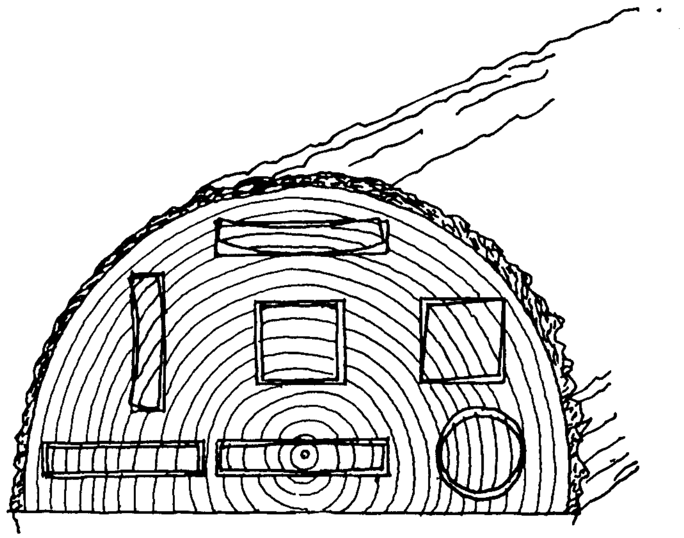

Fig E.1:6 Deformation of a panel during drying [1]

E.1.5.3 Strength and elasticity

As wood is a highly anisotropic material, its strength and deformation properties vary significantly in different directions. They are also influenced by test conditions, e.g. temperature, moisture and rate of load application.

E.1.5.3.1 Stress – deformation curves

Typical stress – deformation curves for wood loaded parallel to the grains are shown in fig E.1:7. It can be seen that the tension strength is roughly twice the strength at compression. At tension the curve is nearly linear up to failure. At compression the curve shows a horizontal part, where the deformation increases at constant stress. The reason is that the shear strength is reached and there are formed two distinct parts and an inclined "slip surface" will develop, see fig E.1:8.

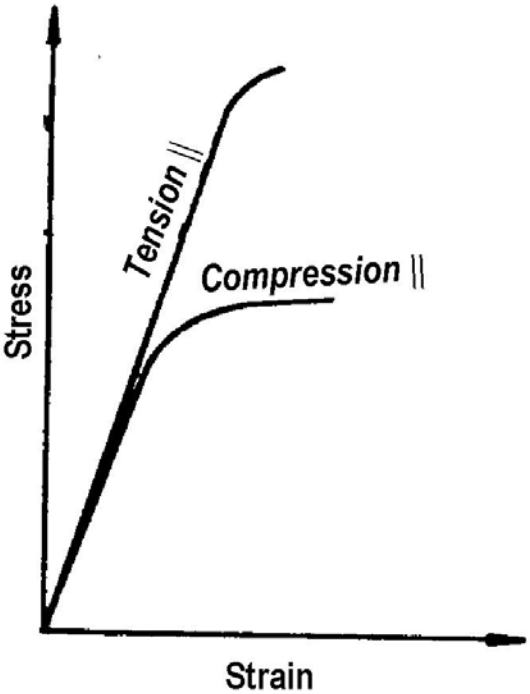

Fig E.1:7 Stress – deformation curves for wood

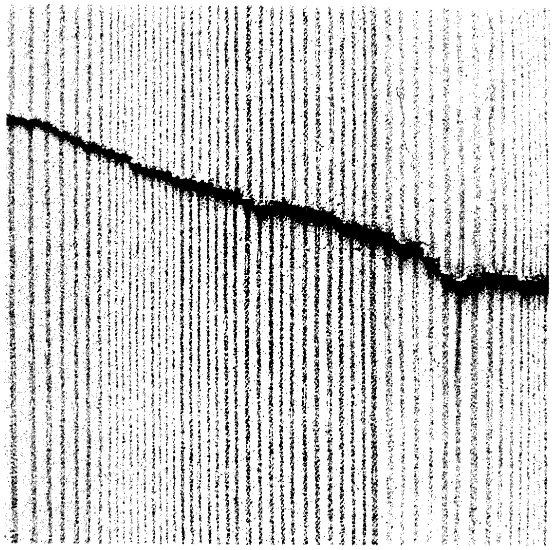

Fig E.1:8 Detail of compression failure in wood [1]

The strength varies strongly with the angle between the grain direction and load, the strength perpendicular to the grains being only about 10% of the strength parallel to the grains. In fig E.1:9 is shown typical curves for the strength of *pinus sylvestris* at different loading modes as well as the influence of moisture ratio. It can be seen from the figure that the strength decreases with increasing moisture ratio up to fibre saturation and then remains constant.

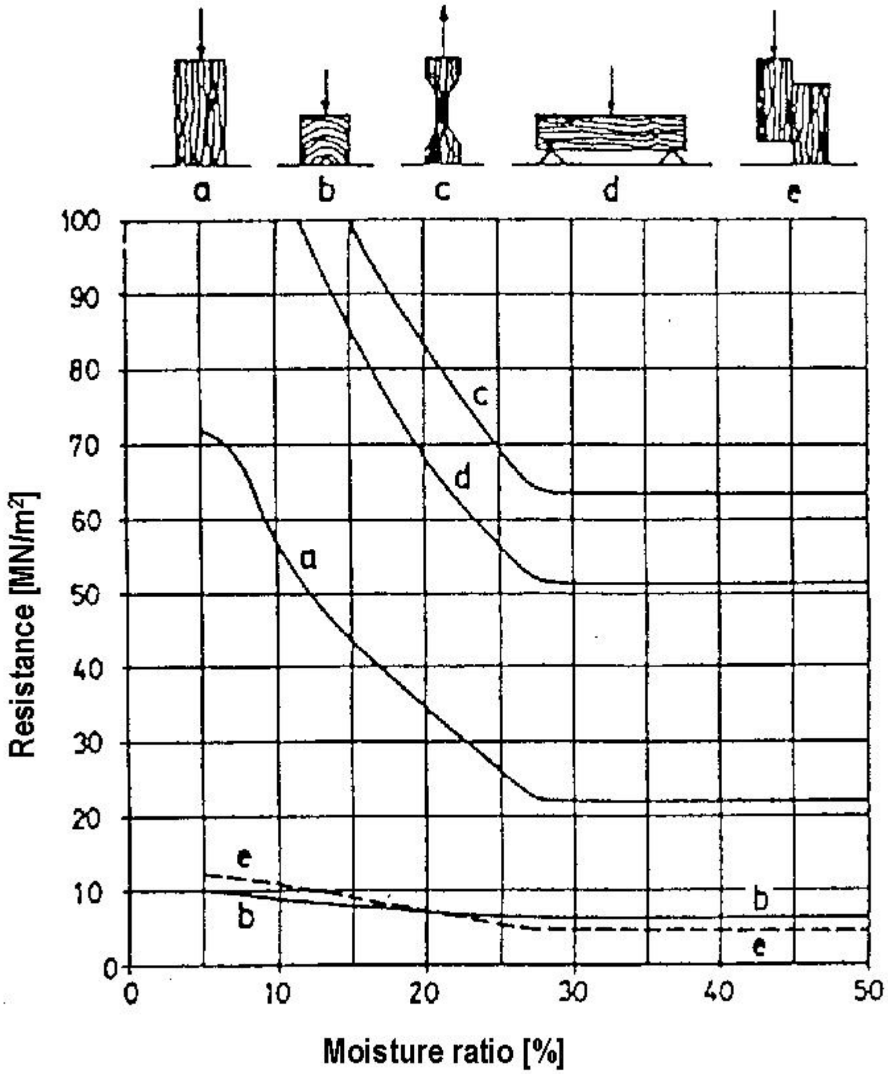

Fig E.1:9 Influence of the moisture ratio and load direction on the strength of wood [4]

a = compression // grains
b = compression ⊥ grains
c = tension // grains
d = bending ⊥ grains
e = shear // grains

E.1.5.3.2 Modulus of elasticity (Young´s modulus)

The slope of the linear parts of the stress – deformation curves is approximately the same for tension and compression that means that the modulii of elasticity are the same. For Scandinavian softwood it is in the region 11-12 GPa parallel to the grains and 4,5-5,5 GPa at perpendicular loading. It should be underlined that characteristic values of strength and elasticity taking the statistical variations into account may be significantly lower than indicated above.

E.1.5.4 Rheology (creep)

The creep deformations of wood must generally be taken into account. The visco-elastic behaviour is important both for load-bearing structures and for the development of deformations and cracks when the moisture content is changed. The creep number (creep deformation vs. short time elastic deformation) after one year is of the order of magnitude of 1 -1,5 parallel to the grains and at constant moisture content.

The creep deformations increase with increasing moisture content. In special the increase is dramatic when the moisture content is changed during the loading. One example of this is shown in fig E.1:10. The lower curve is determined on test specimens at constant moisture conditions, whereas the upper curves holds for test specimens where the moisture has been chanced cyclically. It can be seen that the deformations in the latter cases are much higher than when the moisture is constant. The phenomenon is explained by so-called mechano-sorption.

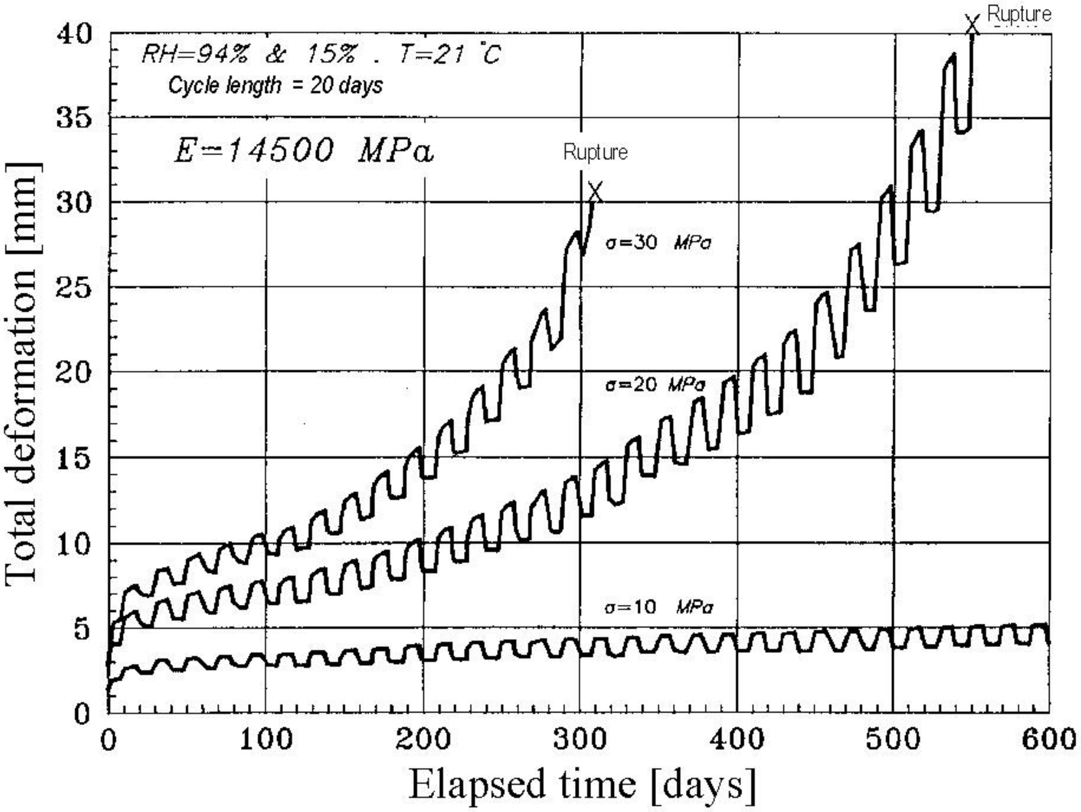

Fig E.1:10 Creep deformations of wood at different moisture ratios [5]

E.1.6 Internal factors of inherent solid wood properties

Wood is generally considered to be more resistant to deterioration by micro-organisms than most other plant tissue. The natural resistance of wood to the biological agents is due to the presence of toxic substances in the wood. Other reasons include that the woody cell wall consists of highly complex, insoluble polymers of high molecular weight. These substances must be depolymerised by enzymes, produced by the attacking organisms, into simpler products that can be assimilated by them. The action of depolymerising enzymes is restricted mainly to the non-crystalline region of the cellulose. As the cellulose in wood is more crystalline than that of most other plant tissues, it provides greater resistance to fungal and bacterial degradation. Higher moisture ratio is required to initiate deterioration in wood than in other plant tissues. For instance, in cotton fibres, 10 percent is adequate to initiate degradation by micro-organisms, while nearly no decay can be initiated in wood below the fibre saturation point, i.e. 26 to 30 percent [6].

Besides the above mentioned characteristics with wood, many other inherent wood properties have a close correlation with the durability of the material. Some of the important properties are density, uniformity of growth rings, fibre length, percent of clear bole, straightness of grain, proportion of heartwood, percentage of vessels (in hardwoods), extractives, chemical composition, and presence of juvenile and reaction woods, etc.

E.1.7 Factors associated with wood growth characteristics

E.1.7.1 Density

Many heavy woods are highly durable. This fact may suggest that the wood density is often used as a criterion of decay resistance. That this does not necessarily prevail all the time is indicated by the fact that a number of light woods are among the most durable woods. On the other hand, the heartwood of the relatively heavy woods, such as beech, oak, and maple, is among the least decay-resistant woods. This is because there is no significant difference in the decay resistance of the extractive-free wood substance, irrespective of the species. The superior durability of some woods, including those of light density, is traceable directly to the presence of toxic ingredients in sufficient quantities to inhibit deterioration. Unless the higher durability is correlated with the greater accumulation of toxic substances, there is no convincing and conclusive evidence that variations in density within species or a given tree have much effect on wood durability.

Although wood density has no direct relationship with wood durability, in practice density of wood is still used as one of prime determinants of wood quality (or durability). This is understandably dependent upon the fact that the more dense a timber is, the more substance there is present to be decayed, and for this reason it may endure longer than a more porous one.

Forest growth rate can significantly affect density. This is particularly true for ring- and semi-ring porous hardwoods that tend to show increased density and hardness with increased growth rate. It is commonly believed that softwoods decrease in density with an increase in growth rate despite some studies published reject this notion [7,8]. For hardwoods, regardless of growth rate, the width of the earlywood portion of the ring tends to remain relatively constant.

E.1.7.2 Sapwood and heartwood

The sapwood of nearly all wood species is readily decayed by fungi, and it is only the heartwood of the durable species that is resistant to decay. The susceptibility of sapwood to deterioration by biological agents, besides the fact that it lacks extractives in sufficient quantity or toxicity to inhibit the growth of micro-organisms, is also dependent on that the presence of reserve foods in the parenchyma cells of sapwood may increase its susceptibility to decay, and particularly to bacteria and staining fungi.

The greater durability of heartwood, in comparison with sapwood of the same species, is attributable largely to the presence in the former of a wide variety of toxic extractives such as essential oils, tannins, and phenolic substances. When these are present in sufficient amounts, they prevent or at least considerably minimise the severity of the attack by destructive organisms. This resistance ability of heartwood attributed to the presence of extractives toxic to decay fungi, is often referred as the natural durability of wood. The effect of natural durability on prolonging the service life of wood materials has been demonstrated unequivocally.

Other factors that may explain the greater durability of heartwood includes its lower moisture content, its lower diffusivity, and the blocking of cell cavities by gums, resins and tyloses in the vessels, and tylosoids in the resin canals. Any of these might conceivably and adversely affect the balance between air and water necessary for the growth of fungi.

It is very clear that as the heartwood is more durable, the proportion of heartwood being important where wood materials are exposed outdoors. The lack of well-developed heartwood, for example, in species such as walnut and cherry, would negatively affect the durability. It is also fairly obvious that, even in a wood specie with relatively durable heartwood, the service life of an untreated wood component is determined to relatively large extent by the amount of sapwood present. The disintegration of the less-durable sapwood, especially if it completely surrounds the heartwood, as in poles, may result in functional failures, even though the heartwood remains unaffected. On the basis of many research studies conducted in Sweden and other European countries, the commercially important Swedish timber species for buildings, e. g. pine and spruce, belong to the classes of moderately resistant and slightly or non-resistant species.

E.1.7.3 Lignification

Lignification was the important development in higher plant evolution that provided stiffness to stem tissues, facilitating wood stem aerial growth and protection against destruction by micro-organisms. Lignin type and amount have significant influence on the rate and types of decay found in wood. Lignin contents often vary widely between wood species and within individual portions of the same tree, rendering some portions of the wood, such as the primary cell wall and middle lamella, more resistant to microbial attack. Hardwood lignin is composed of both syringyl and guaiacyl lignin, whereas coniferous lignin only contains guaiacyl residues. Hardwood lignin also differs from coniferous lignin in the types of linkages between the phenyl-propane units.

Wood lignification creates a physical barrier to enzymatic attack on the polysaccharides. Therefore only those organisms which possess enzymes capable of destroying the lignin, or at least of altering its protective association with the polysaccharides, are capable of decaying wood.

Many studies have noted that the resistance of softwoods to soft-rot attack is decreased by partial delignification [9]. Lignin type , content, and pattern of deposition appear to play critical roles in the inception of soft-rot attack. Minor changes in lignin content can produce large decreases in decay resistance. These effects suggest that the pattern of lignin deposition in wood cell wall may be as critical as lignin type in determining natural decay resistance to certain fungi.

E.1.7.4 Uniformity of growth rings

Uniformity of growth rate has an effect upon wood structure and density variation both within and between growth rings. It is indicated that lack of uniformity represents one of the greatest wood quality problems facing wood-using industries[10-12]. Wood characterised by significant within-ring density variation (i.e. by bands of very dense latewood and alternate zones of low-density earlywood) can present a problem when painted and exposed to the elements. The paint film tends to flake off after a period of weathering. The very dense wood shrinks and swells more than the low density wood, thus causing a relative movement between the film and wood surface. Such wood is also difficult to machine to a smooth condition because of the different hardness between early- and latewood bands. The uniformity of wood structure both within and between growth rings is determined to a great extent by growth rate and conditions under which growth occurs. Silvicultural treatments can affect the uniformity as well as density of wood.

E.1.7.5 Straightness of grain

Grain orientation that is not parallel to the long axis of a stem often results in slope of grain in manufactured products. This can drastically reduce strength. This kind of grain orientation also adversely affects machining properties and the nature of moisture-induced dimensional changes. Spiral grain and other kinds of non-normal grain orientation are thus to be avoided if possible. A number of investigators have noted an apparent connection between development of spiral grain and growth conditions. It appears that intensive culture practices, and irrigation in particular, tend to reduce the occurrence of spiral grain [13].

E.1.7.6 Knots

From the standpoint of saw-timber and veneer production, size and frequency of knots is perhaps the single most important aspect of wood quality. Knots greatly affect both appearance and strength, and because of this their presence is a primary factor in determination of log and lumber grades. A number of silvicultural practices can influence knot development, including spacing at time of planting, timing of thinning, treatments to accelerate rate of growth, and pruning.

E.1.7.7 Occurrence of juvenile and reaction woods

Juvenile wood tends to be low in cellulose and high in lignin compared to mature wood. Acceleration of growth early in a rotation increases the proportion of juvenile wood and tends to cause a reduction in cellulose. Numerous investigations about the effect of growth rate on mature wood have yielded conflicting results.

Acceleration of growth can affect the properties of juvenile wood in a stem. Proportion of juvenile wood that develops in a stem is related to the growth rate at a young age. Stem grown rapidly during the juvenile period will have a relatively high proportion of juvenile wood, as compared to those grown more slowly, early in the rotation period. Reaction wood formation is also apparently affected by growth conditions.

E.1.7.8 Fibre length

Fibre length, like grain orientation, has an effect on wood strength properties. Numerous experiments [14,15] have shown reduced fibre length in softwoods as a result of intensive culture. In contrast, studies involving hardwoods almost invariably show longer fibre lengths after growth stimulation [16,17]. While more information is needed, it is nevertheless clear that fibre length is affected by forestry practices.

In both hardwoods and softwoods, rotation age and growth-accelerating treatments such as fertilisation and irrigation have an effect upon average fibre length in a stem. Rotation age affects fibre length for two reasons. First, juvenile wood near the core is made of shorter fibres than those characterising subsequently formed wood.

E.1.7.9 Forest decline

Ecological problems, such as the genetic decline of the forest resources and the negative impact of air pollution on soil and ground water, have damaged forestry and caused forest decline. This tends to negatively influence the wood quality, and in the end the durability of wood building materials. In an investigation comparing wood properties of healthy and diseased trees from a same stand in Sweden, it was concluded that the sapwood percentage in diseased trees was clearly smaller than in healthy ones, diseased trees had higher moisture deficit in the inner sapwood, and discoloration on wood from the diseased trees started earlier and more intensively [18]. It was also found that phenol compounds in outer heartwood of diseased trees was higher than that in healthy trees. The phenol extractives are important. They strongly influence the natural durability of wood materials.

One factor related to natural durability that has become very important is the decay resistant of second-growth timber. Until recently, naturally durable species were primarily cut from old-growth forests. However, these stocks are nearly depleted, and we now harvest many trees from second-growth stands. Preliminary results suggest that the wood from these second-growth trees lacks the natural durability found in the old-growth trees [19,20]. As timber today is produced largely from managed, second-growth forest, it is necessary to change our reliance on natural durable woods, or develop methods for encouraging the growth of these woods at earlier stages within a rotation age. Increased attention should be paid to decay resistance as an important property to be selected for and enhanced in tree-breeding process.

E.2 CHEMICAL PROPERTIES

E.2.1 Chemical composition

Wood consists basically of carbon, oxygen and hydrogen. There are also small inorganic content. These basic elements are combined into a number of organic polymers viz.:

- Cellulose
- Hemicellulose
- Lignin

These agents are formed in the photo-synthesis.

E.2.1.1 Cellulose

Cellulose is built up by glucose units $(C_6H_{12}O_6)_n$ where n (the degree of polymerisation) is of the order of magnitude of 30 000. The length of the longest cellulose molecules is about 10 mm. The diameter is less than 1 nm (10^{-9}m).

E.2.1.2 Hemicellulose

Also the hemicellulose is a polymer, however with a much lower degree of polymerisation than cellulose. The number of units in the molecule is some hundreds. The polymer is ramified in contrary to the molecules of the cellulose that are of the chain type.

E.2.1.3 Lignin

The lignin molecule is rather complicated and has a high molecule weight. The basic element is of a phenol-propane type. The composition varies highly between species. In the wood structure the lignin appears between the single cells as well as in the cell walls. It acts – together with pectin – as adhesive between the cells and it is therefore of major importance for the mechanical properties of the material.

E.2.1.4 Structure of the cells

The shape of the wood cells is complicated and rather fascinating. The cellulose molecules are bundled into tube-shaped structures – microfibrils – where the single molecules are bound together with hydrogen bonds. The cellulose molecules in the fibrils appear both as parallel, almost crystalline and as unorganised amorphic parts. The structure is shown in fig E.2:1

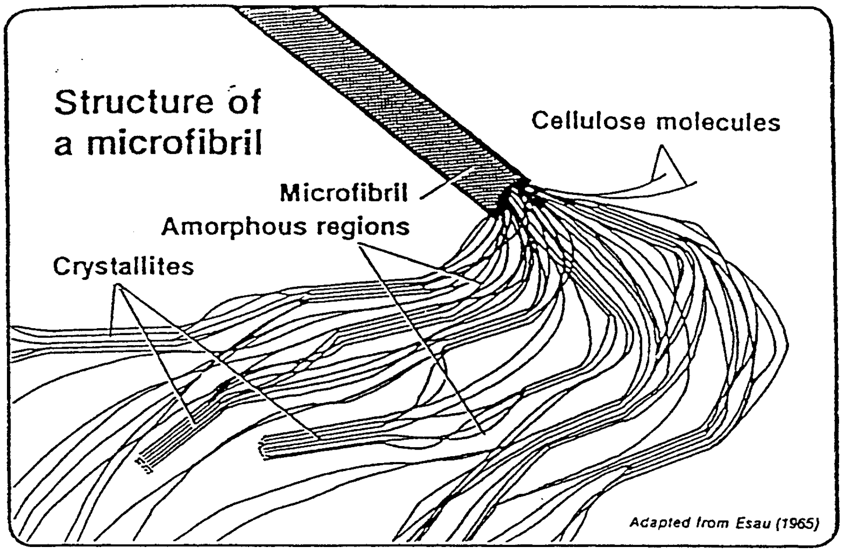

Fig E.2:1 Schematic structure of the microfibrils [21]

In the cell walls a number of parts can be seen, and a cross section of a mature wood cell is shown in fig E.2:2. The primary wall is lignin-rich and reinforced by a net of randomly oriented microfibrils. In the secondary wall the microfibrils are oriented in a direction that varies during the formation of the cell. First the angle between the fibrils and the length axis is 50-70° (layer S_1). Later the direction is changed to 10-30° (layer S_2) and finally almost perpendicular to the axis (layer S_3). Therefore, the secondary cell wall has a very pronounced spiral structure.

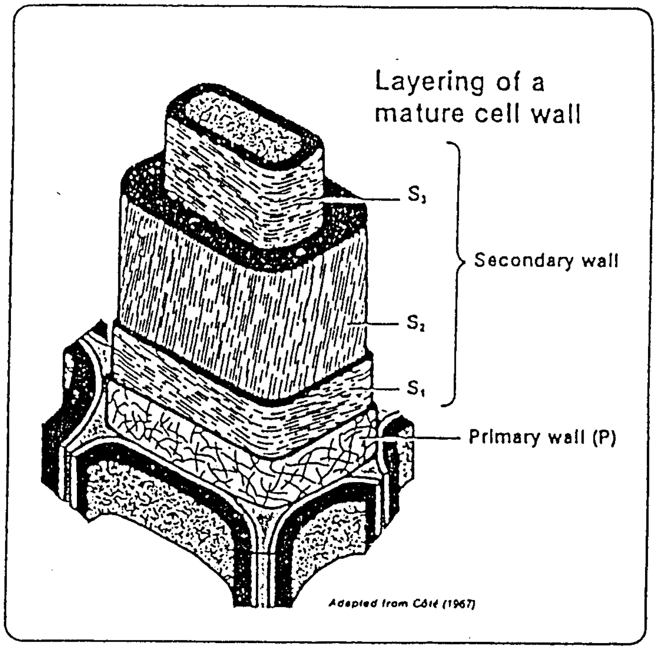

Fig E.2:2 The different parts of the cell wall [21]

Wood has a low nitrogen content, ranging from 0,03 to 0,1 percent by weight, in contrast to the herbaceous tissues that normally contain 1 to 5 percent. The low nitrogen content in wood reduces its susceptibility to decay.

Acceleration of growth can affect chemical composition of a stem. Some studies suggest that fertilisation using high-nitrogen-content fertilisers can adversely influence durability. In theory, fertilisation should increase tree growth, producing a wider band of decay-susceptible sapwood. Furthermore, higher nitrogen levels are often correlated with increased susceptibility to fungal attack [22].

E.3 TYPICAL USAGE

Wood and wood based materials are used in a wide variety of applications in modern buildings. It is found in load-bearing structures, lightweight partitions and facades with or without surface treatment. It is highly appreciated by architects as a valuable material when creating an attractive external and internal environment. It is also a dominant material for doors, window frames, cabinets and kitchen furniture. Critical properties that sometimes restrict their use are related to fire protection, biological attack and moisture-induced deformation checking and cracking.

E.4 TYPICAL MICRO ENVIRONMENT AND LOADS

E.4.1 Durability

Wood forms part of the natural cycling and is influenced by biological and other processes, deteriorating the material. This causes obvious problems and requires special measures in buildings that normally are expected to have a significant service life.

E.4.2 Deterioration mechanisms

The dominating deterioration mechanisms for wood in buildings are biological, mainly rot fungi of various types but also insects and sea organisms. Some observations of attack from bacteria are also reported. Also UV-radiation affects the wood structure.

E.4.2.1 Fungi

There are a large number of fungi that attack wood. They can be separated into two main groups, *staining fungi* and *decay fungi*. They all need high moisture content (fibre saturation) or higher and a suitable temperature. One consequence of this relationship is that if the moisture content can be kept sufficiently low, no deterioration of this type will occur and, accordingly, dry wood structures have demonstrated very high durability. Well known examples are some Norwegian wooden churches which are about 1000 years old. On the other hand, if moisture and temperature conditions are favourable for the fungi, the deterioration can be very fast (a few years).

E.4.2.1.1 Staining fungi

Staining fungi as such are rather harmless. They do not attack the wood substance but is living from nutrition agents solved in the cell lumen. However, the existence of stain is an indication of high moisture content and that the conditions for development of the far more dangerous decay fungi may be present. Important discoloration is caused by staining fungi of *Ascomycotina* and *Deuteromycotina* that invade mainly parenchymatous cells in the sapwood, and the discoloration result from the masses of pigmented hyphae in wood cells.

E.4.2.1.2 Decay fungi

The main types of decay fungi are *brown rot, white rot* and *soft rot*. In buildings, brown rot is the far most common. The most important decay fungi causing brown rot are:

- *Serpula lacrymans*
- *Coniophora puteana*
- *Paxillus panuoides*
- *Antroida sinuosa*
- *Lentinus lepideus*
- *Gloeophyllum sepiarium*

E.4.2.2 Insects and sea organisms

The most important insects attacking wood components in buildings are:

• *Hylotrupes bajulus*	(Long-horned beetle)
• *Anobium punctatum*	(Common furniture beetle)
• *Callidium violaceum*	
• *Camponotus spp*	(Carpenter ant)
• Termites	

Termites exist approximately between the 50th latitudes (north and south) and can cause very rapid and extensive damage to wood structures. In Europe their presence is to be regarded in the Mediterranean area but generally not north of the Alps.

The wood destroying sea organisms require high salt content (above about 3%) and is therefore not a problem e.g. in the Baltic Sea. The most well known organisms of this type is *Teredo navalis.*

E.5 TYPICAL DEGRADATION AND FAILURE MODES

E.5.1 Variations in durability within and between trees

Wood is a highly anisotropic material. There are many variations in the natural durability within and between wood species.

E.5.1.1 Variations within individual trees

The decay resistance of heartwood generally tends to be at maximum in the outer heartwood and decreases inward to the pith [23,24]. This pattern is particularly marked in the butt log of the tree in natural durable softwoods and in hardwoods of both temperate and tropical regions, with heartwood that is resistant to deterioration. The variation in durability of heartwood in the radial direction is due to the kind and concentration of toxic chemical constituents of heartwood [25]. That any appreciable difference in durability of heartwood exists in the same growth increment of the tree, as determined at different height of the stem, is fragmentary and inconclusive. Reduction in durability of heartwood within the same increment with increased distance from the base of the tree has been reported [24,25].

E.5.1.2 Variations between trees

The natural durability of the wood of individual trees of the same species may vary within wide limits. Such variability is thought to be largely genetically controlled, although tree vigour and the fertility of the soil on which the trees are grown are known to influence fungal resistance of the heartwood. In addition, size and age of trees also have some effect.

E.5.2 External factors associated with environment in service

The external factors, in contrary to the internal factors that are associated with various wood properties, are related to the environmental conditions in service, which the wood materials are subjected to. When wood is exposed outdoors, it undergoes two kinds of deteriorating processes: (1) natural weathering or ageing process, which is caused by a complex combination of chemical, mechanical, and photo energy factors; and (2) decay and discoloration process, which results from decaying and discolouring micro-organisms acting in the presence of excess moisture for a relatively extended period of time.

As wood exposed outdoors mainly experiences the two degradation processes, the external factors can, accordingly, be further divided into two kinds of degrading agents, e.g. factors associated with the natural ageing process and micro-organisms, respectively.

Besides internal factors (the natural durability of wood derived from distribution and quantity of extractives, and wood properties such as moisture content, density, texture, resin content, width and orientation of growth ring, etc. as discussed previously) and the external factors (the natural ageing process and biological decay attack), the service life of wood building components exposed outdoors is also dependent on many other contributing factors. These factors include the property of finishes used, application techniques, building design, pretreatments, and maintenance. The influencing factors and degrading effects are summarised in fig E.5:1. As the attention here is focused on the durability evaluation of wood materials, other influence factors on wood materials used in outdoors buildings associated with architectural design, maintenance, and protective coatings are excluded in this discussion.

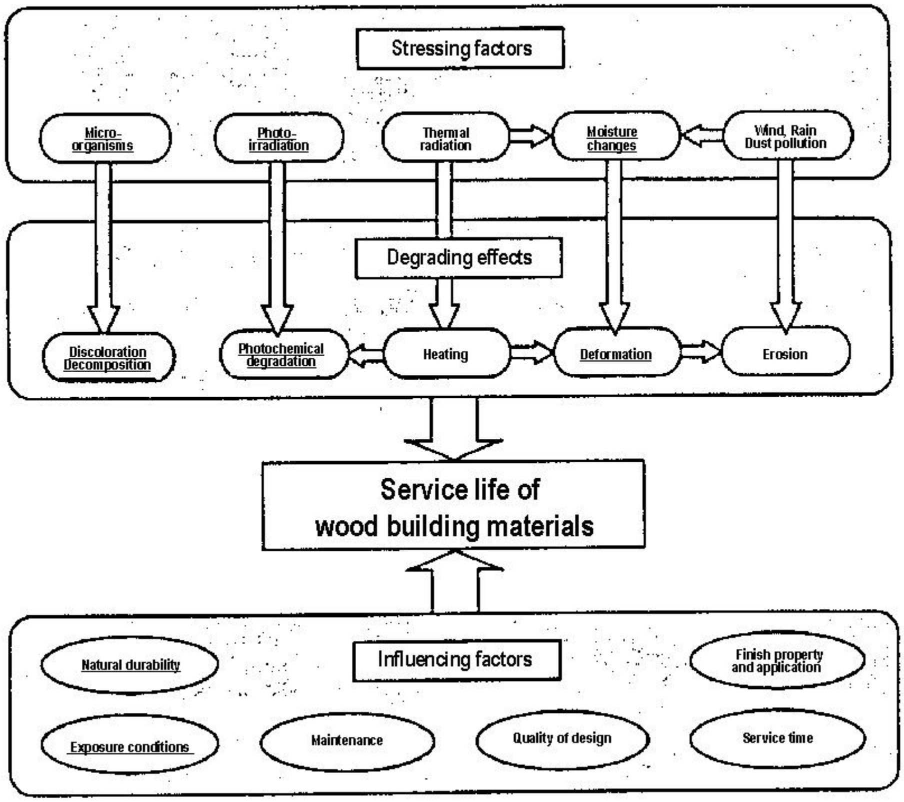

Fig E.5:1 The stressing and influencing factors for wood in buildings

E.5.2.1 Ageing process and its effect

Like all other building materials, when wood is unprotected by finishes and exposed outdoors, a slow chemical and physical disintegration occurs near the surface. The colour of the wood turns to grey and silver, the cells at the surface are roughening and slowly broken down, and the wood surface is gradually eroded. This degradation is commonly termed as natural ageing or weathering process of wood. The ageing process and its effect on wood is schematically shown in fig E.5:2.

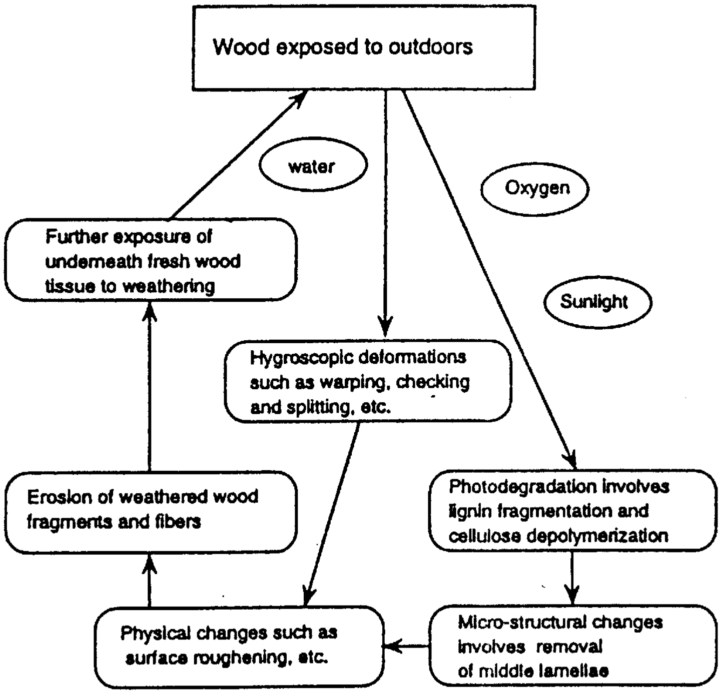

Fig E.5.2 Weathering and ageing process of wood exposed outdoors

The major deterioration mechanism associated with the natural ageing process is photochemical degradation (UV-light) to wood cell-wall constituents, photo-oxidation of the breakdown substances, leaching of the soluble decomposition products, and related mechanical damage of surface elements from the constant swelling and shrinking of the wood associated with surface wetting and drying.

Wood ageing is a complicated natural process and its mechanism is not fully understood. Wood ageing results from a series of combined factors. The initial colour change is the result of lignin and extractive photochemical decomposition, forming free radicals, which lead to further decomposition of the structural carbohydrates and oxidation of phenolic compounds. Light usually does not penetrate wood more than 200 µm in depth. The natural degradation reactions, therefore, are surface phenomena. Xylans are decomposed and leached more readily than cellulose or glucan-rich hemicelluloses. Continual wetting and drying of wood surface with its concomitant swelling and shrinking, lead to surface checking and erosion. These ageing losses are negligible over the service life of most wood assemblies. However, they may become significant for long-term uses of wood materials in outdoor applications [26].

E.5.2.2 Decaying and discolouring organisms and their effects

Many changes in other physical and chemical properties are caused by decaying organisms. These changes range from drastic effects on wood to subtle modifications in properties such as density, and hygroscopicity, electrical conductivity, acoustics, calorific values and dimensions. These changes and their rates of development in wood building materials vary with wood species, the environmental conditions and the organisms involved. It is important to remember that changes in one property are invariably associated with changes in other properties. For example, a loss in wood weight decrease its strength properties. Some subtle changes in properties such as moisture permeability may increased decay susceptibility and set the stage for further colonisation of other more destructive agents of decay.

Strength and volume reduction are the principal losses associated with decay. Drastic strength losses occur in the incipient decay. At high-stress utilisation of wood, such as in outdoor applications, failure to detect early decay can lead to serious losses. Drastic alternation in dimensions and chemical and physical changes of wood adversely affect many properties, including decreased volume and mass resulting in dimensional collapse of supporting timbers or reductions in many strength properties, increased permeability to liquids, aesthetic losses due to abnormal colours, rough textures or pulled fibres.

E.5.3.3 Protection of wood by chemicals

When the conditions do not ensure that the moisture content can be kept below critical values – by design or surface protection – a chemical treatment with fungicides can be applied. This treatment can be performed in different ways depending on the situation and the required level of protection. The most efficient method is to get the fungicide agent into the wood structure by an industrial vacuum/overpressure technique. There are a large variety of agents and processes for this purpose. Environmental concerns have led to restrictions on certain agents and there are on going and extensive R&D activities to develop new agents, tailor-made to specific applications and with an acceptable impact on the environment.

It is common to distinguish between pressure and vacuum impregnation depending on the type of industrial process. At pressure impregnation water based agents based on salts of chromium, copper and/or arsenic are used which give the wood a characteristic green colour. Vacuum impregnation is normally made with uncoloured agents solved in white spirit. It is solely used for joineries such us windows, doors and outdoor furniture.

E.6 DEGRADATION MONITORING METHODS

E.6.1 Evaluating weathering resistance

When discussing the durability evaluation in the present chapter, both deteriorating factors associated with ageing process and micro-organisms should be considered in accordance with the author's definition of durability of wood building material. There is no apparent standard method for evaluating the ageing resistance of wood building material. For the evaluation of natural resistance ability to micro-organisms, many standard laboratory test methods have been developed, for instance, ASTM D 2017-81(1986).

E.6.2 Evaluating natural durability

For wood building materials exposed outdoors and above-ground, the most important property which largely determines its performance is its resistance to microbial attack. This resistance ability for untreated wood materials is principally dependant on the natural durability of wood material itself.

In general, natural durability has been evaluated by exposing wood samples to the decay agents for various periods and rating the resultant degree of degradation. These tests are sometimes supplemented by extracting the wood and evaluating the toxicity of various fractions against specific organisms. Field trials of natural durability were prevalent beginning in the 1920s, as scientists began to evaluate the properties of many wood species [27,28]. These tests reflected a desire to identify wood with properties similar to existing natural durable species. In USA the toxic nature of heartwood extractives was first established and posed as the reason for decay resistance [29].

Laboratory assay to evaluate natural durability began in the 1940s in an attempt to further explain the durability and identify the toxic compounds [30-35]. In most cases, water, ethanol, or other solvents were used to remove extractives from the wood. These extractives were then tested for activity against a variety of decay and non-decay fungi. In addition, these extracts were often tested for activity against common wood-destroying insects [36]. Most tests were performed in petri dishes or decay chambers using nutrient agar. Although such tests provide a relative guide to chemical toxicity they cannot evaluate more subtle effects such as variation in deposition of extractives in the wood or interactions between different extractives that must also play roles in natural wood durability.

Much of the laboratory research was performed to identify chemicals that could be synthesised as natural preservatives under the premise that such chemicals would be inherently safer and more effective than other biocides. In fact, many of the chemicals responsible for natural durability are as toxic or more toxic than existing wood preservatives. The major advantages of employing naturally durable woods over artificial wood preservatives is the elimination of the need for treating facilities to deliver chemical into the wood. Natural durability, however, can never completely replace the need for wood pressure-treated with chemicals, since some hazards, such as outdoor exposure, are too severe for adequate performance of even the best naturally durable wood species [37].

Improved chemical assay methods, using such techniques as radioisotope labelling, C^{13} nuclear magnetic resonance (NMR) spectroscopy and ion magnetic spectroscopy, have enhanced the study of natural durability, but methods for studying *in situ* deposition of toxic extractives are still lacking. The use of tissue-culture techniques may improve our knowledge of synthesis of heartwood extractives, but it will be difficult to study subsequent deposition processes using these methods. Developing an improved understanding of the nature and distribution of extractives related to durability could be especially useful for identifying new approaches to depositing chemicals in wood and protecting it. This approach is particularly interesting because of evidence that the decay-resisting extractives in some very durable woods are distributed in the wood in a manner more resistant to leaching than fungicides that are applied artificially.

However, as concerns about the use of artificial wood preservatives increase, we may come to depend increasingly on natural durability for wood used in some locations, such as outdoor exposure, even under higher decay hazards. This increased demand comes at a time when supplies of durable species are declining and when concerns are being raised about decreased durability of second-growth timber. These trends suggest that renewed efforts must be made to identify methods for improving the genetic capabilities to produce durable heartwood and develop silvicultural practices that favour maximum production of this wood in the shortest period.

E.6.2.1 The British standard BS 1982

One of most widely accepted and used standards for evaluating natural durability of wood is the BSI standard (1980:1990, part 0, part 1, part 2, and part 3) "Fungal resistance of panel products made of or containing materials of organic origin", which specifies a common soil or agar/block test. This test is often not suitable for some types of wood panel products, e.g. wood-based board. The method in part 2 involves small stakes partially buried vertically in non-sterile soil. The moisture gradient that occurs in these stakes allows the test fungi to attack in the area of most suitable moisture content. It is well known that some chemical constituents provide temporary protection against wood decay fungi. This protection is eventually lost in service. The ageing effects on wood durability and the relation between ageing of wood and fungal susceptibility of wood material is not considered in the standard. It is essential that ageing effects on durability should be included in the durability evaluation.

The test samples are usually small blocks. Comparison of test results from small block with that from larger full-size products in practice can be difficult to be used as indication of relative performance of the wood materials in service.

E.6.2.2 The American standard ASTM D 2017-81(1986)

The American standard for evaluation natural decay resistance is ASTM D 2017-81(1986): "Method of accelerated laboratory test of natural decay resistance of wood". This method is similar to the BSI standard and has almost the same limitations. None of the methods is specially designed for testing of wood materials exposed outdoors and above-ground. It is not suitable to use this method to assess the combined effects of ageing in service conditions and the decay fungi on the durability. The methods are also too simple to evaluate the multiple actors and their inter-reactions on the durability. They are mainly designed to clarify effects of one or two detrimental factors.

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Joint CIB W080 / RILEM TC 140 – Prediction of Service Life of Building Materials and Components

Guide and Bibliography
to
Service Life and Durability Research for Buildings and Components

PART II – Factors Causing Degradation

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PART II — FACTORS CAUSING DEGRADATION

S. E. Haagenrud[1]*, NILU - Norwegian Institute for Air Research*

A Introduction

A.1 GENERAL

In the developed countries, the building stock and infrastructure constitute more than 50 % of each country's real capital. The built environment is currently in a deteriorating state. After the "build and let decay" age during the last 30 years, the concern is not only the environmental impact on the cultural heritage. Generally, the damages to building materials and constructions have become an enormous economic, cultural and environmental problem.

The wasteful consumption of energy and materials linked to the degrading built environment makes this a major environmental problem in the context of sustainable development.

To safeguard our built environment, action is urgently needed, see fig A.1:1. In principle, there are two possibilities – and both should be pursued in parallel. Firstly, the society should try to improve the exposure 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 via cost-benefit analysis of degradation of materials and buildings, while the second issue is the concern of many R&D and standardisation programmes around the world [1-4].

An international standard for estimation and prediction of service life of building materials/components and buildings is currently in the process of being elaborated within ISO/TC59/SC14, previously ISO/TC59/SC3/WG9 [5-6]. This group was set up from the joint initiative for standardisation of service life methodologies by the EUREKA umbrella project EUROCARE and CIB/RILEM, towards CEC and CEN in 1991. The work is based upon the generic RILEM recommendation for prediction of service life [7]. In Europe, the entry into force of the Construction Products Directive (CPD) also creates an urgent and increased need for standards addressing the issue of durability [8].

The service life prediction models for building materials and components are based upon knowledge of degradation mechanisms and dose-response functions. For industry to respond to the standards and requirements, a lot of data and knowledge in this field need to be compiled or generated. As shown in the following, much of these data could be provided for through extensive co-operation with the meteorological and environmental research community.

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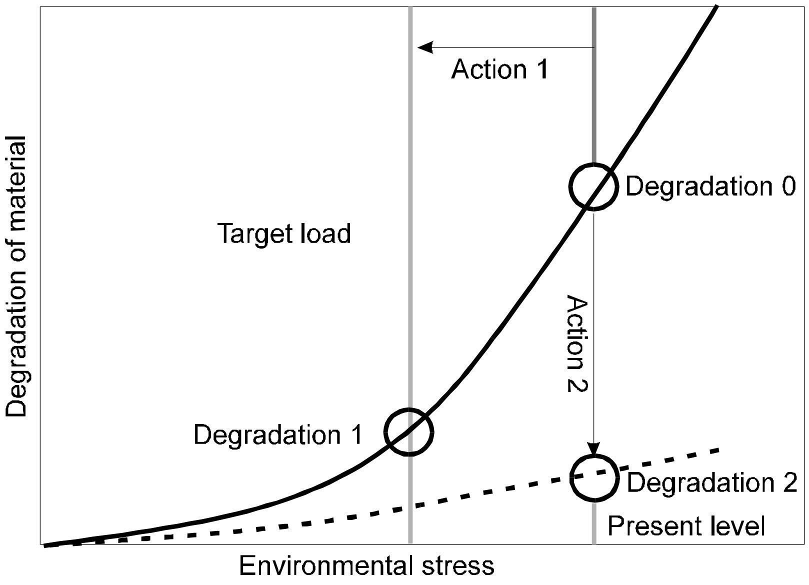


Fig A.1:1 The two principal actions for improving the durability of the built environment

A.2 AIM, SCOPE AND SUMMARY OF PART II

The *aim* is “to present *state of the art* and *research needs* regarding *characterisation* of degradation environment, including *methods* and *equipment* for *micro climate* monitoring”.

The *scope* is “to identify methods for measuring and characterising these primary environmental factors, to demonstrate how these measurements can be related to laboratory measurements, and identify areas in which standards are necessary to improve the precision and accuracy of these measurements”.

Characterisation of environmental degradation factors should be linked to the knowledge of degradation mechanisms and dose-response functions. Therefore the knowledgebase of such functions will also be reviewed.

Further it will be discussed what measuring methods and data that are available for characterising the ambient exposure environment, and how these data can be classified. Focus will then be put on *micro environmental* measurements and data, the lack of which are one of the main barriers for reliable service life predictions of building materials and components.

Section B introduces an overview of the content of this part. A generic model for degradation of materials and the important concepts of *dose-response, damage* and *performance-over-time functions*, and their relationship are presented. Dose-response functions transfer to damage functions when the performance requirements or limit states for allowable degradation are implied. The establishment of the limit state is complicated and should involve consideration of technical, economic and environmental aspects.

Levels of geographical scale are defined, and the requirements for characterisation, classification and mapping of environmental degradation factors are described. The importance of exploitation of information technology in building assessment is emphasised.

In order to be able to characterise and report the right type and form of the environmental degradation factors, they have to be related to the degradation mechanism and dose-response functions for the specific materials in question.

Section C is a review of recommended dose-response functions produced in important environmental field studies during the last three decades. These are:

1. NAPAP Lipfert study, who composed a data base of atmospheric corrosion test results for metals, together with environmental variables published from eight test programmes at up to 72 test sites in many countries [9];
2. Kucera et al. established dose-response functions for carbon steel and zinc via exposure at 32 test sites in Scandinavia [10];
3. Feliu and Morcillo in Spain surveyed and compared data from 250 test sites in 28 countries in many parts of the world, and developed dose-response functions for carbon steel, zinc and copper [11]; and
4. The UN ECE ICP on materials, who is evaluating the effect of airborne acidifying pollutants on corrosion of materials, involving exposure at 39 test sites in 12 European countries and in the United States and Canada [12].

Section D provides basic characterisation of the principal agents that affect the durability of building materials, and some knowledge on the degradation mechanism. It follows the characterization in standard ISO 6241-1984 (E) [13] of agents relevant to building performance and requirements. The degradation factors are classified according to *their nature* as *mechanical*, *electromagnetic*, *thermal*, *chemical* and *biological* agents, and to their *origin* (external to the building: atmosphere and ground - internal to the building: occupancy and design consequences), and *not to the nature of their action* on the buildings or components. In this part an overall *emphasis is on the outdoor atmospheric exposure environment*.

There is a clear lack of data on quantitative characterisation of degradation factors in their relevant type and form, and in geographical resolution. It will require much more research on quantitative damage functions, also incorporating the relevant degradation mechanisms, for the right characteristics to be provided. Another problem is the identification of the right performance characteristics or degradation indicators. Although quite a few good examples on the right approach are shown [14-16] and a whole lot of good studies on dose-response functions exist from the environmental research area, it is a clear lack of relevant functions. This lack of systematic approach is a major barrier for further progress in the area.

Section E *contains an overview of international programmes for measuring, modelling and mapping environmental degradation factors in the environmental research area of air quality*. Point measurements are very expensive, and for a broader assessment of air quality, required for policy development and assessment, public information etc., measured data need to be combined with modelling based on emission inventories. This would allow proper assessment of exposure, and of the effects of the pollution on public health and on other receptors, such as buildings. In the context of assessing building performance, a huge bulk of data on global, continental (macro) and national (meso) levels is therefore available for exploitation. In order the UN Global Environment Monitoring System (GEMS/AIR), the Transboundary UN EC EMEP Programme [17] and the tasks organised under the European Environmental Agency in Copenhagen are described, and some data are given.

The European policies and directives on air quality monitoring, including a status of the European Air Quality Monitoring Networks and the European Environment Agency, is given. On the regional scale about 750 sites are in operation totally in Europe, with very extensive monitoring of sulphur and nitrogen compounds in air and deposition, while ozone data are available from about 500 monitoring sites in 14 member states.

On the local urban scale, monitoring is carried out at a very large number of sites in Europe, totally close to 5000 sites, where compounds of the EU directives (SO_2, particles, NO_2, ozone, lead) are extensively covered. The *extensive reporting of events of exceeding threshold values* should be of extreme importance also for the building society, when threshold values for damage to building materials could be established.

Further on, the section contains overviews of ISO standards for air quality and of measuring devices for continuous monitoring of emissions and ambient air quality.

Fully integrated information and management systems for air quality are now rapidly entering the market, and one such system is described. The system also contains *effect modules* allowing for assessment of damages due to population and building *exposure*, and for evaluation of recommended abatement strategies. A module for modelling and calculating buildings degradation, service lives and maintenance costs has been developed and used.

Section F *describes the principle in the classification and mapping of environmental degradation factors and corrosivity*. Classification as described in the international standards ISO 9223-26 “Classification of atmospheric corrosivity for metals” [18] employs two approaches: classification in terms of the corrosion determining environmental parameters; and classification based on corrosion rate measurements of standard samples exposed in the environment concerned. In this report emphasis is put on the first approach. These ISO standards have improved maintenance as their prime objective, and represent a huge step forward as they for the first time in this context describe a system for quantitative characterisation and classification of the important environmental degradation factors. These standard approaches can also be used for other types of materials, and such standards are beginning to emerge.

Examples of mapping are given according to the service life (and ISO 9223-26 [18]) approach from Japan, UK, Spain, US and the worldwide ISO CORRAG programme.

In the recent years the environmental concern and the strive for sustainable development have also generated a need for mapping the corrosivity of the exposure environment, in order to provide input to environmental regulations. This is done both by using corrosivity mapping for proper cost-benefit analysis of building degradation, and now also mapping according to the critical-load/level approach being used in environmental research.

The mapping procedure developed in UN ECE [17] and the interesting, related Environmental Assessment Level approach in UK are described, and examples of implementation of these approaches are given for the urban area of Oslo and for UK, respectively.

The mapping of dose-response functions and acceptable corrosion rates would be of extreme value for the building society. Such maps can easily be transformed into maps for service life and maintenance intervals, if the performance requirement is defined for the material in question. Input from the building society in this context is a necessity, and could then serve as a tool for maintenance planning for individual users etc. These maps would therefore be most important for the standardisation work going on within CEN and ISO/TC59/SC14 Design life of buildings. In that respect the question of transformation and validation of the dose-response and damage functions to the microenvironment on the building surface has to be addressed.

Section G *overviews some of the monitoring systems available for measurement of important degradation factors in the microenvironment on buildings*. Multi-sampling instruments with automatic data handling presentation tools are now available for temperature, wetness and UV, and passive samplers for gases and particles also exist. Other methods are certainly available around the world. Examples of results and use are given for methods described.

The air quality and exposure of buildings can also be modelled by available dispersion models for roads and street canyons. In the environmental research, traffic planners often require practical tools for studying the effect of abatement strategies on air quality in streets where people are exposed. A PC-based model exists that calculates total emissions, concentrations along each road segment and the air pollution exposure of the population and buildings along each road.

Section H lists the proposed needs for R&D and standardisation concerning the issue of characterisation of environmental degradation factors, as follows:

1. There is a lack of systematic knowledge of the right type and form of degradation factors to be characterised. In order to produce this knowledge, extensive service life research based on damage functions approach has to be carried out. It should involve models for degradation mechanisms and should be tested out in well-monitored relatively short exposure programmes in field and in laboratory.
2. Damage function based research and proper characterisation of degradation factors both in field and laboratory is one absolute criterion for relevant coupling of field and laboratory tests.
3. Some methods for automatic and continuous monitoring of important degradation factors in the microenvironment on buildings exist, but testing and further development of methods are strongly needed.
4. Quite a few dose-response functions exist today after extensive research in the environmental research area. However, these functions have to be tested and validated in the microenvironment on buildings. Measuring and modelling methods for micro environmental loading and materials degradation have to be developed and extensive measurements carried out.
5. The dose-response functions produced so far often stems from the environmental research area. They are therefore limited in terms of choice of degradation indicators. In addition, they do not contain identified limit states, and are therefore not damage or service life functions.
6. Interdisciplinary co-operation between the building and environmental research community is a must.

B Service life prediction concept – need of data

B.1 MODEL FOR DEGRADATION

With reference to the “definition” and “preparation” steps in the generic service life methodology [6-7], data are needed on the (see fig B.1:1):

- Performance-over-time functions for materials and products;
- Dose-response or damage functions for materials; and
- Degradation agents at a macro-, meso- and microenvironment level.

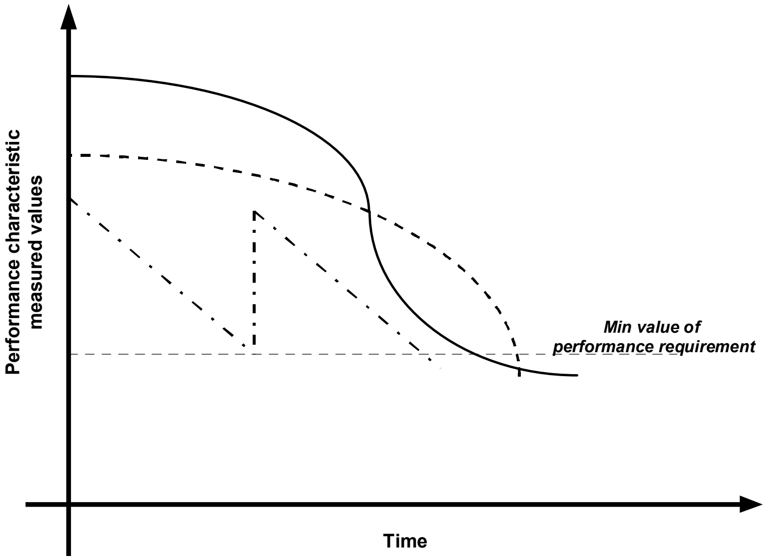


Fig B.1:1 Performance over time functions

Materials degradation and loss of characteristic properties, as described by performance-over-time functions (fig B.1:1), are in most cases due to chemical or physical deterioration or corrosion. The corrosion can be expressed by a mathematical model consisting of a power function of degradation factors and elapsed time:

$$M = at^b \qquad (1)$$

where:

M corrosion at time t

a rate constant, which is a function of the deposition of pollutants or other degradation agents to the surface

b power exponent governed by diffusion processes, where $b \approx ½$ for the case of corrosion products forming a protective layer through which fresh reactants must diffuse (fig B.1:2)

In the present context eq. (1) will be defined as a *dose-response function*, aiming to describe the physico/chemical relationship between material degradation and degradation factors of the exposure environment. This terminology is not in accordance to a strict definition of dosage [19], but the term is already established in the environmental research area, where the generic eq. (1) has been used to analyse and reconcile a great number of so-called dose-response functions. In the environmental area a huge amount of studies have been performed aiming to find the corrosive effect of air pollutants and to establish the relationship between materials' decay and the environmental degradation factors. Although generating a lot of useful data and knowledge about the effects, the studies lack a homogeneous approach in terms of measurements, time frames and data-analytical procedures. A survey of many of these studies and their reconciling is given in section C.

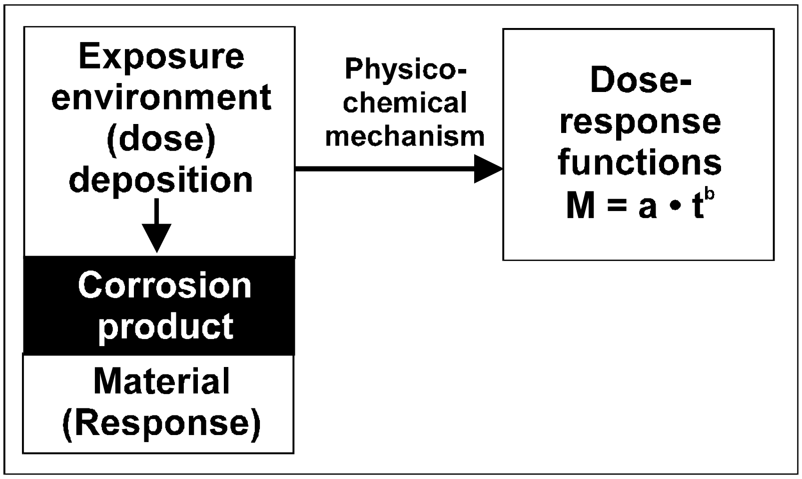


Fig B.1:2 Degradation model

After having generated experimental data from long-term and/or short-term tests [20], the dose-response functions can be elaborated by regression analysis. This can be done simply by screening the best fitting function, or, even better, adopting the regression analysis to the best available model for the degradation mechanism. Examples of both approaches are shown in section C.

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.

The establishment of the limit state is complicated, and can be discussed both from a technical, economic and environmental point of view. Within the building society, the first two aspects have so far been dominating [8,13]. However, within the environmental area the discussion has started on fixation of the limit state from a "sustainable requirement" point of view [21], see section F. A convergence of these requirements would have great interest and impact in the building sector.

A major barrier for further progress concerning the durability and service life aspects within the building community is the lack of knowledge and implementation of the damage function approach. This approach is the basis for the level of knowledge in, for instance, high-tech industries and in the medical, biological and agricultural community [16]. Until this approach is taken seriously and adopted by the building community on a broad scale, no substantial improvements in the field of durability of building materials and components can be anticipated. The lack of this systematic approach has become quite evident during the preparation of this part.

The mathematical dose-response function approach has been used, eq. (1), and developed into a cumulative damage model as a computer software application [15]. It was feasible to use the right type and form of the environmental degradation factors, and therefore also to predict and compare degradation rates from long term field exposure and laboratory exposures. These results are convincing and very promising.

The same approach is advocated in [16] where methodologies for predicting the service lives of coating systems are discussed and proposed. *A set of criteria* for assessing the adequacy of the current durability methodology and a reliability-based methodology is established, which includes the ability to:

1. Handle high variability in the time-to-failure data for nominally identical coated panels exposed in the same service environment;
2. Analyse multivariate and censored time-to-failure data;
3. Establish a connection between laboratory and field exposure results; and
4. Quantitatively predict the service life of a coating system exposed in its intended service environment.

They conclude that in order to implement a reliability based methodology substantial changes in the current experimental procedures will be required. These changes result from the quantitative nature of the service life data, and will include:

1. More systematic characterisation of the initial properties of a coating system;
2. Quantitative characterisation of each of the weathering variables comprising the in-service environment;
3. Quantification of macroscopic degradation via cumulative damage function models;
4. Utilisation of experimental design techniques in planning and executing short term laboratory based experiments; and
5. Development of computerised techniques for storing, retrieving and analysing the collected data.

A lot of knowledge can also be transformed from the many studies that have been performed in the environmental research area to establish dose-response and damage functions [22]. However, it should be pointed out that the dose-response functions that are currently available are very limited in terms of choice of degradation indicators and establishment of performance requirements.

Another major barrier to reliable predictions of service life and/or maintenance intervals is insufficient knowledge of the relevant exposure environment. However, substantial knowledge and data exist on the environmental exposure conditions on the *macro* and *meso* level. It is a serious problem that these tend to be in a generalised form such as a contour map of average data, for example mean temperature, humidity etc., while researchers and designers need to consider the *specific form*, for instance time of wetness (see section D.4.1.1), and also the local- and micro-environmental conditions of the building (see section B.2.1).

A *third* barrier is just this adaptation of data and knowledge to the *local* and *micro environmental* conditions. The complexities of a structure can result in very different climatic and environmental conditions on a single structure and greatly affecting damage rates [23-27].

A main problem would also be the differences of the standardised surfaces and real surfaces of components connected with the object [23]. The dose-response functions are primarily established under more or less controlled experimental conditions, and a *major task* would be the transition to real constructions. The dose-response functions must be validated for micro environmental conditions, and testing under in-use conditions have to be done. Measuring and modelling methods for micro environmental loading and materials degradation have to be developed and extensive measurements carried out.

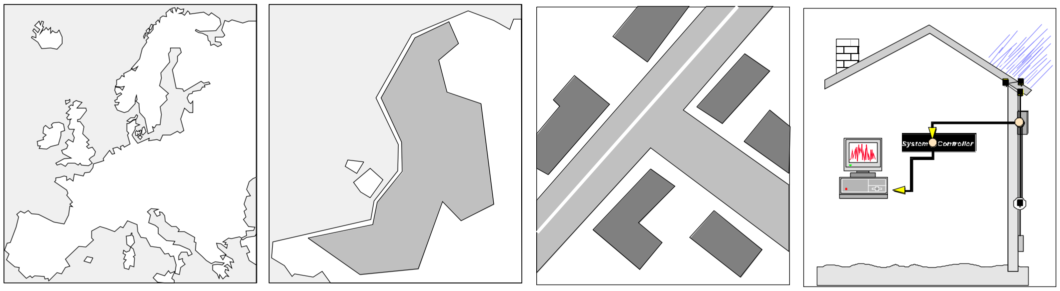
B.2 CHARACTERISATION AND MAPPING OF THE MOST IMPORTANT ENVIRONMENTAL DEGRADATION FACTORS

B.2.1 Definition of scale

The proper use of dose-response functions requires characterisation and mapping of the relevant degradation factors on the different geographical scales.

One frequently used basis of classification for climate, and environment, is the division into macro, meso, local and micro scales [28]. This division means a definition of different scales describing the variations in the meteorological variables. There exist no common and exact definitions of the different scales, but in this report the following will be used (fig B.2:1):

By *macro* is normally meant the gross meteorological conditions described in terms like polar climate, subtropical climate and tropical climate. The descriptions are based on measurement of meteorological factors such as air temperature, precipitation etc. [29].



Macro-Europe map **Meso-**urban area **Local-**road / building **Micro-**building

Fig B.2:1 Exposure environment on different geographical scales

When describing *meso* climate, the effects of the terrain and of the built environment are taken into account. The climatological description is still based on the standard meteorological measurements.

In this report we also use the term the *local* scale. By that is meant the local conditions in the building proximity, such as for example in the streets around the building. The ***micro***climate describes the meteorological variables in the absolute proximity of a material surface. The microclimate or microenvironment is crucial to a material's degradation.

B.2.2 Systems for describing degradation agents

Various systems have been used to classify the degradation agents. The standard ISO 6241-1984 (E) [13] presents a detailed list of agents relevant to building performance. The degradation factors are classified according to *their nature* as *Mechanical, Electro-magnetic, Thermal, Chemical and Biological agents,* and to *their origin* (external to the building: atmosphere and ground - internal to the building: occupancy and design consequences). The same set-up will be used in this report with the overall emphasis on the *outdoor atmospheric exposure environment.*

However, in order to characterise and report the right *type* and *form* of the environmental degradation factors, they have to be related to the degradation mechanism and dose-response functions for the specific materials in question [14,16]. This will facilitate comparisons to be made between field and laboratory measurements.

In the discussion of environmental characterisation in [16], it is stated: “With respect to statistical characterisation, it is known from laboratory based studies that weathering factors act *synergistically* in causing a coating system to degrade, and that often, for values below some threshold, the rate of degradation related to a weathering factor is so low that the degradation at level below this threshold can be neglected for all practical considerations. The existence of *synergistic effects*, the effects of *wet-dry cycles*, and the possible presence of threshold values raise serious questions as to whether characterising weathering factors by their mean value and viewing the degradation effects of these factors as acting independently is meaningful in the context of service life prediction. As an alternative, it is recommended that until more is known about the effect of the individual weathering factors on the degradation process, *time series* for each of the primary weathering factors be simultaneously monitored and characterised.

The quantitative characterisation of in service environment raises some critical questions including:

- Can the weathering factors causing the *predominant* degradation of a coating system in a specified in service environment be isolated from the factors which have only a *secondary* effect?
- Is the *average intensity* of each weathering factor sufficient for characterising the severity of an in service environment, or will a more precise knowledge of each of these factors be necessary?
- Can the value of weathering factor be converted into some *common metric of degradation* (e.g., total dosage, which is the same for both laboratory and field studies)?

In addition to changing the way weathering factors are *statistically characterised*, it is recommended that changes be made in the way they are *physically characterised.* For the purposes of the service life prediction of coatings, metrics for expressing the *severity* of the outdoor weathering factors of temperature and UV degradation are proposed.”

It is interesting to see that the European Organisation for Technical Approval (EOTA), in their Guidance Paper on “Assessment of Working Life of Products” to the Convenors of the Technical Committees has adopted the service life methodology of RILEM [7], the ISO 6241 [13], the damage function approach and that it emphasises the specific need for characterising the exposure environment on the geographic scales of Europe [30].

B.2.3 Sources of data

The measuring, testing and evaluation of air quality are assuming growing importance in developed countries as elements of a comprehensive clean air policy, geared into sustainable development. A huge bulk of data is therefore generated on the various geographical levels.

In the context of assessing building performance, data on global, continental (macro) and national (meso) levels are available for exploitation. To some extent data are also available in the local scale, while most is lacking for the microenvironment on buildings.

Section E, therefore, contains an extensive state-of-the-art report on availability of data and methods from the atmospheric environmental research area on the global and, as an example, the macro and meso scale in Europe.

Methods for making micro environmental characterisation are also available, as discussed in section G.

B.2.4 Environmental classification concept

Based on knowledge of dose-response functions (see B.1), the environmental degradation factors can be characterised and from this characterisation it should be possible to classify an exposure environment relative to its severity. This can be mapped and exhibited in geographical information systems software applications. Such classification systems already exist for some metals [18]. They have been developed for the classification of atmospheric corrosivity and employ two approaches. Either environmental classification in terms of the corrosion-determining parameters, Time of Wetness (TOW) and pollution, or classification based on corrosion rate measurements of standard metals exposed in the micro environment concerned.

The above-mentioned standard approach for classification can also be used for classification of other materials' corrosivity if the dose-response functions/degradation mechanisms are known.

Degradation of non-metals is generally caused by other mechanisms than those responsible for the corrosion of metals, even if oxidation processes are important in most cases. For example, a generic classification of degradation environments for polymeric materials in outdoor use calls among other things for improved knowledge concerning surface temperature, UV-radiation, moisture periods, and the influence of air pollutants [15].

Corrosivity mapping and environmental characterisation were basically undertaken to meet the needs for better maintenance and design of constructions.

However, in the recent years the environmental concern has generated a need for mapping the corrosivity of the exposure environment, in order to provide input to environmental regulations.

A review is made of the various classification approaches used today. According to the scope of this part, the emphasis is on systems for classification of the environmental degradation factors (see section F).

B.3 USE OF INFORMATION TECHNOLOGY IN BUILDING ASSESSMENT

Monitoring and assessing the performance of a building and its components will require extensive amounts of data. The inability to integrate the accumulated information has hitherto been a serious barrier to further development. With the recent leap forward in information technology this delimitation is no longer present. Micro environmental multisampling techniques, user-friendly on-line presentation and complete facility information and management systems are available for managing and integrating geographical information. Examples of the development and use of such systems will be described (see sections E.8 and F.6).

C Review of dose-response functions

Many studies have been performed to establish dose-response functions [21,22]. Conclusions from some of the most important studies are presented in Table C.4:1.

C.1 NAPAP – LIPFERT

In the US National Assessment Program for Acid Precipitation (NAPAP), a compilation of a database of atmospheric corrosion test results for metals together with environmental variables, published from eight test programmes carried out at up to 72 test sites and dating back to the 1960's, was carried out [9]. A substantial part of these data were obtained from the Scandinavian countries. Statistical regression analysis was used to infer dose-response functions using several different basic model formulations.

In [9] it is concluded that time of wetness is most important for steel and that SO_2 seems to be the only pollutant that is important for the four metals (see Table C.4:1), on the basis of confidence limits calculated for the various coefficients.

The critical level of relative humidity for corrosion to occur differs among the four metals, indicating differing corrosion behaviour. The final conclusion was that large numbers of observations are required in order to generate statistically robust dose-response functions based on essentially uncontrolled field exposures. Regression analysis indicates the predictability to be only within a factor of two or more, for any given time period and location with weight loss of the specimen used as the damage indicator.

Lipfert has also surveyed corrosion data and environmental data for calcareous stone from nine field tests in USA and other countries [31]. The theoretical dose-response function developed is based on recognition of three basic mechanisms for stone degradation:

1. Normal calcite dissolution in clean rain (pH < 5,6);
2. Dry corrosion by deposition of gaseous air pollutants, notably SO2 and HNO3; and
3. Wet corrosion acceleration due to rain acidity.

C.2 SCANDINAVIAN STUDIES

In a Nordic field exposure programme sponsored by NORDFORSK from 1975–1984, dose-response functions for carbon steel and zinc were established via exposure at 32 test sites in Scandinavia [10]. Dose-response functions were developed for carbon steel and zinc by linear regression analysis.

NILU performed, on contract for the Norwegian Pollution Control Authority, detailed corrosion assessment studies in four areas in Norway. Linear statistical analysis was used to develop the dose-response functions, of which the ones for metals in the urban area Sarpsborg/Fredrikstad are presented in Table C.4:1. For this district the steel corrosion was modelled from the developed dose-response function, based on the modelling of SO_2 distribution by means of NILU's air dispersion model [32].

C.3 FELIU AND MORCILLO

In Spain, Feliu and Morcillo surveyed and compared data from 250 test sites in 28 countries in many parts of the world [11]. Statistical analysis was performed to establish the dose-response functions for carbon steel, zinc and copper shown in Table C.4:1.

C.4 UN ECE ICP

The most wide-ranging and best-designed test programme is the International Co-operative Programme (ICP) undertaken within the United Nations Economic Commission for Europe (UN ECE). The programme, started in September 1987, aims to evaluate the effect of airborne acidifying pollutants on corrosion of materials and involves exposure at 39 sites in 12 European countries and in the United States and Canada. (fig C.1:1).

Samples have been withdrawn after one, two and four years of exposure and dose-response functions have been developed for carbon steel, zinc, aluminium, copper, bronze and calcareous stone [12], as shown in Table C.4:1. The equations should at present be seen as provisional and may be subject to further elaboration when the results after eight years of exposure will be available in 1996.

For unsheltered exposure most of the dose-response functions have the same form:

$$ML \text{ or } MI = a + b\,TOW\,[\mathrm{SO_2}][\mathrm{O_3}] + c\,Rain\,[\mathrm{H^+}] \qquad (2)$$

This is the first time a synergistic effect of O_3 and SO_2 has been indicated in a field exposure. However, there are also other very complex interactions between O_3, SO_2 and also NO_x. The mechanisms for chemical interaction and degradation mechanisms for the materials will be dealt with more in detail in section D.4.2.5.

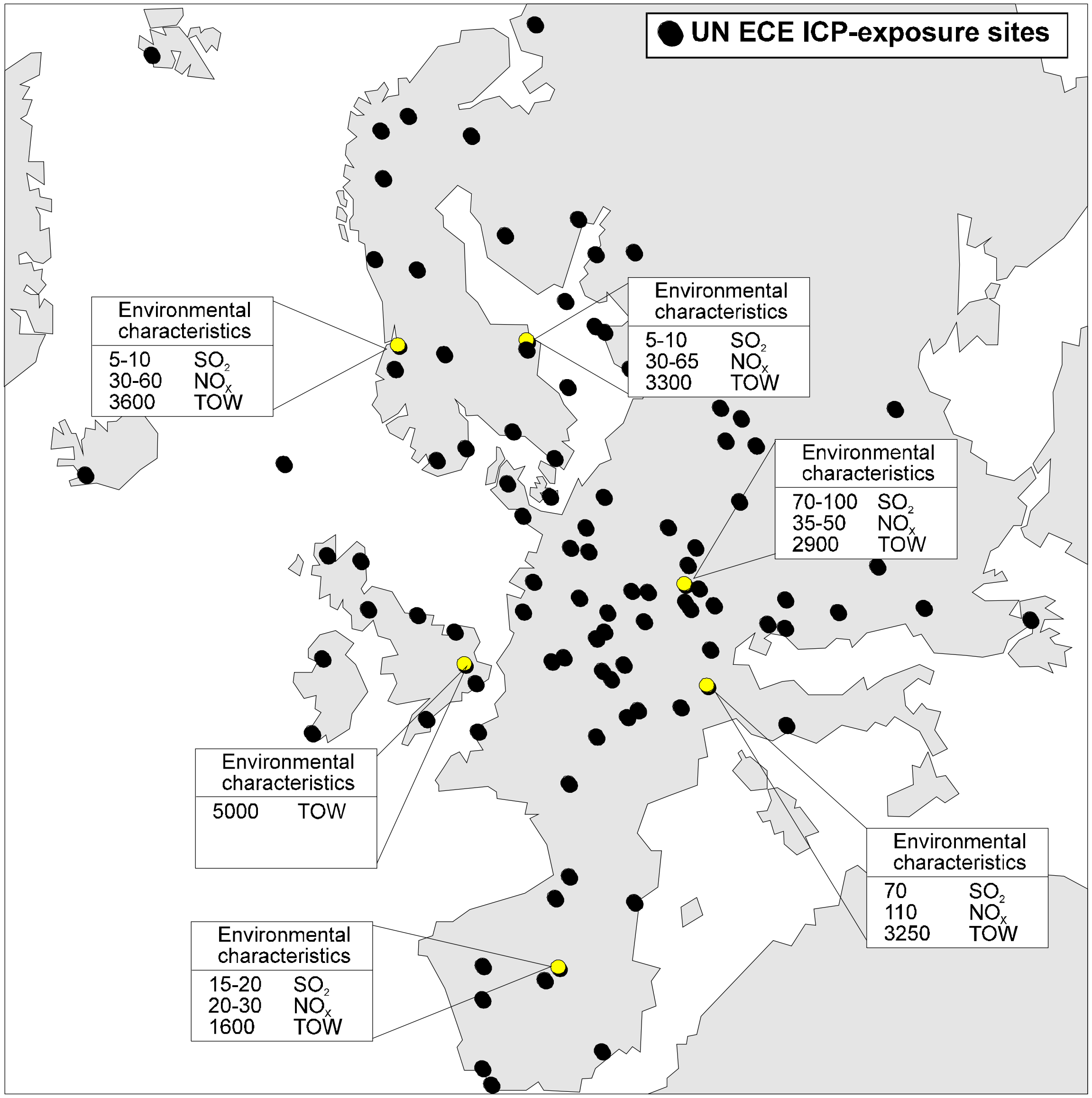


Fig C.1:1 UN ECE ICP sites (SO_2 and NO_2 (µg/m^3), TOW (h/year))

Table C.4:1 Universal dose-response functions for building materials [2]

Material	Equation	Corr.	No.
NAPAP [9] Data from eight test programmes comprising 72 test sites in many countries. In the dose-response functions recommended for assessment use, M_{Me} is mass loss (g/m^2), [H$^+$] the acidity (meqv/m^{2}year) based on 1 m of precipitation per year and f_x is the fraction of time above the critical relative humidity x. Assumed values are for the chloride concentration [Cl$^-$] 10 mg/m^{2}d and for the dust-fall (50 mg/m^{2}d).			
Zinc (galv. steel)	$M_{Zn} = (t^{0,779} + 0,0456\ln[H^+])(4,534 + 0,547 f_{85}[SO_2] + 0,0293[H^+])$		(3)
Copper	$M_{Cu} = 2,23 t^{0,805}(0,492\, f_{80}[SO_2] + 0,016[H^+] + 3,04)^{0,641}$		(4)
Aluminium	$M_{Al} = 0,203 t^{0,987}(0,099 + 0,139 f_{90}[SO_2] + 0,0045[H^+])^{0,870}$		(5)
Carbon steel	$M_{Fe} = 300,4(f_{90}t)^{0,541}\cdot(f_{90}[SO_2])^{0,257}\cdot[H^+]^{0,067}$		(6)
Lipfert [31] Dose-response function for carbonate stone developed from experimental data from nine field tests in USA and sites from other countries, where *ER* is the erosion rate (mass loss/m rain), [SO$_2$] and [HNO$_3$] the pollutant concentrations (μg/m^3), V_X^d the deposition rate (cm/s) of pollutant X, and *r,* the rainfall (m).			
Carbonate stone	$ER = 18,8 + 0,016 + 0,18(V_{SO_2}^d\,[SO_2] + V_{HNO_3}^d\,[HNO_3])/r$		(7)
Scandinavia [10] Eight-year exposure of carbon steel and zinc at 32 test sites in rural, urban and marine environments. Dose-response functions developed by linear regression analysis, where C_{Me} is the reduction of thickness (μm), [SO$_2$] (μg/m^3) and [Cl$^-$] (mg/m^{2}d) are the pollutants concentrations, and *t* the time.			
Carbon steel	$C_{Fe} = t^{0,59}\,(0,77[SO_2] + 0,42[Cl^-] + 14)$	$R = 0,94$	(8)
Zinc	$C_{Zn} = t(0,023[SO_2] + 0,011[Cl^-] + 0,53)$	$R = 0,89$	(9
Haagenrud et al. [32] The corrosion in urban Sarpsborg/Fredrikstad area (Norway) was modelled and mapped in a four year exposure programme at 13 sites with specimen withdrawal monthly, quarterly, yearly and after two and four years. Dose-response functions for yearly corrosion developed by linear regression analysis, where K_{Me} (g/m^2/year) is the corrosion rate and *TOW* (h/year) the time of wetness (hours per year with $T > 0°C$, $RH \geq 80$ %).			
Carbon steel	$K_{Fe} = 7,6[SO_2] + 172,0$	$R = 0,93$	(10)
Zinc	$K_{Zn} = 0,35[SO_2] + 0,01\,TOW - 21,8$	$R = 0,94$	(11)
Copper	$K_{Cu} = 0,14[SO_2] + 0,007\,TOW - 16,9$	$R = 0,91$	(12)
Aluminium	$K_{Al} = 0,01[SO_2] + 0,00053\,TOW - 1,2$	$R = 0,78$	(13)
Feliu and Morcillo [11] Data from 28 countries comprising 250 test fields in various parts of the world, where *C* is the corrosion depth (μm), *t* the time (years), [Cl$^-$] and [S] are the yearly average deposition rates (mg/dm^2/d), *T* the yearly average temperature (ºC), *D* the number of days/nights of precipitation, *RH* the yearly average relative humidity (%) and *R* the multiple correlation coefficient.			
Carbon steel	$C_{Fe} = At^n$, where $A = 33,0 + 57,4[Cl^-] + 26,6[S]$ $n = 0,57 + 0,0057[Cl^-]T + 7,7\cdot10^{-4}D - 1,7\cdot10^{-3}$	 $R = 0,73$ $R = 0,40$	(14)
Zinc	$C_{Zn} = At^n$, where $A = 0,785 + 5,01[Cl^-] + 2,26[S]$ $n = 0,53 + 0,55[S](1-0,068T) + 0,025T$	 $R = 0,73$ $R = 0,62$	(15)
Copper	$C_{Cu} = At^n$, where $A = 1,03 + 1,45[Cl^-] + 2,00[S]$ $n = 0,82 - 0,068T(1-0,011RH)$	 $R = 0,60$ $R = 0,47$	(16)

Table C.4:1 cont.

Material	Equation	Corr.	No.
UN ECE ICP [12] Exposures at 39 sites in 12 countries in Europe, USA and Canada. Dose-response functions below, where ^{4}ML and ^{4}MI are mass loss and mass increase (g/m^2), respectively, while ^{4}CD is the spread of damage from scratch in coatings, all after four years of exposure, *TOW* is the time of wetness (RH>80 %, *T*>0°) as time fraction of a year, [SO$_2$] and [O$_3$] are the concentrations (µg/m^3), [Cl$^-$] is the concentration (mg/l), *Rain* is the precipitation per year (m/year) and [H$^+$] is the concentration (g/l).			
METALS Weathering steel			
Unsheltered	$^{4}ML = 85 + 0{,}26TOW + 432TOW$	$R = 0{,}67$	(17)
Sheltered	$^{4}ML = 106 + 0{,}54TOW\,[SO_2][O_3]$	$R = 0{,}59$	(18)
Zinc			
Unsheltered	$^{4}ML = 14{,}5 + 0{,}043TOW\,[SO_2][O_3] + 80Rain[H^+]$	$R = 0{,}83$	(19)
Sheltered	$^{4}ML = 5{,}5 + 0{,}013TOW\,[SO_2][O_3]$	$R = 0{,}72$	(20)
Aluminium			
Unsheltered	$^{4}ML = 0{,}85 + 0{,}028TOW\,[SO_2][O_3]$	$R = 0{,}49$	(21)
Sheltered	$^{4}ML = -0{,}03 + 0{,}053TOW\,[SO_2][O_3] + 74[Cl^-]$	$R = 0{,}47$	(22)
Copper			
Unsheltered	$^{4}ML = 19{,}3 + 0{,}011[SO_2][O_3] + 162Rain[H^+]$	$R = 0{,}49$	(23)
Bronze			
Unsheltered	$^{4}ML = 11{,}8 + 0{,}047TOW\,[SO_2][O_3]$	$R = 0{,}59$	(24)
Sheltered	$^{4}ML = 5{,}3 + 0{,}024TOW\,[SO_2][O_3]$	$R = 0{,}52$	(25)
STONE Limestone			
Unsheltered	$^{4}ML = 34{,}4 + 5{,}96TOW\,[SO_2] + 338Rain[H^+]$	$R = 0{,}66$	(26)
Sheltered	$^{4}MI = 2{,}536 + 0{,}80TOW\,[SO_2][O_3]$	$R = 0{,}40$	(27)
Sandstone			
Unsheltered	$^{4}ML = 29{,}2 + 6{,}24TOW\,[SO_2] + 480Rain[H^+]$	$R = 0{,}63$	(28)
Sheltered	$^{4}MI = 2{,}84 + 0{,}88TOW\,[SO_2]$	$R = 0{,}69$	(29)
PAINTED COATINGS	$^{4}CD = -6{,}1 + 0{,}18[SO_2] + 0{,}18[O_3],$	$R = 0{,}35$	(30)

C.5 COMPARISON OF A RANGE OF DOSE-RESPONSE FUNCTIONS REPORTED FOR ZINC

A comparison of the different published dose-response functions for each type of material will determine the reliability of the functions and the decay rates calculated. However, comparison of dose-response functions is not straightforward as the units and forms of the variables and therefore the functions are different.

Table C.5:1 shows a set of recommended functions for zinc, taken from Table C.4:1. To allow a useful comparison, Zn-losses using Oslo as an example have been calculated with the following set of environmental variables:

- Cl$^-$ deposition 10 mg/m^{2}d
- dust deposition 50 mg/m^{2}d
- rain 0,6 m/year
- [H$^+$] 0,025 mg/l (15 meqv/m^{2}year)
- *pH* 4,5

- f_{85} 0,23 (2000 h/year)
- TOW 0,32 (2785 h/year)
- T_{year} 8°C
- $[SO_2]$ 1) 0 µg/m^3
  2) 10 µg/m^3 (12,5 mg/ m^{2}d)
  3) 100 µg/m^3 (125 mg/m^{2}d)

Table C.5:1 Comparison of a range of dose-response functions reported for zinc

Study	Zinc Loss equation
1. Lipfert et. al. [9]	$M_{Zn} = (t^{0,779} + 0,0456\ln[H^+])(4,534 + 0,547f_{85}[SO_2] + 0,0293[H^+])$
2. Saunders [33] (from Shaw data for average UK conditions)	$M_{Zn} = 1,806 + 0,022[SO_2]$
3. Haynie [34]	$M_{Zn} = 2,36 + 0,022[SO_2]$
4. Atteraas & Haagenrud [35]	$M_{Zn} = 1,58 + 0,078[SO_2]$
5. Mikailovskii [36] (assuming temperature of 15°C)	$M_{Zn} = 3,92 + 0,067[SO_2]$
6. Kucera et al. [10]	$M_{Zn} = t(0,023[SO_2] + 0,011[Cl^-] + 0,53)$
7. UN ECE [12]	$^4ML = 14,5 + 0,043TOW\,[SO_2][O_3] + 80Rain[H^+]$
8. Feliu & Morcillo [11]	$C_{Zn} = (0,785 + 5,01[Cl^-] + 2,26[S])t^{0,53 + 0,55[S](1-0,068T) + 0,025T}$

Fig C.5:2 displays the results, and average values of dose-response functions for Zinc in Oslo.

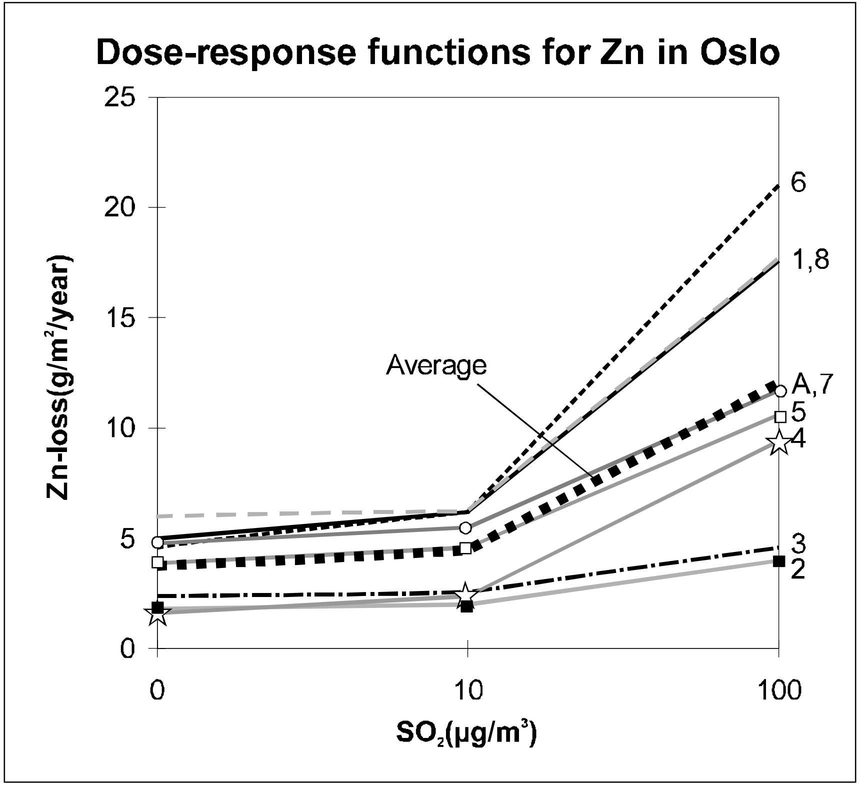


Fig C.5:2 Comparison of different dose-response functions published for zinc

D Characterisation of Key Environmental Degradation Factors

D.1 INTRODUCTION

This section provides basic knowledge and characterisation of principal agents affecting the durability of building materials. In principle it follows the detailed classification in the standard ISO 6241–1984 (E) [13] of agents relevant to building performance and requirements. The degradation factors are classified according to *their nature* as *Mechanical, Electro-magnetic, Thermal, Chemical and Biological agents*, and to *their origin* (external-internal to the building, atmosphere, ground etc.). The same classification is used in this report for *outdoor atmospheric exposure environments.*

This classification implies that the agents are *listed according to their own nature and not to the nature of their action* on the buildings or components; for example, a thermal agent may have a physical action (for example thermal expansion) or a chemical action (for example catalysis); a chemical agent like water may have a physical action (for example swelling) or a chemical action (for example hydration dissolution); moreover the agents in combination may have additional physical actions (for example wetting followed by freeze-thaw cycles), chemical actions (for example photo-oxidation by atmospheric oxygen and solar radiation) or biological actions (for example spread of roots at high humidity).

The standard thus provides lists the agents to be taken into account when defining performance and specifying requirements. The agents that apply in any particular situation, and their magnitudes, will depend on the building’s situation, form, intended use and the way it is designed to perform.

In other words, in order to be able to characterise and report the right *type* and *form* of the environmental degradation factors, they have to be related to the degradation mechanisms and dose-response functions for the specific material and material characteristic in question.

Reference is made to the survey in section C, and when possible there is an explanation of the process that takes place and examples to illustrate the data needed.

D.2 MECHANICAL AGENTS

D.2.1 Snow loads, rain and water loads

The most significant action on buildings of water in solid form, i.e. snow (or ice), is structural loading on horizontal surfaces. The effect of snow on walls is minimal unless when drifting takes place. Problems can sometimes occur when snow is blown into buildings, particularly into roof-spaces, through small holes. The availability of data on snow is sparse since snowfall is normally measured as equivalent rainfall [37].

Precipitation in the form of hail is seldom in most parts of the world and may, apart from the rare instances of impact damage from exceptionally large hailstones, be regarded in the same category as rain. In the US, however, hail damage is a major source of damage to asphalt shingled roofs. The annual damage to these shingles is approximately one thousand million dollars per year [38].

Publications are available that contain data on creep and fatigue factors for most traditional building materials under loading. However, for many plastics and other modern composite materials, manufacturers may need to carry out tests on specific building components. When considering these factors, the expected design life for the structure must be taken into account. Tall reinforced concrete structures are particularly subject to contraction in height during their early life, not only from shrinkage but also from creep under self-loading. Sufficient forces may consequently develop in infill clay brickwork or cladding, or rendering applied to them, eventually causing cracking or loss of adhesion. Other examples of long-term movements that may lead to damage to adjacent construction are the expansion of clay bricks and butted floor and wall tiles due to absorption of moisture.

D.2.2 Ice formation pressure, thermal and moisture expansion

D.2.2.1 Daily temperature difference

All external construction materials experience diurnal, seasonal and annual temperature fluctuations [16,39,40]. Building materials are heated by sunshine in the daytime and are cooled by thermal emission from the material at night. The fluctuations of temperature induce movements in the joints between building elements or cracks in concrete. Sealants and exterior finishing materials suffer from repeating tensile and compressive and shear stresses and are fatigued sufficiently as to deteriorate.

One special temperature variation that may greatly affect, for example, the micro cracking of coatings, is rapid drop in surface temperature. A dark coating on an insulated sheet metal facade with a surface temperature of approximately 70°C, when exposed to cold heavy rainfall may fall 50-60°C in temperature in a few minutes.

The majority of available temperature data originates from measurements of the ambient atmospheric temperature. From measurements at meteorological stations, data are readily available on mean temperatures, temperature extremes and differences between maximum and minimum temperatures for specified periods of time. However, the relations between data on ambient temperature, surface temperature and temperature in the bulk of materials are very complex.

The *daily temperature difference* of building elements can be treated as a *mechanical deterioration index*. Tomiita [14] collected daily maximum and minimum BPTs (Black Panel Temperatures), T_{pmax} and T_{Pmin} (°C), respectively, and expressed them as functions of climatic data:

$$T_{\mathrm{Pmax}} = 1{,}143T_{\mathrm{max}} - 2{,}244W_{12} + 0{,}928S_{\mathrm{D}} + 5{,}214 \qquad (31)$$

$$T_{\mathrm{Pmin}} = 1{,}049\ T_{\mathrm{min}} + 0{,}022W_{\mathrm{N}} - 2{,}389C_{1} - 3{,}056 \qquad (32)$$

where T_{max} is the daily maximum ambient temperature (°C), W_{12} the wind velocity at 12.00 hours (m/s), S_{D} the daily total solar energy (MJ/m^{2}d), T_{min} the daily minimum ambient temperature (°C), W_{N} the daily average wind speed at night (m/s):

$$W_{\mathrm{N}} = (W_3 + W_6/2 + W_{18}/2 + W_{21} + W_{24})/4$$

and C_1 the clearness index (the ratio of total daily solar radiation to extra terrestrial solar radiation).

By inserting the meteorological data observed at 66 points during 1976-1985 into eqs. (31) and (32), the daily maximum and minimum BPTs were estimated. The yearly averages of the daily differences of BPTs are then classified and mapped for Japan (section F.4.1). By multiplying the length of a building element by its thermal expansion, the daily movement in the joint or crack can be calculated.

D.2.2.1.1 Freeze/thaw

When water freezes it expands, and when it thaws the effects of this expansion can become apparent in the form of frost damage.

Frost damage occurs when there is sufficient water entrapped in the pores of the material to freeze. The necessary conditions therefore include wetting (tending towards saturation) and freezing.

The wetting may be due to rainfall, but melt-water from partial thaw of snow or ice is a common precursor to damage, as it is likely to saturate the pore structure of the material and to start freezing. The air temperature at which frost damage occurs can be substantially below freezing, since temperatures just below zero may not freeze water in small spaces, and the temperature of the material is often higher than the surrounding air. For this reason the number of *zero transitions (air temperature changes passing through 0°C*) *is not likely to give an accurate indication of the number of freeze thaw cycles in the material.* Zero transitions may, however, provide a useful means of ranking the frost resistance of porous materials.

Recent research on the risk of frost damage to exposed concrete has found the following factors were more critical than either the air temperature at the time of thawing, or the duration of the frost:

1. Severity of frost (lowest air temperature);
2. Speed for freezing (rate of fall of air temperature, and therefore rate of thermal change in the material);
3. Change of air temperature during periods of frost; and
4. Number of frost cycles (i.e. daytime thaw after frost at night).

Data on freeze-thaw cycles may be used as indicators of the risk of frost damage to porous materials. Organic coatings may also be affected through movements in ice layers on the coating surface and by water freezing in cracks. As the thermal expansion coefficient of most polymeric systems is 10 fold greater than that of most metals, the volumetric contraction of a coating would greatly increase the tensile load within the coating, thereby fostering cracking [38].

In the UK there are at present *two approaches* to identifying sites susceptible to frost damage. One approach classifies a site as severely exposed to frost when all of the following factors apply [41]:

1. The average annual frost incidence is in excess of 60 days;
2. The average annual rainfall is in excess of 1000 mm; and
3. The altitude of the site is in excess of 91 m above sea level. (This factor is likely to reflect the greater liability to wind driven rain on higher ground).

The other approach is based on driving rain only. It recommends that suitable grades of mortar and bricks or blocks should be used in areas subject to severe driving rain, as defined in the map of the all direction driving rain index drawn from BS 7543: 1992 [42] (see fig D.2:1).

In both approaches the saturation of material by rain is an important factor. Frost damage can be reduced by protecting a structure from saturation by detailing projecting eaves, cappings etc.

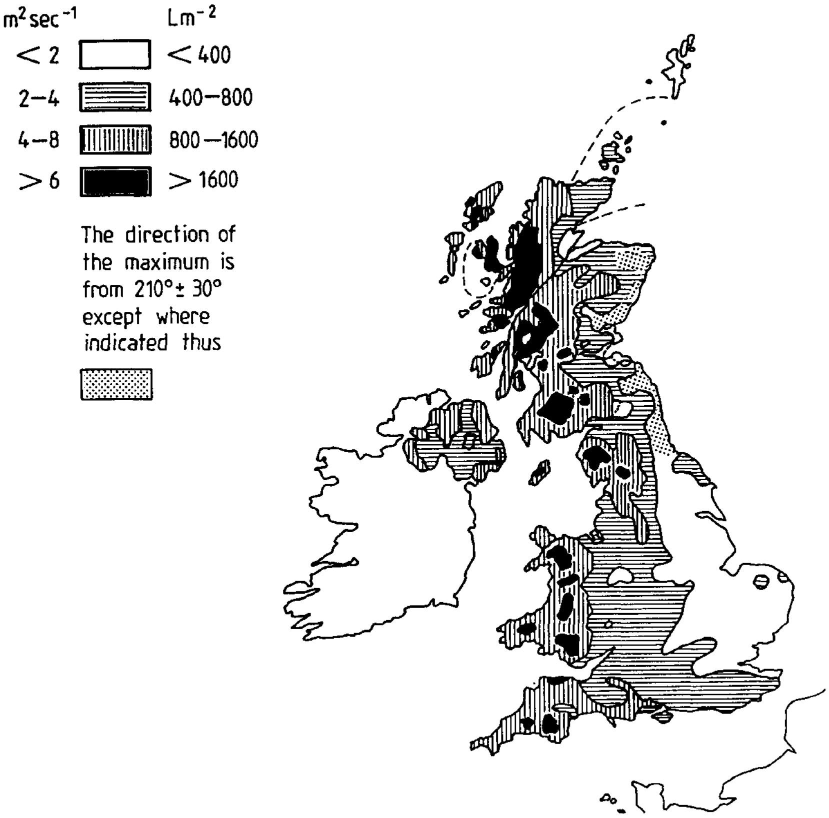


Fig D.2:1 Maximum directional annual driving rain index map for UK

D.2.2.2 Moisture

The presence of moisture is likely to enable physical, chemical and/or biological degradation reactions to take place. Different processes occur:

D.2.2.2.1 Driving rain

The quantity of water falling on the vertical faces of buildings is related to the combined effects of wind and rainfall [43], (see fig D.2:1).

Most organic materials and many inorganic materials absorb moisture to varying degrees. The direct effect of water alone on a building part can be as follows:

1. Volumetric expansion;
2. Change in mechanical properties (e.g. strength);
3. Development of bending and twisting forces;
4. Change in electrical properties;
5. Change in thermal properties; and
6. Change in appearance.

Quantitative dose-response functions exist between driving rain and some of these functional characteristics. In Germany 140 l/m^2 is given as the maximum amount of driving rain that a timber-framed houses can withstand without special protection being necessary for sealants [44].

D.2.2.2.2 Differential wetting or drying

Internal forces may be generated within a construction as a result of differential wetting or drying or from variations in moisture characteristics of the in-going materials. Consideration should be given to dimensional tolerances to avoid such development.

Examples of the water absorptivity of a material are given in percentage mass fraction:

1. Bricks can absorb up to 30 %. However, it should be noted that there is no direct relationship between water absorptivity of a brick and its durability [45,46].
2. Natural stone and dense concrete can absorb up to 10 %.
3. Nylon can absorb 2 - 10 %.
4. Glass reinforced plastic can absorb approximately 0,5 %.
5. Wood – the maximum amount of moisture absorbed by wood varies according to species. However, in general terms wood swells as it takes up moisture to it’s maximum content. This maximum is the content beyond which any additional moisture is accumulated in the cell lumina as free water. Maximum moisture content is highly dependent on the specific gravity of the wood. After it has reached it’s maximum water content, more moisture will still be absorbed but the wood will remain dimensionally stable. Sapwood can absorb its own weight in water (see section D.5.1.2).

For quantitative dose-response functions reference is made to section C, where time of wetness is shown to be a significant variable for the degradation of many materials. In D.4.1.1 this very important variable is defined and discussed in detail.

On the macro and meso scale, general meteorological data on the various forms of water are available from national meteorological institutes.

D.2.3 Wind

The major considerations relating to wind loads and durability include:

1. Effect of driving rain and incursion of particulate solid matter;
2. Effect of differential pressures that may be developed in localised positions on a structure; and
3. Catastrophic failure.

The former is a major cause of erosion and rain penetration into the vertical structures.

Differential pressure may be built up in large areas such as roof coverings, cladding panels and glazing. This may cause deflections or loosening from fixings. It can also affect flashing, joints and seals.

Only generalised data relating to wind loads can be obtained. Local topography, including the influences of nearby structures, especially tall buildings, considerably affects the direction of the local airflow, and there are wide variations in wind speed over periods of time. Generalised data are available from the meteorological community.

BRE Digest 346 [47], which is in seven parts deals with different aspects of wind loads, includes, in Part 7, fatigue loads caused by oscillation of a structure by the wind and fatigue due to the repeated gusts.

Examples of other intermittent stresses include earthquakes and man-made vibrations such as those due to road traffic, trains, music, dancing, and machine induced vibrations.

D.2.4 Use factors (wear and tear, abuse)

W*ear and tear* is abrasion resulting from traffic, the action of detritus and contact with moving parts.

The severity of abrasive action is dependent on the nature of the abrasive material, its quantity and the influence exerted by the agency bringing it into contact with the building. The cleanliness of the environment also has an important effect. Abrasion caused by the combined action of traffic and detritus is considerably greater than that of traffic alone.

What is considered normal wear and tear for one type of building may be seen as untypical or excessive for another. For walls and flooring materials there are recognised grades that will withstand the expected levels of use, washing down etc., that are likely to occur in particular situations. For example, The British Carpet Manufacturers’ Association use a six level classification from “extra heavy wear” to “light wear” and the UPEC classification used by the British Board of Agreement for thin plastics floorings identifies four separate properties: resistance to wear, to indentation, to water and to chemicals.

The frequency and nature of cleaning and maintenance needs to be assessed in relation to the likely consequence on the service life of a component, particularly in cases such as:

1. Masonry, where the actual cleaning procedure may lead to deterioration;
2. Plastics surfaces that can be scratched by abrasive cleaning materials, e.g. unsuitable graffiti-removal processes and solvents; and
3. Porous masonry when a coating of impermeable paint can lead to increased moisture contents and consequently to frost damage.

Uninformed and uncaring use of a building can accelerate its deterioration. An example is condensation caused by the combination of blocking of ventilation grilles and the excessive quantities of water vapour produced by the heavy use of liquid petroleum gas (LPG) room heaters, cooking and washing equipment.

With some components in certain types of buildings (e.g. school windows) breakage has to be considered as part of normal wear and tear. BS 6262 [48] gives the extra precautions to be taken in areas where there are added risks to children, whose more active behaviour is likely to lead to more frequent accidents.

D.3 Electromagnetic agents

D.3.1 Solar radiation

The energy that reaches the earth's surface from solar radiation is concentrated in certain wavebands. Fig D.3:1 shows the variation in the energy received at different wavebands from direct sunlight. A portion of solar radiation is absorbed or reflected by the earth's atmosphere. This portion varies from one waveband to another and is affected by cloud cover. The degree of shading from direct sunlight also affects the quantity of the radiation received in each waveband.

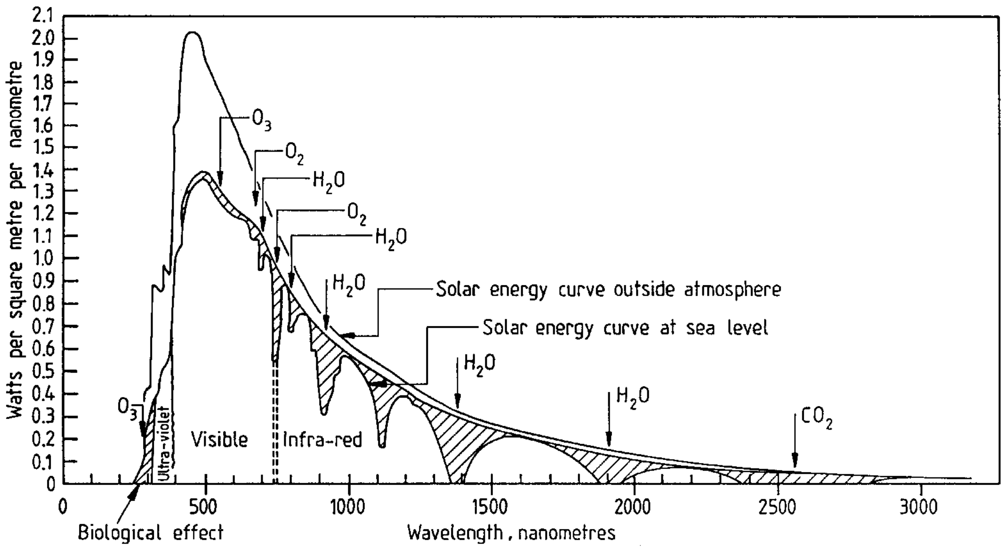


Fig D.3:1 Curves showing the distribution of energy in the solar spectrum outside the atmosphere and at sea level [42]

D.3.1.1 Infrared radiation (approximately 700 nm to 3000 nm)

Infrared radiation is absorbed to some extent by all forms of matter and this causes an increase in temperature, thus creating surface temperatures greater than that of the surrounding air (in some cases markedly so). The opacity, texture and colour of a surface involved have considerable effect on the amount of radiation absorbed. The absorbed radiation will raise the temperature of a material by an amount dependent upon the specific heat and thermal conductivity of the material and the structure behind (see D.2.2.1.1).

For a given surface texture the colour of a surface considerably affects the absorption of solar radiation. An assumed, perfectly black surface has an absorptivity of 100 %, whereas the absorptivities for the following real colours are [42]:

1. Ordinary black 80 %
2. Dark green or grey 70 %
3. Light green or grey 40 %
4. White 12-20 %

D.3.1.2 The visible waveband (400 nm to 700 nm)

There is no distinction made in most published data about the spectral absorption of visible light. The majority of published information concerns solar radiation measured either as bright sunshine or as total radiation. For most purposes the intensity of solar radiation on clear cloudless days is taken as the significant design value but it should be remembered that there would be considerable variation in the total radiation received at a given place on the ground depending upon:

1. Cloud cover both in type and quantity;
2. Season of the year; and
3. Local topography.

Peak solar radiation is likely to occur around noon inland, and during the late morning at the coast. Surfaces normal to midday summer sun, or those receiving substantial reflected radiation from adjacent surfaces, experience the highest temperatures. A value of l.36 kW/m^2 ±3,5 % can be taken as the total radiation intensity at the mean distance of the earth normal to the sun, the variations being due to seasonal distance variation factors. This value ignores losses due to scattering by dust and water vapour particles and by the earth's atmosphere that will significantly reduce the value for a large proportion of time. In approximate terms, the radiation received on an overcast day is about one third of that received if it had been cloudless. On very dull days the proportion would be much less.

D.3.1.3 Ultraviolet radiation (290 nm to 400 nm)

A large proportion of the ultraviolet (UV) waveband is scattered by the atmosphere, the remainder reaching the earth's surface has no adverse effect on inorganic materials. However, UV radiation is significant by its potential for deterioration of some organic materials. Although UV radiation constitutes no more than 1-7 % of the total solar radiation intensity, in practice it decides the service life of polymeric materials in outdoor use. The penetrating power of the UV radiation is not great, and the action is consequently confined to surface layers exposed to radiation [49].

Measurements of total UV radiation over the spectral range are of limited value because of the sensitivity of materials to specific narrow wavebands. For instance, polystyrene shows a maximum sensitivity at about 318 nm whereas polypropylene peaks at about 370 nm. As a general rule it has been found that radiation shorter than about 360 nm tends to cause yellowing and embrittlement. Radiation of longer wavelength tends to cause fading [50].

Many organic dyestuffs are degraded by UV radiation, which may affect appearance.

Bituminous materials and some other polymers are also degraded by UV radiation, and the changes of surface properties may alter the performance of the material.

UV radiation can also play an important role in initiating a degradation reaction which is then propagated under suitable conditions of moisture and temperature, for example the yellowing and surface denudation of glass fibre reinforced polyester roofing sheets caused by the combined actions of UV radiation and moisture.

The UV intensity varies with the atmosphere, local weather, air pollution, time of day and other factors. Due to the environmental ozone-UV problem there are now extensive intensity measurements being carried out on a global scale (see E.2.4).

Estimates of the UV environment for a certain geographic location can be made from meteorological data on global solar radiation or number of sunshine hours.

Tomiita [14] has used a photodiode sensor (range 305-390nm) to measure the hourly solar UV energy on a horizontal plane in an outdoor exposure in Japan for a year. This was expressed as a function of the entire range of solar radiation, observed by a pyranometer, and solar altitude as follows:

$$U_{\mathrm{hour}} = -\,0{,}894 Z^{-0{,}229} S_{\mathrm{hour}} + 0{,}796 Z^{-0{,}165} S_{\mathrm{hour}}^{0{,}968} \tag{33}$$

where:

U_{hour}	hourly solar UV energy (MJ/m^{2}h)
Z	solar altitude (degrees)
S_{hour}	hourly entire range of solar radiation (MJ/m^{2}h)

By applying this equation to the hourly solar energy calculated from the percentage of sunshine at 141 points in Japan, the hourly solar UV energy was estimated and the yearly solar UV energy integrated. The results were classified and mapped (see section F.4.1).

Tomiita also produced quantitative dose-response functions for some polymeric materials, using the service life methodology with short- and long-term exposure to estimate the constants in the functions. This would be a very valuable approach on a broad basis, but at present there is a considerable gap of knowledge concerning quantitative dose-response functions for solar radiation influences on the performance characteristics of various types of materials and components.

Martin et al. [16] similarly proposed the following metric for UV dosage, quoting: “Ultraviolet radiation is deleterious to most organic materials. The medical, biological, and agricultural communities characterise the degradative effects of ultraviolet radiation using a different approach from the one used in the coatings’ community. They relate the degradative effects directly to total effective UV dosage, D_{tot}, whereas the coatings’ community tends to use total UV irradiance. The total effective UV dosage is defined by:

$$D_{\mathrm{tot}}(t) = \int_{0}^{t} \int_{\lambda_{\mathrm{min}}}^{\lambda_{\mathrm{max}}} E_0(\lambda, t')\left(1 - e^{-A(\lambda)}\right)\Phi(\lambda)\,\mathrm{d}\lambda\,\mathrm{d}t' \tag{34}$$

where:

λ_{min} and λ_{max}	Minimum and maximum photolytically effective wavelengths, respectively
$E_0(\lambda,t)$	UV spectral irradiance to which the material is exposed at time t (W/m^{2}nm)
$1–e^{-A(\lambda)}$	Spectral absorption of a material (dimensionless)
$\phi(\lambda)$	Spectral quantum yield of the material (dimensionless)

Thus, the total effective UV dosage is computed by integrating the product of the spectral irradiance, $E_0(\lambda,t)$, the spectral absorption coefficient, $1-e^{-A(\lambda)}$, and the UV solar spectral efficiency of the absorbed radiation, $\phi(\lambda)$, over both the range of photolytically effective wavelengths and the duration of the exposure. Experimentally, the spectral absorption and the spectral quantum yield coefficients are determined from laboratory-based experiments. The spectral irradiance measurements are monitored in both the laboratory and field. The total effective dosage, D_{tot}, can then be related to biologic damage using a damage function. The possible application of this approach to coatings has been discussed by Martin et al. [16] and some preliminary analyses of spectral UV irradiance measurements have been performed."

D.3.2 Thermal radiation

Heat accelerates the degradation of materials. By using the service-life approach, Tomiita [15] has expressed the change in a property of a building material as a function of temperature, utilising the following dose-response function based on the Arrhenius equation:

$$\ln\left(P / P_0\right) = C_{\mathrm{H}} t e^{-E_{\mathrm{H}}/\mathrm{R}T}$$

where P is the current property of the material, P_0 the initial property of the material, C_{H} the thermal deterioration constant of the material, E_{H} the activation energy of thermal deterioration (kJ/mol), R the gas constant ($8{,}314 \cdot 10^{-3}$ kJ/mol K), T the absolute temperature of the material (K), and t the elapsed time.

By various assumptions Tomiita has been able to calculate the degradation curve as a function of solar radiation energy, S_{D}, daily average wind speed, W_{D}, average daily temperature in the daytime, T_{D}, and night-time, T_{N}, and the daily equivalent black panel temperature, *BPT*. The percentage degradation over a ten-year period has been mapped for the various parts of Japan (see section F.4.1).

Quoting Martin et al. [16], who proposed to compute the panel temperature for coated panels: "In laboratory experiments, the temperature of the coated panels can be monitored directly by outfitting each panel with a thermocouple or indirectly by controlling the exposure conditions. Obviously, such methods would be expensive if the number of specimens was large, as at many commercial outdoor exposure sites. A more economical and practical approach to monitoring panel temperature, but one that is dependent on the availability of adequate models, would be to determine the thermal properties of a coating system before its exposure in its expected in-service environment. Additionally, common meteorological variables like ambient temperature, sky temperature, background temperature, wind speed, and total solar radiation during its exposure need to be monitored. Then the panel temperature for all the coated panels at an exposure site can be computed by solving for T_{p} for each panel using the following energy balance equation:

$$AH_{\mathrm{t}}\alpha = 2hA\left(T_{\mathrm{p}} - T_{\mathrm{a}}\right) - \varepsilon A\sigma\left(T_{\mathrm{s}}^4 - T_{\mathrm{p}}^4\right) - \varepsilon A\sigma\left(T_{\mathrm{b}}^4 - T_{\mathrm{p}}^4\right) \tag{36}$$

where:

A	the surface area of a panel (m^2)
H_{t}	total solar radiation (W/m^2)
α	absorptivity of the coating
h	convection coefficient (W/m^2K), which is a function of wind speed
σ	Stefan-Boltzmann constant that is $5{,}67 \cdot 10^{-8}$ W/m^2K^4
T_{a}	ambient temperature (K)
T_{b}	background temperature (K)
T_{p}	panel temperature (K)
T_{s}	sky temperature (K)
ε	panel emissivity

The feasibility of this approach has been demonstrated [51].

Accurate techniques to measure surface temperature on exposed plates have been developed [52]. Although detailed analysis is not yet complete, preliminary results indicate that generally a plate is not in thermal equilibrium with the environment. Compared to the ambient temperature the surface temperature of the plate can be appreciably lower during the night and the early morning and appreciably higher during the day, the deviation depending on the material and the climatic conditions. The implication of these results is that the state of wetness on plates is as much controlled by the surface temperature as by ambient conditions.

D.4 CHEMICAL AGENTS

D.4.1 Water and temperature-humidity, condensation, precipitation

D.4.1.1 Time of wetness or critical wetness for deterioration

The term “Time-of-Wetness” (TOW) is very much used, and often misleadingly so. To avoid misunderstanding, the original definition, originating from the field of atmospheric metal corrosion, should be remembered. Here TOW is defined as the *period of time* during which the material surface is subjected to *enough moisture for the corrosion rate to be significant*. This leads to the concept of a *time above a critical wetness, i.e. that both the level and duration of wetness is important.*

Therefore, to avoid misinterpretation, the term should generally include the word “critical”, and perhaps be called "the time of critical wetness" (TOW_{crit}), and should be defined as a material specific property.

The total time of wetness, *TOW*, is described as the time during which the surface is covered with a few adsorbed (possibly incomplete) monolayers, T_{ads} (below 80 % RH), and/or the time during which there is a substantial moisture film on the surface, T_{pha}.

$$TOW = T_{ads} + T_{pha}$$

Table D.4:1 shows the relationship between the occurrence of various humidity events in ambient air and the thickness of the wetness film on the material surface.

Table D.4:1 Various humidity conditions and resulting wetness film thickness on the material surface

Humidity event	70 % RH	85 % RH	100 % RH	dew	rain
Wet film thickness (μm)	0,01	0,1	1	10	100

Now, depending upon the material critical wetness properties, the TOW_{crit} may contain only the T_{pha} term if pronounced wetness is needed, and so on. It also implies that a material can have different TOW_{crit}-values, depending upon the performance characteristic or degradation indicator in question (see also section B.1).

For metals the corrosion is caused by electrochemical reactions on the surface, where the formation of an electrolyte is essential. This formation depends on humidity and the pollutants available, as illustrated in fig D.4:1 for carbon steel. In extremely clean laboratory conditions, 100 % relative humidity (RH) and dew are needed to initiate the corrosion process. In practice the electrolyte will be formed at lower RH due to pollutants. In areas where gas pollutants like SO_2 are present, a critical humidity level of 80 % for corrosion is observed, and in areas having sea-salt aerosols present in the air, the critical humidity level will be even lower.

From this figure it is also obvious why Lipfert et al. in their reconciliation process found best correlation with various expressions of the time of wetness [9], (see section C.1). Practically all dose-response functions contain a term expressing the time of wetness, or more precisely, the *time above critical wetness*.

From empirical observations, the term TOW for metals is defined as the time when the relative humidity is greater than 80 % at temperature above 0°C. This definition is used in ISO standard 9223 "Corrosion of metals and alloys – corrosivity of atmosphere – classification" [18], where the TOW is calculated together with selected climatological characteristics of the macro-climatic zones of the world. This classification is extracted from the standard IEC 60721-2-1 [29], (see section F.2).

The level and duration of moisture in the immediate vicinity of the material surface is also crucial to the degradation of *non-metallic materials*, if they are prone to deteriorate under the influence of moisture. Condensed moisture on coating surfaces during longer periods of time is, for example, considered to have a more deteriorating effect than shorter periods of rain [49,54].

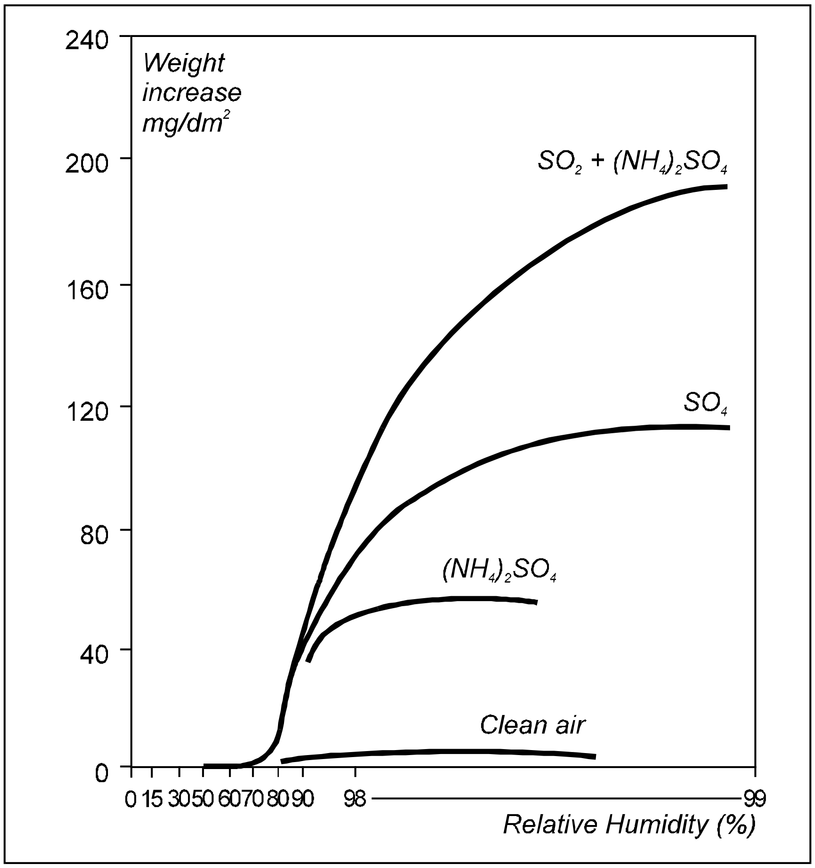


Fig D.4:1 Corrosion attack on carbon steel at increasing relative humidity, based on work by W.H. Vernon [53]

The effect of humidity is also observed for *porous materials* such as wood, stone, rendering and concrete, underlining the extreme importance of the fact that the concept also accounts for the wetness inside porous materials. For example, for wood the degradation process of rotting takes place after a prolonged period of time above a moisture mass fraction of 15–20 %.

An extensive study of temperature and humidity in wall cavities of houses in Australia has been carried out [55,56]. Calculations of existing equilibrium moisture contents (EMC), and comparison with required EMC levels for growth of wood decay fungi were used to assess probable differences in material durability in the microenvironment.

D.4.1.2 Data on “time of critical wetness” – TOW_{crit}

Monitoring methods and data for TOW_{crit} are very important and should be the subject of extensive research. Data can be established indirectly from measurements of air temperature and relative humidity, or directly through measurements with electrochemical cells.

With the indirect method, TOW_{crit} is calculated as the time when the RH exceeds a certain level, normally 80 %, during periods when the temperature is 0°C or higher. The data on temperature and RH may originate from daily measurements, or even be monthly mean values. The TOW_{crit} according to the ISO criterion from available meteorological data have been calculated [14,32,57], as has the number of wet-dry cycles causing condensation [58,59]. These values have been classified and mapped for Japan (see section F.4.1).

In direct measurements electrochemical cells are used, attached to the material surface. Also, in direct measurements the TOW_{crit} depends on the predefined level of wetness expressed as a critical current in the cell [60].

D.4.1.3 Factors influencing TOW in the building envelop

In the actual microenvironment for constructions, TOW_{crit} will vary enormously. Consequently, there is a great need for developing monitoring methods for mapping the level and duration of wetness in the microenvironment for the various constructions and materials in use. The WETCORR instrument can be used for such a purpose [24,61], (see G.1.1).

In an Australian context, Cole [56] investigated the conditions in a number of wall cavities in the different climatic zones in Australia. These studies indicated that high humidity could occur in wall cavities, particularly in tropical and sub-tropical Australia, and that these high humidity levels gave rise to relatively high time of wetness values, which have the potential to promote corrosion. In further work, a more extensive study of the factors controlling building envelope microclimate in houses located in tropical and sub-tropical Australia was completed [25]. It was found that in non-marine locations, wetness within the building envelope was primarily controlled by condensation cycles. These cycles were promoted by temperature differences between different building spaces and between exterior surfaces and building envelope air spaces. The surface wetness (on sensors) within the building envelope of buildings located in areas of high airborne salinity was controlled by deposition of hygroscopic salts.

In an early work, a model of the mass transfer from a metal plate to the air was derived [62]. The application of this model to conditions in the building envelope and on the exterior demonstrated that while the time taken for a surface to dry following wetting was likely to be minimal in the open air, it could be significantly longer within the building envelope. Accordingly, the total TOW in the building envelope should be considered as the time of active wetting plus the time of drying, with these two components having similar magnitudes.

D.4.2 Oxidising agents

D.4.2.1 Oxygen

Photochemical oxidation of organic materials, such as coatings, is a synergistic degradation reaction where the agents are oxygen and sunlight. When an oxygen molecule in the ground state absorbs a photon with high enough energy, it may transfer to an excited state having a new electron configuration. The effects of oxygen in different states (molecular oxygen in the ground triplet state (O_2) and in excited singlet states (O_2^*), respectively, atomic oxygen (O) and ozone (O_3)) on the photochemical degradation of polymers have been studied in several investigations. The following is a brief summary of the dominating photochemical degradation mechanisms [63], involving oxygen in different states and ozone, of common polymers such as polystyrene, polyvinyl chloride, polyethene.

D.4.2.1.1 Molecular triplet and singlet oxygen, and atomic oxygen

In the oxygen-rich atmosphere, in combination with UV radiation, photochemical oxidation of polymers occurs. The process propagates in two steps: first an absorption of UV quanta in the polymer, when double bonds are excited and free radicals formed, which in a second step react with molecular oxygen in its ground triplet state, resulting in formation of hydroxy, hydroperoxy, ketone and aldehyde groups. The reactions lead to chain scission and a molecular weight decrease affecting the technical and aesthetical properties of the polymer.

In the atmosphere the usual state of molecular oxygen is the ground triplet state. However, molecular oxygen in excited singlet states is also present through a number of photolytic processes: direct absorption in molecular triplet oxygen of sun-light quanta; photolysis of ozone under elevated pressure; and photolysis of air pollutants combined with an energy transfer between the pollutants and molecular triplet oxygen.

Molecular oxygen in singlet states is quite reactive by its own, and no further UV radiation is needed to oxidise a polymer (the photo excitation has already taken place but with the very oxygen molecule as the target). Otherwise the mechanism for photo-oxidation of polymers with molecular singlet oxygen as the reactive constituent is similar to that involving molecular triplet oxygen, leading to the formation of free radicals, hydroxy and peroxy groups, and chain scission of the polymer molecules.

Atomic oxygen forms naturally through photolysis of ozone. Photo-oxidation with atomic oxygen occurs by the formation of alkyl radicals, unsaturated chain ends and, finally, chain scission.

D.4.2.1.2 Ozone

Ozone is one of the most important oxidising constituents and reacts with many polymers such as polyethene, polybutadiene, polystyrene. The effects are normally discoloration and embrittlement. Ozone attacks double bonds in the polymer, causing chain scission and crosslinking reactions. The degradation reactions occur via the formation of peroxy radicals, which photochemically may be excited to other free radicals. The reactions take place on the material surface.

There are great variations in the occurrence of ozone in the atmosphere. At high altitudes, more than 20 km above earth level, the amount is more than one thousand times higher than at the earth level.

Ozone may exist in nature as an effect of solar irradiation of oxygen in the presence of air pollutants. This has caused major concern in the last few years, and evidence for damages to health and vegetation has triggered off a European Council Directive (92/72/EEC) for monitoring and warning of ozone. This is dealt with more closely in section E.4.

D.4.2.2 Carbon dioxide

In the presence of moisture, carbon dioxide will attack susceptible inorganic materials since it forms a slightly acidic solution.

Atmospheric carbon dioxide is a major cause of deterioration in exposed reinforced concrete structures. The penetration of the concrete by carbon dioxide from the atmosphere neutralises the alkaline calcareous materials that inhibit corrosion of the steel reinforcement. This process is called carbonation. The rate of carbonation depends on the quality and type of concrete, the level of humidity in the concrete cover, the content of CO_2 in the ambient air, etc. [64]. When the carbonation has reached the depth of the embedded steel, moisture can cause corrosion with severe expansion disruption of the concrete.

The rate of carbonation increases rapidly with increased CO_2 content, as shown in fig D.4:2. The rate is therefore substantially higher in industrial than in rural atmosphere. The average global CO_2 concentration for 1994 was $358 \cdot 10^{-6}$ (0,0358 %), undergoing a yearly average increase in the last decade of $1{,}5 \cdot 10^{-6}$ [65].

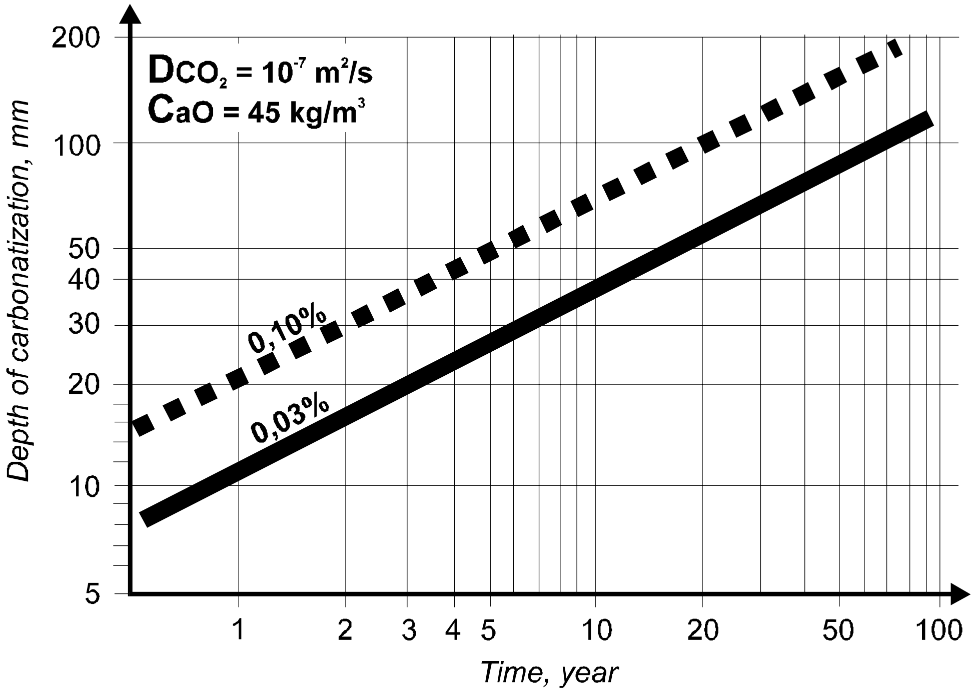


Fig D.4:2 Effect of ambient CO_2 concentration on the rate of carbonation of concrete (normal background CO_2 concentration is 0,03 %) [64]

D.4.2.3 Sulphur dioxide

Sulphur dioxide (SO_2) is, in connection with atmospheric corrosion, the most studied air pollutant. SO_2 is considered a major contributor to acid rain and environmental acidification. Local concentrations of SO_2 show considerable variations. During recent years the SO_2 concentration has declined in most countries, due to the combined effects of growing environmental awareness and improvements of industrial processes and combustion techniques (section E).

SO_2 is easily adsorbed on material surfaces, and the deposition may be wet or dry. The concentration, or deposition rate, of SO_2 is a parameter in the classification of atmospheric corrosivity according to the ISO/DIS standard 9223 (see section F). SO_2 transforms in the atmosphere to SO_3, which may also be both wet and dry deposited. The transformation reactions may take place both in gas phase or in aerosol phase, e.g. in cloud droplets.

SO_2 degrades materials that are susceptible to acid attack. Moisture conditions are of vital importance. As shown in section C, relatively good quantitative dose-response functions exist for quite a few materials.

D.4.2.4 Effects of nitrogen pollutants, SO_2, and ozone on materials

The most important sources of NO_x are vehicular traffic and combustion of petroleum products. Increased traffic intensity has led to high concentrations of many oxidised forms of nitrogen, such as NO, NO_2, HNO_2, and HNO_3. Whereas the concentration of SO_2 generally has decreased during recent years, NO_x pollution has increased.

From a publication of Kucera [66] (Swedish Corrosion Institute - project-leader of the UN ECE ICP on materials 1996), the following to D.4.2.5.6 is quoted.

In contrast to SO_2 the effects of NO_x (nitrogen oxides) and O_3 (ozone) are not well documented. Some reviews in this area have been published, usually focusing on the effects of either NO_2 [67] or O_3 [68,69] on metallic or organic materials and surfaces. The increased interest in the effects of nitrogen pollutants and VOC within the Convention on LRTAP has evoked systematic investigations also in the field of materials. In the 1980´s the effect of NO_x attained increasing interest. In recent years, finally, the effect of O_3 in combination with S- and N-pollutants has been subject to systematic investigations that have revealed synergistic corrosive effects. Consequently the effect of acidifying air pollutants is now considered as a combined effect in the multi-pollutant complex situation [70]. To treat this field exhaustively is beyond the scope of this presentation. In the sequel, a brief description of the mechanisms of main corrosive effects involving NO_x and O_3 will be given, including examples of effects on different inorganic materials. The effect of ozone on organic materials will be treated in the following presentation [71].

D.4.2.4.1 Effect of NO_x

In the combustion of fossil fuels, nitrogen oxides are emitted largely in the form of NO (nitrogen oxide) which is subsequently oxidised to NO_2 (nitrogen dioxide) in the air, the most rapid being the reaction with O_3 and the nitrate radical NO_3:

$$NO + O_3 \rightarrow O_2 + O_2 \qquad (38)$$

$$NO + NO_3 \rightarrow NO_2 \qquad (39)$$

The deposition rate of NO_2 on material surfaces is much lower than that of SO_2 and this is certainly one of the reasons that low contents of nitrates compared to sulphates are usually found in corrosion products also in rain protected areas. It is known that the direct corrosive effect of NO_2 is low except on some organic materials such as like polymers and textiles.

Pollution is a complex process and gaseous HNO_3 can be formed by photolytic oxidation of HNO_2 or by further oxidation of nitrogen oxides in the following sequence of reactions:

$$NO + O_3 \rightarrow NO_3 + O_2 \qquad (40)$$

$$NO_2 + NO_3 + M \rightarrow N_2O_5 + M \qquad (41)$$

$$N_2O_5 + H_2O \rightarrow 2HNO_3 \qquad (42)$$

Since HNO_3 (nitric acid) is a strong acid with a high deposition rate, it is of principal interest for corrosion. The HNO_3 concentration has been supposed to be low in urban areas close to emission sources due to the slow reaction rate. During the last years, daily mean values of HNO_3 between 4 and 28 mg/m^3 have been reported in Rome [72]. The corrosive action of HNO_3 on calcareous stones has been demonstrated to occur under natural conditions in Athens and in the laboratory [73]. HNO_3 exerts its corrosive effect also at low relative humidity. This is important in warm, dry climates like in California or in southern Europe, and can give rise to a summer peak of atmospheric corrosion. It can be noted that in Athens, the highest values of HNO_3 were found in the summer period.

D.4.2.4.2 Synergistic effect of SO_2 and NO_2

In the 1980´s, a strong synergistic effect of NO_2 and SO_2 was found in several laboratory investigations. The mechanism may be either a direct oxidation according to:

$$SO_2 + NO_2 + H_2O \rightarrow SO_4^{2-} + 2H^+ + NO \qquad (43)$$

which has been found on gold and lead, while the oxidation of sulphite to sulphate on, for instance, zinc and calcareous stones, can be accelerated with NO_2 acting as catalyst. It may be emphasised that in both of the synergistic reactions involving NO_2, no solid nitrogen containing corrosion products are created.

D.4.2.4.3 Effects of ozone

The principal mechanism of formation of ozone in the troposphere is by reduction of NO_2 to NO due to photolysis:

$$NO_2 + h\nu \rightarrow NO + O \qquad (44)$$
$$O + O_2 + M \rightarrow O_3 + M \qquad (45)$$

where M denotes a third body molecule.

Ozone levels are controlled by the emissions of NO_x and volatile organic compounds (VOC), which is illustrated in fig D.4:3.

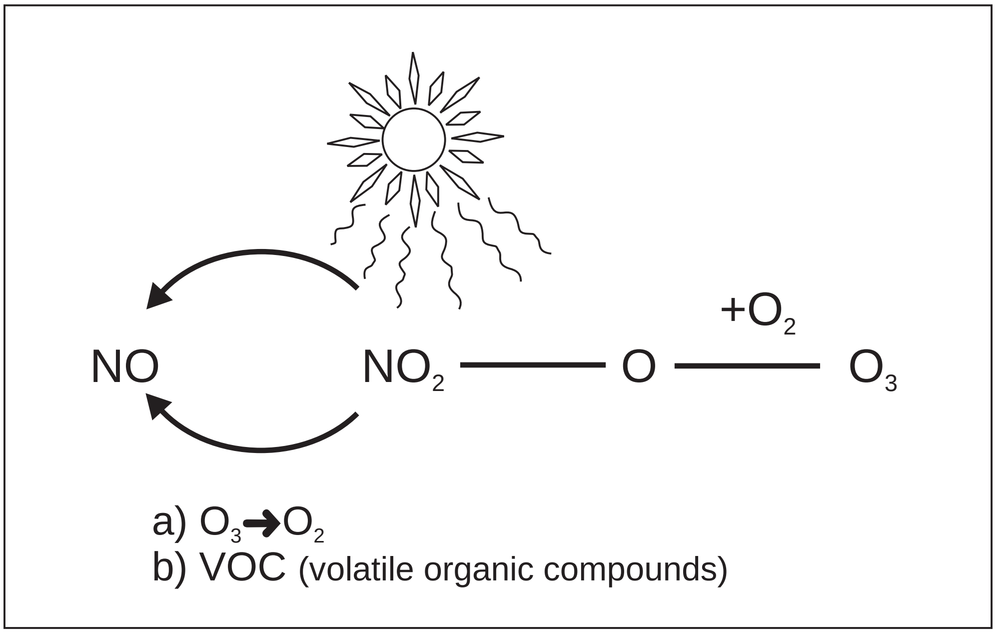


Fig D.4:3 Ozone created from NO_2 under the influence of UV radiation. NO_2 then regenerates from NO by (a) reaction with ozone; (b) by reaction including VOC

The ambient levels of the tropospheric ozone have almost doubled in the last 50 years. The use of catalytic converters on motor vehicles is expected to reduce O_3 concentrations in the long term. However, as NO emitted by motor vehicles is a major sink for O_3, decreased NO concentrations will probably increase O_3 levels in urban areas where the bulk part of the material stock is located [74].

This may be illustrated by the strong negative correlation between NO_2 and O_3 that has been found on the UN ECE ICP Materials test sites, described by the relation [66]:

$$[O_3] = 60{,}5e^{-0{,}014[NO_2]} \qquad (46)$$

For this reason, O_3 levels are usually lower than in areas with heavy traffic as in town centres outside city areas. Ozone in contrast to NO_x can thus create both a local and a more regional problem.

Ozone exerts a direct corrosive effect on natural rubber, on some plastic materials, textiles and on paint and surface coatings [69,71].

D.4.2.4.4 Synergistic effect of SO_2 and O_3

Recently, as mentioned above, a synergistic effect of SO_2 and O_3 has been disclosed in laboratory exposure of zinc, copper, nickel and calcareous stone materials. The following mechanism is proposed for O_3:

$$SO_2 + O_3 + H_2O \rightarrow SO_4^{2-} + O_2 + 2H^+ \qquad (47)$$

When comparison has been possible it has been shown that, the synergistic effect of SO_2 + O_3 is stronger than for SO_2 + NO_2 [75], see fig D.4:4.

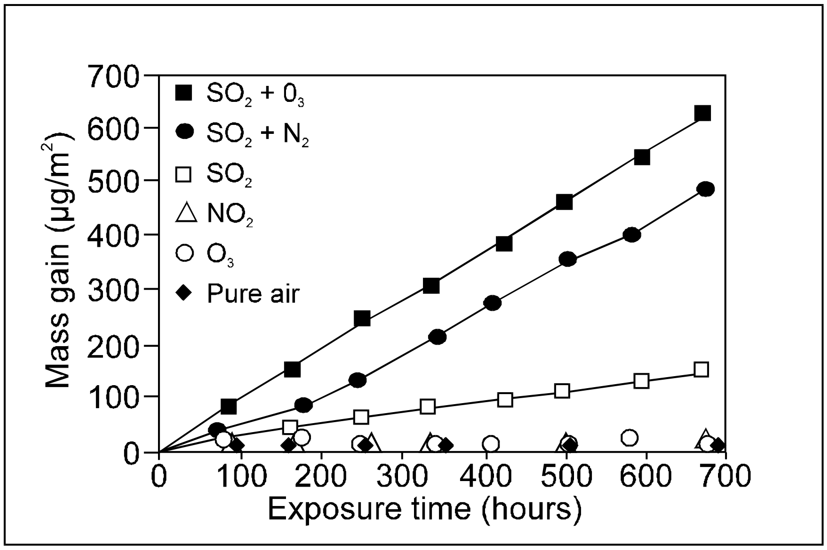


Fig D.4:4 Mass gain of zinc exposed at 95 % RH; influence of SO_2 ($225 \cdot 10^{-9}$), NO_2 ($400 \cdot 10^{-9}$), and O_3 ($400 \cdot 10^{-9}$) [75]

D.4.2.4.5 Effects on materials

In the following sections, individual materials are treated separately:

D.4.2.4.5.1 Steel

Only a few field exposures have been performed and the results are inconclusive. A Japanese investigation has shown a positive but not significant correlation between corrosion rate and NO_2 concentration, while another study has shown that SO_2 increased, NO_2 decreased and O_3 had no effect on the corrosion rate. In the UN ECE exposure programme, a dose-response function was presented for weathering steel after four years exposure involving both SO_2 and O_3 as explanatory variables, see Table C.4:1. This is the first time a synergistic effect of O_3

and SO_2 has been indicated in a field exposure. The role of NO_2 has not been clarified in the UN ECE exposure. In laboratory studies both inhibitive effects of NO_2 and a synergistic corrosive effect in combination with SO_2 have been reported [75].

D.4.2.4.5.2 Zinc

Field studies in the United States have indicated that dry deposition of HNO_3 can be an important parameter. In a statistical analysis of the degradation rate measured as Zn^{2+} run-off, NO_3^- significantly improved the fit compared to an equation with only SO_4^{2-}. Also in a California study, the corrosion rate for galvanised steel was higher in the summer than in the winter, which is the opposite of the seasonal dependence observed at most other locations. This could be due to HNO_3 produced by photochemical oxidation in the summer.

In the UN ECE exposure programme a dose-response relation for zinc, involving both SO_2 and O_3 as explanatory variables, has been presented, see Table C.4:1.

D.4.2.4.5.3 Copper materials

In the UN ECE exposure programme copper and bronze are exposed as a construction material and copper also more sheltered as an electrical contact material. The statistical evaluation of structural metals after four years exposure resulted in dose-response functions for copper and bronze, involving both SO_2 and O_3 as explanatory variables, see Table C.4:1. In the evaluation of electrical contact materials it was concluded that both O_3 and NO_2 accelerate the deposition of SO_2 on copper. Strong weight increase was observed both at sites with high SO_2 concentrations and at sites with low SO_2 concentrations where O_3 reached high levels.

In laboratory exposure the combination of NO_2 and SO_2 exerts a synergistic corrosive effect but only at high relative humidity levels. The SO_2 deposition rate on copper was seven times higher when O_3 instead of NO_2 was added to SO_2 at 90 % relative humidity.

D.4.2.4.5.4 Nickel

In field exposures conducted at indoor sites, nitrates were not found in the corrosion products. In the UN ECE exposure programme it is the nickel corrosion that correlates most strongly with the SO_2 concentration. No effect of the NO_2 or O_3 concentrations could be proved by statistical methods, although nitrate as well as chloride was detected on the surface together with sulphate, which was the main constituent.

D.4.2.4.5.5 Calcareous building materials

Attack on either $Ca(OH)_2$ or $CaCO_3$ by sulphuric acid generally leads to the formation of gypsum, $CaSO_4 \cdot 2H_2O$, while nitric acid gives calcium nitrate, $Ca(NO_3)_2 \cdot 4H_2O$. The nitrate is more soluble than the sulphate.

In a UK study of limestone decay, the nitrate level was often lower than the detection limit and none of the weathering indices correlated with NO_x [76,77]. Also in other field exposures it was not possible to correlate high NO_x concentration to the low nitrate content of the stone samples. It should, however, be pointed out that calcium nitrate is deliquescent and readily absorbs water from air at relative humidity higher than 50 %. One can then find nitrates on a greater depth in stones than sulphates, which accumulates as $CaSO_4$ in the surface layer.

In the UN ECE exposure of limestone and dolomitic sandstone as well as in the UK national exposure programme, the dose-response relations obtained contain only an SO_2 term characterising dry deposition, and another term describing the effect of wet deposition. The statistical analysis has not shown any effect of O_3 as for the metals.

Limestone, marble and travertine at laboratory exposure to humid air, NO_2 and SO_2 in combination are drastically affected due to synergism. The SO_2 deposition on marble is also strongly enhanced by the presence of O_3. Recent laboratory exposure confirmed that similar to the case of zinc, NO_2 catalyses sulphate formation, while O_3 acts by direct oxidation. The catalytic effect of NO_2 is only efficient at high humidity, whereas the direct O_3 effect is efficient at both dry and humid conditions [78].

D.4.2.4.6 Conclusions

- There are considerable gaps of knowledge concerning atmospheric corrosion effects of NO_x and O_3. Traditionally, scientists working with inorganic materials have focused on SO_2 as the most significant pollutant, while those working with organic materials primarily have dealt with O_3. In this way the synergistic effects of SO_2 with O_3 and NO_2 have not been subject of any systematic investigations until recently.
- Laboratory and field investigations performed in the last decade have contributed to the understanding of the interrelated role of SO_2, NO_x and O_3 and underlined the necessity to treat the deterioration of materials as a multi-pollutant situation, which may increase the significance of degradation from pollutants in less polluted areas and in indoor locations.
- A warning should be issued for the corrosive action of HNO_3. Its importance for atmospheric corrosion, especially in warm regions, should be clarified.
- Reliable dose-response functions are needed in order to be able to calculate acceptable levels of pollutants and for calculations of economic impact.

D.4.3 Acids

The acids most important to the deterioration of materials through atmospheric corrosion are sulphuric acid (H_2SO_4), nitric acid (HNO_3), and hydrochloric acid (HCl) [21].

Sulphuric and sulphurous acids are used in many industrial processes and local pollution may occur around certain industrial facilities. The acids are easily deposited on exterior materials. The pH-values on surfaces close to major pollution sources may be very low, promoting corrosion attack on the materials. The transformation of SO_2 to sulphuric acid in the atmosphere has been mentioned earlier. Sulphuric acid may be transported over long distances and be a major cause of acid precipitation (see section E.3).

Nitric acid is used in many industrial processes. As HNO_3 is very volatile, local pollution may easily occur around industrial facilities. Quantitative data on nitric acid concentration in the atmosphere are scarce. The acid is easily deposited, both wet and dry, and if deposited on material surfaces it is very aggressive and attacks both native and synthetic polymers (see D.4.2.5).

Hydrochloric acid too is also used in industrial processes, and is often detected around many industrial facilities. Combustion of coal and wood is a known source of HCL formation, and a very important and much discussed source is the combustion of waste materials [63]. Hydrochloric acid is especially aggressive to metals, concrete and rendering materials [21].

D.4.4 Salts/aerosols

An important effect of salts is that they, when deposited on material surfaces and due to their hygroscopicity, may increase the moisture content and prolong the wetness periods of the materials. Besides that, salts are more or less aggressive. Recrystallisation of salts in micropores of porous materials such as rendering and many stone materials may cause enough tension for the material to decompose.

The chloride concentration in the atmosphere of is of great importance for metal corrosion, which of course also affects building materials such as coil-coated sheet metal, especially at cut edges, holes in the sheet, fasteners, pores and cracks in the coating etc.

Current levels for wet deposition of chlorides, sulphates and nitrates can be found (see section E.5).

D.4.4.1 Sea spray

The sea is a major source of salts. The effect of sea-salt on the durability of construction materials is, together with that of SO_2, the most studied environmental variable [79,80]. Sea spray may be regarded as a form of particulate pollution that can travel under the influence of wind for several miles inland from coastal areas. The salt concentration in the atmosphere decreases rapidly with distance from the coast [79]. Salt spray from the environment can adversely affect the durability of stone as well as accelerating the corrosion of metals, particularly ferrous metals and aluminium.

In Australia, scientists at CSIRO have developed a hyperbolic function of the corrosivity as function of distance from sea, using standard corrosivity specimens of mild steel and copper steel being exposed at numerous sites at various distances from the sea. The hyperbolic function has the following form [80]:

$$\log_e(corrosivity) = A + \frac{B}{1 + C \cdot (distance\ from\ ocean)} \tag{48}$$

The model describing the variation in one-year corrosivity of mild steel with distance from the ocean are shown in fig D.4:5 for three regions: Newcastle; southeast Melbourne; and Victor Harbour.

The very sharp fall-off in corrosivity with distance inland is evident in all three cases, as are the increasing levels of corrosivity in the order Melbourne, Victor Harbour, Newcastle [80]. It should, however, be remembered that the primary variable is the salt and not the distance [79]. For Newcastle, corrosivity was found to vary by more than an order of magnitude across the area surveyed. The rates at the beachfront were the highest ever measured in Australia, and the marine environment represents a major corrosion hazard.

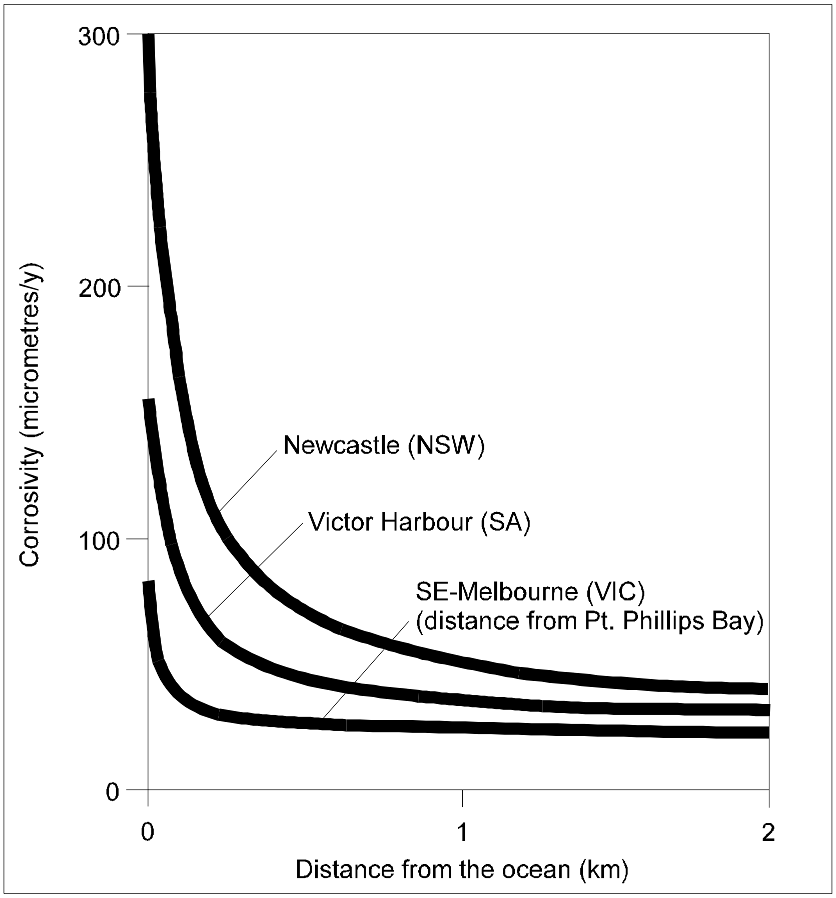


Fig D.4:5 Models showing the variation in the corrosivity of mild steel exposed for one year as a function of distance from the ocean, for three regions in Australia

Quantitative dose-response functions and classification systems exist (see section F). As for dose-response functions a matrix of dose tests have partially been completed, in which a range of materials is subjected to well defined (constant) doses of humidity and airborne salinity during short to medium term exposures [81]. In these exposures, RH was held constant at values from 60 to 100 %, while salt doses equivalent to 0 to 120 mg/m^2d were applied once a week. Exposures without salt went on from 6 to 38 weeks, while those with salt from 6 to 12 weeks. The materials tested included Cu-steel, galvanised steel, solid zinc and aluminium-zinc coated steel.

The data reveal a number of different patterns in terms of corrosion rate dependent on RH and airborne salinity. Galvanised and solid zinc show a strong sensitivity on salinity levels but a weak dependence on RH, while Al-Zn shows a marked sensitivity on RH and a weaker sensitivity on salinity. It is notable that the corrosion rate for the galvanised and the solid zinc plates subjected to a high natural sea salt dosage (120 mg/m^2d) did not decrease when RH was decreased from 90 to 60 % RH. The implication from this result is that the use of the approximation of TOW as the time when RH>80 % is not appropriate when there is significant natural sea salt deposition.

Although the data were not primarily structured for the development of dose-response function type expressions, such expressions can be derived for the data for 90 % RH. As sea salt and NaCl will wet at 90 %, TOW can be regarded as 100 % as long as salt is present. Unit of airborne salinity is mg/m^{2}d. All data were derived at a constant temperature of 30°C. The most appropriate formulation is then:

$$C = a+b\cdot(salinity)$$

For galvanised steel the data analysis, when the TOW factor is explicitly included (adding in data at 100 % RH and thus 100 % TOW but zero salinity), proposes the following form:

$$C_{\mathrm{gal}} = a+TOW\cdot\{b+c\cdot(salinity)\}$$

Cole [81] studied the airborne salinity on the facade and within building envelopes. This study indicated that there was a significant reduction of the salinity level on the facades and a further reduction within the building envelope. However, while the salinity level within the building envelope was considerably reduced, it was not negligible and thus in combination with the relatively high TOW values indicated that a slow but not insignificant corrosion could occur in wall cavities.

D.4.4.2 Soot and particles

During recent years the content of particles in the atmosphere has increased by approximately 4 % per year. Traffic is a main contributor in urban areas. A major effect of soot and particles is that they, if deposited on surfaces, may prolong wetness periods.

Furthermore, soot and particles may contain corrosive compounds, affect surface properties such as light reflection and indirectly cause mechanical damage to surfaces due to the need of cleaning.

More detailed descriptions and classifications of soot and particles and their occurrence in different atmospheres are available [63,82,83].

D.4.5 Chemical incompatibility factors

When different materials are used in conjunction, due consideration should be given to the possible interactions which may occur between them and which may lead to deterioration.

Specific chemical attack from one material on another can occur for many material combinations. Not all such interactions are documented. Often the effect will be dependent on the environmental circumstances and, in particular, the presence or absence of moisture.

For materials in contact or adjacent to each other, the action may be direct or indirect since one material may influence the response of another to the environment. It may also occur that a material degrades and affects neighbouring materials.

D.4.5.1 Leaching/leachates

Certain materials react slowly with water or contain substances soluble in water or are extractable in water. Other materials, whilst not themselves being a source of harmful leachates, may be capable of transporting leachates over quite long distances. The solving action of water for these constituents may cause changes in the properties of the material itself. Alternatively or additionally, the leachate solutions formed can constitute reactive agencies towards adjacent materials that would not be attacked by water alone.

D.4.5.2 Solvents

Other interactions can occur when migrating solvents and solvent vapours cause the plasticisers in certain thermoplastics to migrate and bituminous compounds to soften.

D.4.5.3 Contaminated land

Any building material placed in contaminated soil and groundwater needs special attention as the performance of the materials may be seriously affected by the contaminants. The effect of soil contaminants depends on their combination, on the concentration and on the soil type etc. Examples of causes and effects are:

1. Soil containing solvents which can attack plastic pipes, causing swelling or salvation leading to loss of mechanical properties;
2. Surface active substances, e.g. alcohol, soap and other wetting agents, in combination with stress may cause environmental stress cracking of rigid building products;
3. Many chemicals, typically strong oxidising agents, can chemically attack and degrade plastics and rubber;
4. Land containing explosive gas (e.g. methane) can not only destroy buildings if the gas becomes concentrated in a confined space and is ignited, but the gas can also create voids below ground that lead to gradual subsidence;
5. Bacterial activity, particularly sulphate reducing bacteria affecting corrosion of metals; and
6. Sulphate and acid attack on concrete.

D.5 BIOLOGICAL AGENTS

D.5.1 Fungi and bacteria

Living organisms such as fungi and bacteria are significant environmental degradation agents of organic building materials, but they may also affect inorganic materials such as calcareous stones, sandstone and even metals [63]. While biological factors are not weathering factors, biological attack of exterior building materials depends highly on weather conditions.

Fungi attack on facade materials often appears as mould growth on organic surface treatments or as wood rot. Mould growth on a coating normally causes a darkening of the colour. Linseed oil coatings are regarded to be more affected by mildew attack than coatings based on synthetic resins. However, extenders, stabilisers and plasticisers in synthetic coatings may provide the necessary conditions for biological attack. Modern coatings are normally protected by specific additives, like fungicides and insecticides

Both aerobic and anaerobic bacteria may participate in the deterioration of exterior building materials. Products emanating from the metabolism of the bacteria often cause the attack on materials, e.g. bacteria that feed on sulphides may produce sulphuric acid.

Bacterial activity can induce corrosion in metals when suitable nutrients are available. Concrete structures can be attacked by sulphuric acid produced by bacteria that use hydrogen sulphide derived from the breakdown of putrescible materials as an energy source.

D.5.2 Fungal and insect attack in timber

Fungi causing wood decay and wood boring insects are found more or less everywhere, and they can cause damage in most parts of a building.

A systematic survey of microbiological attack and its causes on timber has been presented [55], see fig D.5:1. *Temperature, moisture content of timber* and *nutrients* are the predominant factors when considering its susceptibility to both fungal and insect attack. The *duration* of their effect is also of great importance.

For fungi to grow on wood a moisture content of 28 % or more is necessary. If growth has started, it can continue at lower moisture levels. A typical activity curve for fungi as a function of temperature is also shown in fig D.5:1, while Table D.5:1 shows moisture-temperature requirements for some typical damaging fungi in Norway [55].

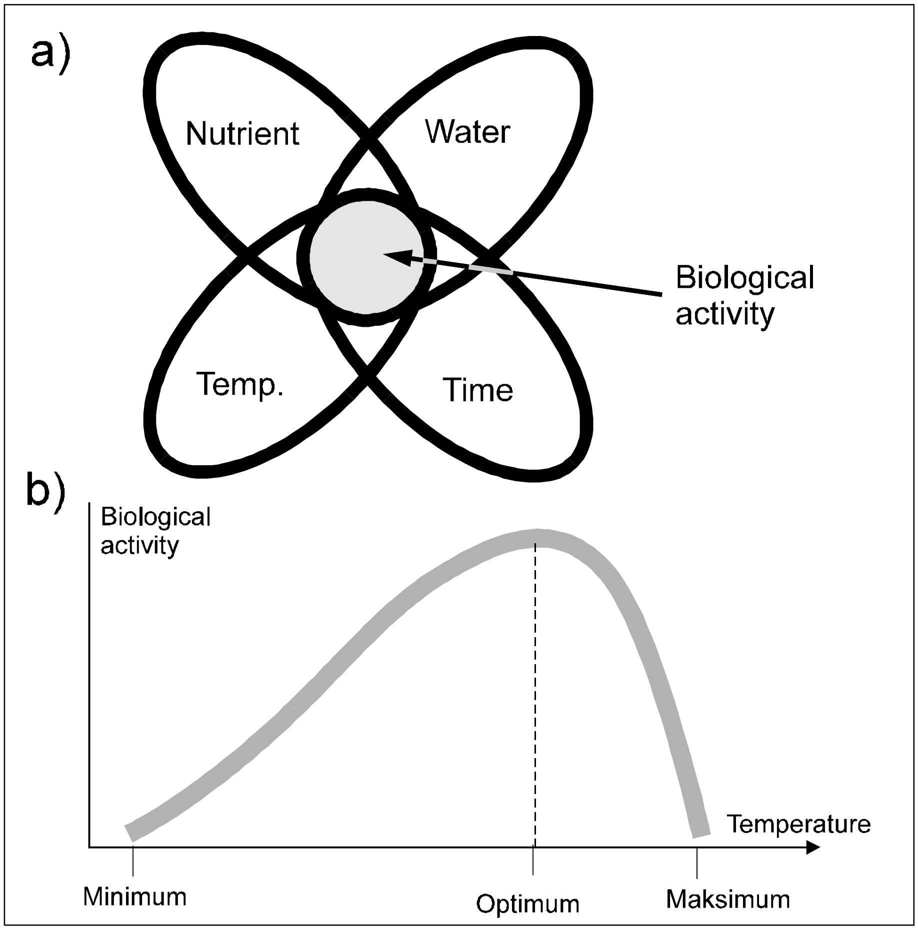


Fig D.5:1 a) Determining environmental degradation factors for micro-biological attack on wood, and b) Typical temperature dependent activity curve for fungi on wood [55]

Table D.5:1 Moisture temperature regimes for typical damaging fungi in Norway [55]

Type of fungi	Moisture in wood	Optimal temperature	Lethal
Corticiacea spp.	50-70 %	~ 30°C	~ 50°C
Blue stain fungi	> 30 %	~ 25°C	~ 45°C
Dry rot fungus(Serpula lacrymans		~ 23°C	~ 35°C
“Poria” fungi, white pore fungus (Antrodia serialis, A, sinuosa, A, xantha, Fibroporia waillantii)		~ 28°C	~ 45°C
Cellar fungi		~ 23°C	~ 40°C
Mould fungi	20-150 %	20-45°C	~ 55°C
Dacrymytces stillatus	20-150 %	~ 23°C	~ 35°C
Gloeophyllum sepiarium	30-50 %	~ 35°C	~ 75°C

The location of the most frequent rot spots on a timber building is likely to depend on moisture content [55].

The information above on fungus activity fits very well with the climatic index developed by Scheffer for the US, as follows [84]:

$$\textit{Climate index} = \frac{\sum_{\text{Jan}}^{\text{Dec}} \left[(T-2)\ \ (D-3) \right]}{17} \qquad (49)$$

where T is the monthly mean temperature in °C and D is the mean number of days in the month with 0,25 mm or more precipitation.

This climatic index, which also is a damage function, for the decay of wood is based on US public bodies' requirements for the estimation of needs for wood protective measures**.** The index was developed from ten years exposure of samples of various types of wood and correlates their decay rate with existing meteorological monthly data for temperature and duration of precipitation. The derivation of this index is thus based on the *assumption, which was shown to be highly valid,* that *precipitation and temperature* by far overshadow other elements of climate that might influence attack by wood destroying fungi.

Scheffer also stated that analysis of typical growth - temperature curves for a few commonly occurring decay fungi indicated that the suitable temperature factor for the climate index would be the mean monthly temperature minus 2°C, taking into account that most of the fungi become active a couple of degrees above 0°C. It is also assumed that for practical requirements the dependence of fungi growth rate, and presumable also of the decay rate, on the temperature can be considered as linear from 0°C to the actual active temperature range. This is not strictly true for all fungi, but fits very well with fig D.5:1.

The *duration of rain* was considered more significant for the total wetting of wood, and gave better correlation, than the precipitation amount. The term "-3" in the formula is added just to keep the index in the range of 0-100 for most parts of the US.

Insect attack is possible at a mass fraction of 12 % of moisture and above, which often applies to roof and ground floor timbers, and sometimes to mid-floor timbers.

D.5.3 Rodents and birds

The pests most likely to cause damage or deterioration to building materials are mice and rats. They can cause considerable damage by gnawing timber, other organic substances and electric cables casings of PVC.

Birds can brake fragile roofs by pecking and sometimes cause damage by the debris that they collect or drop. Woodpeckers sometimes attack external timber claddings.

Animal waste matter is a source of nutrient for plants, insects and bacteria that may cause damage.

D.5.4 Plants and trees

A major cause of damage to buildings from plants and trees is due to roots disrupting foundations or penetrating underground drains and to new tree planting in clay subsoils.

Serious damage has occurred where buildings with shallow foundations or with faulty infilling of the site have been built on sites of shrinkable clay, especially if there are trees or large shrubs nearby. If the trees or other large plants die or are cut down, water will no longer be extracted from the surrounding soil and the clay will swell. Damage to adjacent buildings will occur unless the foundations have been designed to withstand swelling [85].

Plants can also cause damage by growing in gutters and blocking them, and by developing roots in the joints and cracks of masonry walls or in drains.

E Measurements, modelling and mapping of air quality

E.1 INTRODUCTION

The measuring, testing and evaluation of air quality are assuming growing importance in developed countries as elements of a comprehensive clean air policy geared to the objective of sustainable development. Accordingly, a significant amount of data is generated on the various geographical levels. This concerns point measurements of both emissions and ambient air concentration, as well as emission surveys of almost every type of source.

Point measurements are very expensive, and for a broader assessment of air quality, needed for policy development and assessment, public information etc., measured data need to be combined with modelling based on emissions inventories, in order to properly assess the exposure to, and thus the effects of the pollution on public health or buildings. Such air dispersion models exist, and the results can be mapped and exhibited by modern information technology (GIS).

During the last decades it has become evident that air pollution is not only a problem of densely populated and industrialised areas, but pollutants may be transported over long distances and affect the environment far beyond the source regions. This implies that monitoring and control of air pollution on a local or national scale is not sufficient to solve environmental problems, and that international co-operation is a necessity.

In the context of assessing building performance, the bulk of data on global, continental (Macro) and national (meso) levels is therefore available for exploitation. To some extent data are also available for the local scale, while most is lacking for the microenvironment on buildings. This should then be a subject for concentrated R&D effort. Some methods are available from the environmental area for such an effort. This will be dealt with in section G.

The following is a state of the art report on the availability of data and methods from the atmospheric environmental area on the *global*, and then the *macro* and *meso* scale in Europe. In turn, the UN Global Environment Monitoring System (GEMS/AIR), the trans-boundary UN ECE EMEP-programme and the tasks organised under the European Environment Agency in Copenhagen are described, and some data are given.

E.2 GLOBAL ENVIRONMENTAL MONITORING

E.2.1 UN Global Environment Monitoring System (GEMS/AIR) – 1973-1992

GEMS/AIR is an urban air pollution monitoring and assessment programme that evolved from a World Health Organisation (WHO) urban air quality monitoring pilot project started in 1973. Since 1975, WHO and the United Nations Environment Programme (UNEP) have jointly operated the programme as a component of the United Nations worldwide Global Environment Monitoring System (GEMS). GEMS is a component of the UN Earth watch system.

The original objectives of GEMS/AIR were to:

- Strengthen urban air pollution monitoring and assessment capabilities in participating countries;
- Improve the validity and comparability of data amongst cities; and
- Provide global assessments on levels and trends of urban air pollutants, and their effect on human and ecosystem health.

E.2.2 The GEMS/AIR Achievements and Network

Since its inception in 1973, the GEMS/AIR network has included some 270 monitoring sites in 86 cities in 45 countries. Monitoring stations are run by the national or city authorities that voluntarily contribute their data to the GEMS/AIR global database. The cities represent a wide range of climatic and socio-economic conditions as well as different levels of development and air pollution control capabilities. GEMS/AIR is the only global programme that provides long-term air pollution monitoring data for cities in industrialised as well as in developing countries. Thus the programme enables the production of assessment of the levels and trends of urban air pollution worldwide.

E.2.3 GEMS/AIR Phase 2 (1993–2000) – Perspective and Programme Activities

In recognition of the increasing challenges faced by the escalating urbanisation all over the world, an outline for an expanded GEMS/AIR programme (GEMS/AIR Phase 2) was prepared for the period 1993-2000. The overall *objective* of the programme is to provide the comprehensive information needed for rational air quality management.

Within this objective, the following sub-objectives were defined:

- Provide an international framework for co-ordinated and valid monitoring of urban air quality, effective data management, and reliable information dissemination;
- Develop methodologies, adapted to the specific needs of the participating countries, required for the comprehensive monitoring and assessment of urban air quality;
- Produce comprehensive assessments which include levels and trends of urban air quality, pollution sources, and options for abatement, as well as of potential health and environmental effects; and
- Strengthen urban air quality monitoring networks and assessment capabilities in developing countries.

E.2.4 The GRID – Arendal Solar UV Radiation Information Service

Ozone depletion leads to an increased intensity of ultraviolet radiation reaching the earth surface. Excessive exposure to UV rays from the sun leads to sunburn, and, in some cases, also to skin cancer. If people continue their sunbathing habits the health implications could be serious.

UV-intensity maps are being produced by GRID-Arendal in co-operation with NILU. Data from sun-angle and satellite measurements of stratospheric ozone are entered into a model to calculate UV-intensity at noon for the whole earth. Maps are then produced indicating this intensity for selected areas.

Institutions, media and private persons can now order a UV-intensity information package from GRID-Arendal, with colour maps and graphics showing the UV and ozone situation over a selected period of time.

At an additional cost – based on production time – custom made UV-intensity maps for selected areas can also be ordered.

The unit used on the UV-intensity maps produced is the UV-index developed by Environment Canada. The UV-index runs on a scale from zero to ten, with ten being a typical mid-summer day in the tropics. A relative scale from low to extreme is also applied. In extreme conditions, with a UV-index higher than nine, light and untanned skin will burn in less than 15 minutes.

Three main trends in UV-intensity level disparity are easily seen on the UV-intensity maps from GRID-Arendal. These are the variation between north and south, the influence of stratospheric ozone, and the annual fluctuations.

There is now also information on UV monitoring around the world on home pages on the World Wide Web. The address is:

`http://www.srrb.noaa.gov/UV`

This information shows that quite a few countries now perform spectral measurements and this is even more interesting from point of view of materials durability (see section D.3). A newly developed cheap instrument that can be used for measurements on buildings is described in section G.2.

E.3 UN ECE EUROPEAN MONITORING AND EVALUATION PROGRAMME (EMEP)

E.3.1 Background

In Europe, most of the research and monitoring activities related to long-range transport of air pollutants have been connected to the "acid-rain" issue. Towards the end of the 1960s, Nordic scientists presented data that strongly indicated that acidification of rivers and lakes in southern Scandinavia was having a severe impact on aquatic life. These data led to a discussion on whether air pollutants emitted in industrialised areas of central Europe and United Kingdom could cause acidification of precipitation in Scandinavia. To study this, the Organisation for Economic Co-operation and Development (OECD) in 1972 launched a co-operative programme with the objective "to determine the relative importance of local and distant sources of sulphur compounds in terms of their contribution to the air pollution over a region, special attention being paid to the question of acidity in atmospheric precipitation." The programme, which ended in 1977, provided the first comprehensive insight into the transport of air pollutants on a continental scale. It showed that "air quality in any one European country is measurably affected by emissions from other European countries" [86]. Mathematical models developed within the programme made it possible for the first time to quantify the depositions within one country due to emissions in any other country.

During the OECD programme, which had included 11 Western European countries, it became evident that future studies ought to include all European countries. After a period of discussions at a political level, such an all-European programme was launched in late 1977: the Co-operative programme for the monitoring and evaluation of long-range transmission of air pollutants in Europe (EMEP). It was organised under the auspices of the United Nations Economic Commission for Europe (ECE), in co-operation with the United Nations Environment Programme (UNEP) and the World Meteorological Organisation (WMO). Today EMEP is an integral part of the co-operation under the 1979 Geneva Convention on Long-range Trans-boundary Air Pollution [87].

E.3.2 Objective and organisation

The main objective of EMEP is to provide governments with information on deposition and concentration of air pollutants, as well as on the quantity and significance of long-range transmission of air pollutants and trans-boundary fluxes. To achieve its objective, EMEP is based on three main elements:

- Emission data;
- Measurements of air and precipitation quality; and
- Atmospheric dispersion models that, by using emission data, meteorological data and parameters describing transformation and removal processes, provide concentration and deposition fields for relevant pollutants.

A Steering Body for EMEP, where all participating countries are represented, has been established to supervise its work. Canada and United States are also participating in the Steering Body, inter alia to ensure proper liaison with related activities in North America. The Executive Body takes final decisions on work programme and budgetary matters for the Convention on Long-range Trans-boundary Air Pollution.

The EMEP-activities are divided into two main parts, a chemical and a meteorological. A Chemical Co-ordinating Centre (CCC), located at the Norwegian Institute for Air Research (NILU), is responsible for the chemical part of the programme. The main tasks are to collect, check and store data from national measurement programmes in the participating countries and to organise and co-ordinate quality assurance procedures. The meteorological part of EMEP is being undertaken at two Meteorological Synthesizing Centres, an eastern centre (MSC-E) in Moscow, and a western centre (MSC-W) at the Norwegian Meteorological Institute in Oslo. Their main tasks are to design, operate and verify atmospheric dispersion models. The work at the EMEP centres is funded by the participating countries according to a protocol to the Convention, while measurement activities are funded through national monitoring programmes in the participating countries. The participating countries also contribute by organising workshops and expert meetings on selected topics.

Working Groups on effects are also established within the context of EMEP, under which the WG on materials exist, and which has established the UN ECE ICP on materials exposure programme (see section C.4).

E.3.3 EMEP's work programme

During its first years of operation, only sulphur compounds were included in EMEP. The work programme has been gradually expanded to include also nitrogen compounds, volatile organic compounds and ozone.

E.3.3.1 Emission data

Emission data are essential as input for model calculations, and is also needed for other purposes in the co-operation under the Convention. The EMEP models have so far been using a grid size of 150 km × 150 km, but new models under development will be based on a 50 km × 50 km grid. Emission data are to be provided by the participating countries and they are at present working on annual emission data for 1990 in the 50 km × 50 km grid. This is closely linked to the EU project CORINAIR ‘90 which now forms part of the working programme of the Copenhagen based European Environment Agency (EEA) (see section E.4).

For the model calculations, a best as possible time resolution of the emission data are needed. This has so far been solved by specifying seasonal variations based on information on heating demand, etc.

The collection of reliable emission data with the required spatial resolution has been a difficult task within EMEP. Workshops and expert meetings have been used to elaborate guidelines to harmonise the methods used for emission estimates. The present guidelines include SO_2, NO_x, NH_3, CO and VOC. A Task Force was established to strengthen the co-operation between the large number of countries, and to further develop the common methodologies and a guidebook with detailed descriptions of emission inventories, including recommended default emission factors, was issued in 1995. The efforts on improving the quality of European emission data are of vital importance, not only for EMEP but also for efforts related to monitoring of compliance with the various commitments European countries have made by signing agreements on emission reductions.

E.3.3.2 Measurements of air and precipitation quality

EMEP monitoring sites should be located in rural areas in order to be representative for large areas and should not be unduly influenced by local sources. However, emission densities vary much over Europe and it has therefore not been possible to define strict location criteria. The present station network includes about 100 sites in about 30 countries [88], see fig E.3:1.

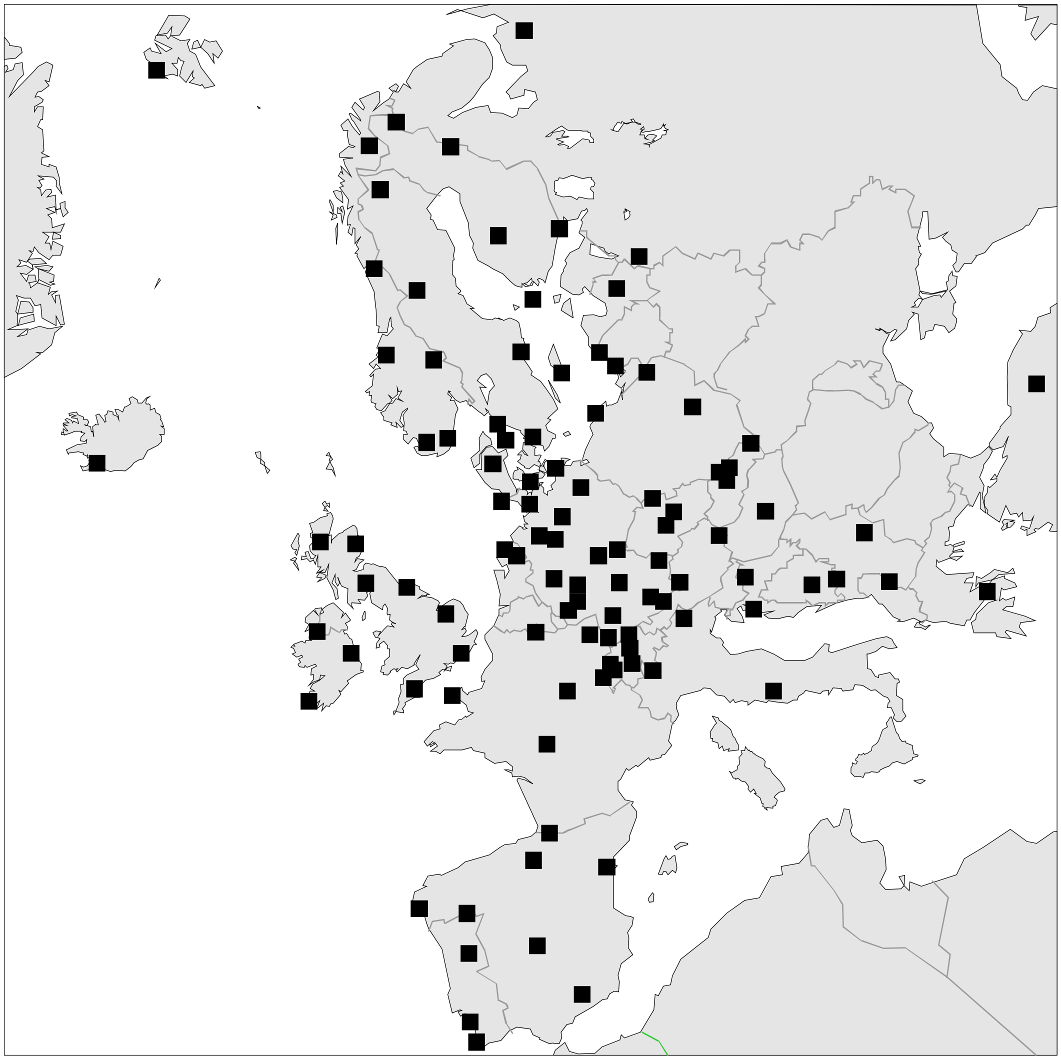


Fig E.3:1 EMEP sampling network in 1993

The present measurement programme includes the components shown in Table E.3:1. For nitrate and ammonium, impregnated filters have been used to measure the sum of these compounds in gaseous and particulate form. This is not a satisfactory solution and is changed so that gaseous and particulate species are determined separately by using denuder techniques.

Measurements of light hydrocarbons were initiated in late summer 1992 at five EMEP sites. Samples are collected in electropolished stainless steel canisters at noon twice a week, and are sent to the CCC for chemical analysis. In addition, aldehydes and ketones (eight-hours samples) were included from spring 1993. One of the objectives is to establish a database for VOC concentrations to be used for comparison with results from photochemical model calculations.

It has not been possible to standardise methods for sampling and chemical analysis within EMEP, but a manual with recommended methods has been issued. Most EMEP stations use either a low volume sampler where an air sample is drawn through a filter for collection of particles and then through a solution for absorption of the relevant gases, or a medium volume sampler with impregnated filters instead of a bubbler. The samples are analysed at a national laboratory and data reported to the CCC. For precipitation, both wet-only and bulk sampling are accepted. Several chemical methods are being used, but ion chromatography is being used by an increasing number of laboratories.

The large number of laboratories participating in EMEP and the lack of standardisation of methods imply that various inter-comparisons play an important role in the quality assurance activities:

- Inter-laboratory tests of chemical analyses; and
- Comparison of the performance of different air samplers – field inter-comparisons have been undertaken.

Table E.3:1 EMEP's measurement programme

	Components	**Measurement period**	**Measurement frequency**
Gases	SO_2, NO_2	24 hours	Daily
	O_3	Hourly means stored	Continuously
	Light hydrocarbons C_2-C_7* ketones and aldehydes (VOC)	10-15 minutes 8 hours	Twice weekly Twice weekly
Particles	SO_4^{2-}	24 hours	Daily
Gas + particles	HNO_3 (g) + NO_3^- (p) NH_3 (g) + NH_4^+ (p)	24 hours	Daily
Precipitation	Amount of precipitation, SO_4^{2-}, NO_3^-, Cl^-, pH/H^+, NH_4^+, Na^+, Mg^{2+}, Ca^{2+}, K^+, conductivity	24 hours	Daily

* Measurements are made at a small number of sites only.

Additional quality assurance elements are:

- Regular questionnaires to collect detailed information on monitoring sites, sampling equipment and methods for chemical analysis as well as on local quality assurance procedures. This information is stored and regularly updated by the CCC;
- Expert consultations, which include visits of a representative from the CCC to participating laboratories, site inspections and visits from the participating countries to the CCC;
- All data received by the CCC are run through a quality control check, e.g. by checking the ionic balance of precipitation analysis; and
- QA elements have been collected in a QA plan.

After completion of quality checks, summaries of the data submitted are presented in annual reports, which normally will be printed one to two years after the end of the measurement period (e.g. the 1993 report was issued in July 1995).

Some results of EMEP measurements of relevance for materials degradation are given in figs E.3:2-3, showing sulphur dioxide and pH in precipitation, respectively, in rural areas in 1993. Further information on EMEP measurements is available [17,88,89].

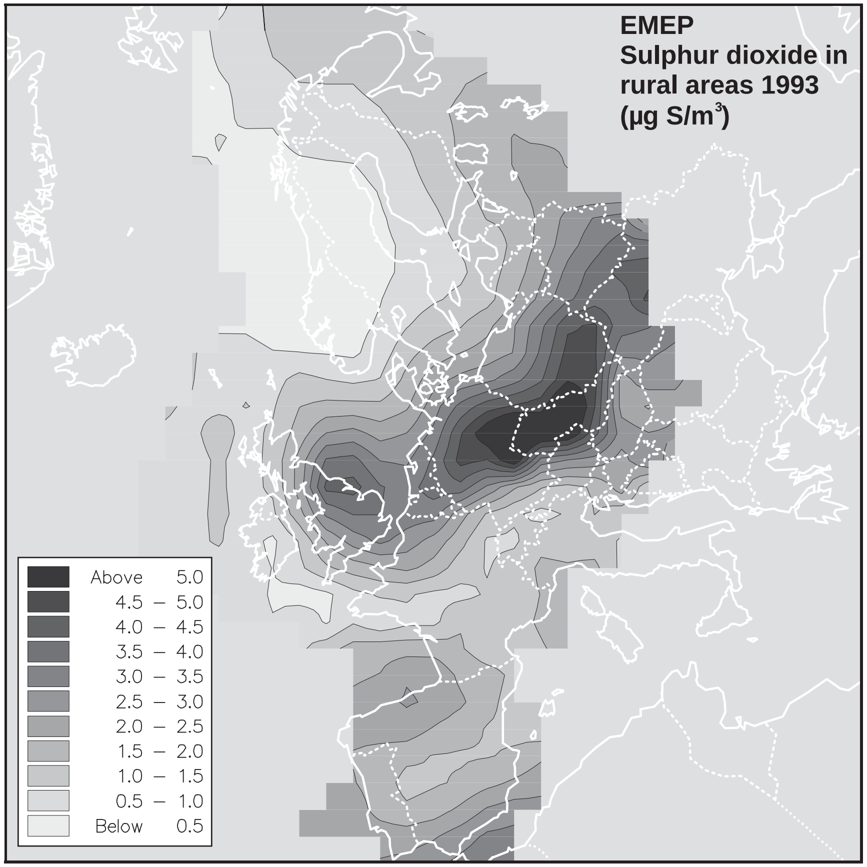


Fig E.3:2 Isoplots of annual average SO_2 concentrations (krieged from measurements in rural areas), for 1993 [90]

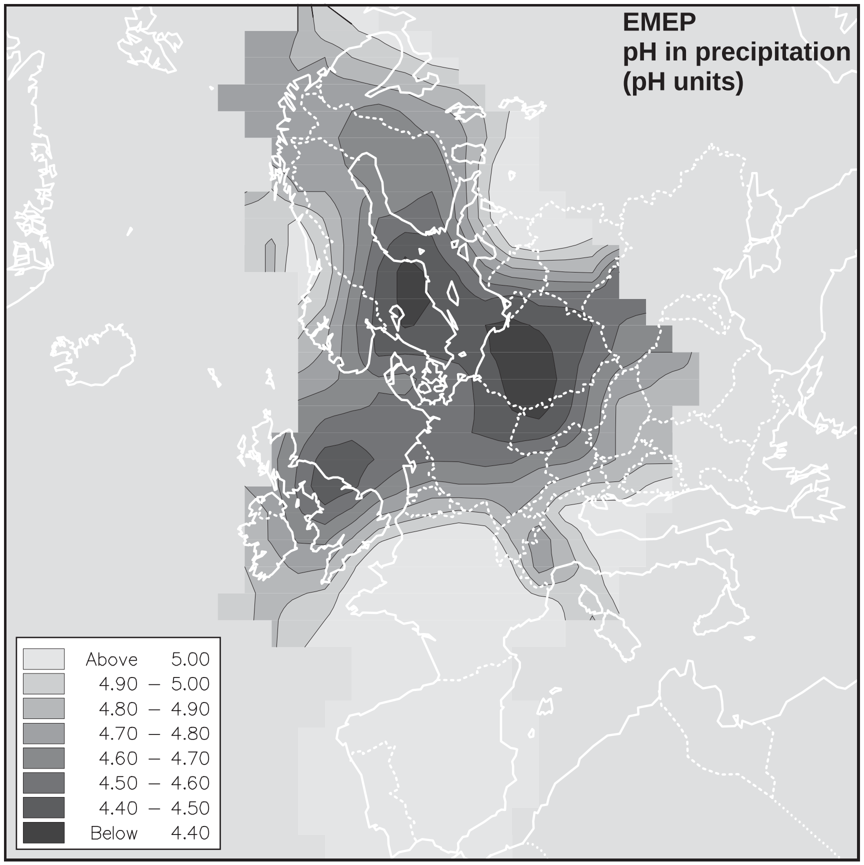


Fig E.3:3 Isoplots of annual average pH (krieged from measurements in rural areas), for 1993 [90]

E.3.3.3 Modelling activities

The meteorological dispersion models employed by EMEP's Meteorological Synthesizing Centres are similar in their basic approach. The one-layer Lagrangian models being used calculate concentrations of pollutants in air parcels following air trajectories from a balance of sources and sinks. For primary pollutants (e.g. SO_2) the emission inventory provides the source term. Chemical transformation may act as a sink (e.g. for SO_2) or a source for secondary pollutants (e.g. sulphate). All pollutants are removed by dry and wet deposition. Model calculations are regularly made for sulphur and nitrogen compounds, and some model estimates for large-scale formation and transport of ozone are also available. The most important basic meteorological data needed are on wind fields (925 mb) and precipitation fields given every six hours. For further information on the models used, including comparisons between model results and measurements, reference is made to reports prepared by the EMEP centres, e.g. [17,91,92,93]. Work has been initiated by MSC-W on multi-layer Eulerian models. However, it is a clear goal for EMEP to keep the models simple enough to allow calculations over long time periods (several years).

The most widely used results from EMEP are the model estimates of sulphur deposition in each European country caused by emissions in other countries. Annual country-to-country emitter-receiver matrices for sulphur and nitrogen compounds are regularly provided by EMEP and form a valuable input to the work under the Convention. The model results have been a prerequisite for the negotiations on emission reductions based on the critical-load approach[2]. Tables E.3:2 and E.3:3 show some results from MSC-W for sulphur.

Table E.3:2 Sulphur deposition tons) in some European countries in 1991 from sulphur emissions in Germany [91]

Country	Deposition
Austria	50
Belgium	10
Belarus	31
Czechoslovakia	123
Denmark	14
Finland	17
France	58
Germany	899
Hungary	21
Italy	33
Netherlands	20
Norway	20
Poland	252
Romania	23
Russia	71
Sweden	29
Switzerland	13
Ukraine	46
United Kingdom	18
(Remaining area inside EMEP-grid)	448

Deposition Unit: 1000 tons;
German SO_2-emissions in 1991: 2870 ktons

Table E.3:3 Sulphur deposition in Germany in 1991 from emissions in various European countries [91]

Country	Deposition
Austria	2
Belgium	27
Czechoslovakia	87
Denmark	5
France	51
Germany	899
Hungary	7
Italy	8
Netherlands	12
Poland	38
United Kingdom	47
(Remaining countries and seas)	(26)
(Indeterminate origin)	(51)

Deposition Unit: 1000 tons
Total deposition: 1261 ktons

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[2] The "critical-load approach" takes the level of deposition or concentration that an ecosystem can tolerate without negative effects as a starting point. The "critical load" may vary geographically.

E.3.4 Further development and concluding remarks

Taking into account the large number of countries participating in EMEP and the quite significant differences in availability of technical resources in the various countries, EMEP has been reasonably successful in achieving its objectives.

An important issue for EMEP in the coming years will be to monitor changes in air and precipitation quality in Europe as a result of emission reduction commitments laid down in the various protocols to the Convention on Long-range Tran-boundary Air Pollution. High priority will therefore be given to maintain and improve the quality of the activities related to sulphur and nitrogen oxides, as well as VOC (and photochemical oxidants). Efforts to improve the quality of the emission data are of particular importance. The measurement strategy will be evaluated, aiming at a more equal distribution of stations over the region and also considering issues such as whether all stations should have identical measurement programmes. Efforts to improve harmonisation and standardisation of methods will also have to be addressed. The modelling work will be improved by higher spatial resolution and possibly also by using multi-layer models.

E.4 AIR QUALITY INFORMATION DISSEMINATION AT EUROPEAN ENVIRONMENT AGENCY (EEA)

E.4.1 Objective and organisation

After years of preparation the European Environment Agency (EEA) was established in Copenhagen in December 1994. The main objective of EEA is laid down in the EEA Regulation (No. 121/90, EEC 1990), where EEA’s main task (Article 1) is to provide the European Community and its Member States with 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.

With regard to monitoring and information gathering, Article 2 lists the Agency’s tasks to be to establish, in co-operation with the Member States, and co-ordinate the network referred to in Article 4 (EIONET). In this context, the Agency shall be responsible for the collection, processing and analysis of data, in particular in the fields referred to in Article 3, among them ambient air quality.

The development of the EIONET technical infrastructure is of special importance. This will support its *decentralised* and *distributed organisation*, emphasising *interconnecting existing systems* so that they can exchange information and operations. The focus on interconnecting existing systems, implies neither a “central information system” nor the implementation of the same system in all nodes, but the creation of a distributed information machine. A layered approach is needed to accommodate the requirements and needs, as illustrated in fig E.4:1.

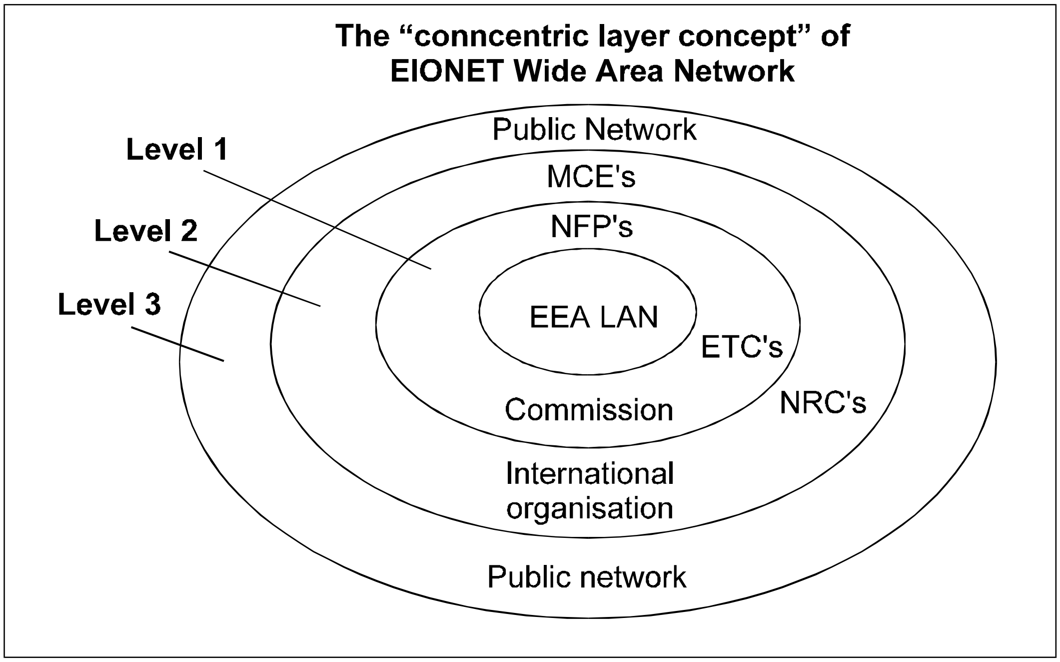


Fig E.4:1 The European Information and Observation Network (EIONET)

Level 1 is composed of EEA, National Focal Points, European Topic Centres (ETCs) and the Commission. This shell comprises the highest operating level and is intended to:

- Support efficient communication;
- Enable inter-organisational co-operation and planning;
- Provide capabilities to track the flow of documents;
- Offer "conference" facilities;
- Enable reporting and data flows; and
- Provide access to relevant EEA data.

So far eight of 12 Topic Centres Consortia have been established. For air pollution two Topic Centres were established in December 1994, the ETC-Air Quality (AQ) and ETC-Air Emissions (AEM). The ETC-AQ is lead by the National Institute of Public Health and Environment (RIVM) in the Netherlands and the other members of the Consortium are Norwegian Institute for Air Research (NILU), Norway, National Observatory of Athens (NOA), Greece, and Norwegian Meteorological Institute (DNMI), Norway.

Other main tasks of EEA are to:

- Provide the Community and the Member States with objective information necessary for framing and implementing sound and effective environmental policies; in particular to provide the Commission with the information that it needs to successfully carry out its tasks of identifying, preparing and evaluating measures and legislation in the field of the environment;
- Record, collate and assess data on the state of the environment, to draw up expert reports on the quality, sensitivity and pressures on the environment within the territory of the Community, to provide uniform assessment criteria for environmental data to be applied in all Member States. The Commission shall use this information in its task of ensuring the implementation of Community legislation on the environment;

- Help ensure that environmental data sets at an European level are reciprocally comparable and, if necessary, to encourage by appropriate means improvement of harmonisation of measurement methods;
- Promote the incorporation of European environmental information into international environment monitoring programmes such as those established by the United Nations and its specialised agencies;
- Ensure the broad dissemination of reliable environmental information. In addition, the Agency shall publish a report on the state of the environment every three years;
- Stimulate the development and application of environmental forecasting techniques so that adequate preventive measures can be taken in good time;
- Stimulate the development of methods of assessing the cost of damage to the environment and the costs of environmental prevention, protection and restoration policies;
- Stimulate the exchange of information on the best technologies available for preventing or reducing damage to the environment; and
- Co-operate with the bodies and programmes referred to in Article 15.

E.4.2 European Policies on Air Quality Monitoring

Current European directives and conventions requiring monitoring of air quality and deposition are mentioned below. In addition the national requirements of each state must also be considered.

The current set of air quality directives in the European Union comprises the compound-specific directives for SO_2, particulate matter/black smoke, NO_2, lead and ozone, of which lead was the first (1982) and ozone the last (1992). These directives require in principle that every occasion where any of the limit values is exceeded shall be detected, and thus that monitoring is carried out in all areas where this is expected to occur.

The Exchange of Information (EoI) decisions (1982 and 1994) provide the framework to make monitoring data from selected sites regularly (annually) available to the Commission.

Under the new directive on Ambient Air Quality Management (the “Framework” directive, Council Directive 95/9514/EC) new air quality target values are being developed by the Daughter Directives Working Groups for SO_2, NO_2, lead and particulate matter (PM), and new requirements to air quality assessment and monitoring are being developed concurrently. Also here, full monitoring/ assessment coverage of areas where target values are approached or exceeded, is required.

These directives and the EoI decisions will set the stage for European air quality monitoring and assessment, and reporting in the years to come.

The monitoring requirements under these directives will mainly relate to urban or industrial hot spot areas where concentrations may be high enough to affect human health. In other (rural, regional) areas, monitoring is required mainly due to effects of deposition of acidic compounds (see below).

E.4.3 Present status of European Air Quality Monitoring

The first State of the Environment report for Europe, initiated by the pan-European Conference of Environment Ministers at the Dobris Castle of the Czech Republic in 1991, and published by the EEA in 1995, took a pan-European approach, naturally enough. It is likely that the area of interest of the Agency will be extended to cover those countries covered by the Dobris assessment, and thus also attain a pan-European outlook.

On the *local/urban scale*, the Dobris report provided information on air quality in more than 100 European cities, mainly for the years 1985 and 1990. The assessment covered SO_2 and particles ("winter smog compounds"), NO_2 and O_3 ("summer smog compounds"), and also to some extent CO and lead. The data coverage was good for SO_2 and particles (mainly black smoke), but less extensive for the other compounds. Data were collected and presented both for high short-term concentrations, and long-term averages. As an example, fig E.4:2 shows a summary of the state for SO_2.

The six-page summary of the local/urban air pollution in the Dobris report was followed up by a double-volume Scientific Background Document for local/urban air pollution, presenting extensive summaries, and two to five pages city reports, one for each of the cities [94].

In 1995 the ETC.-AQ also published a status description on *air quality in Europe in 1993*. This was produced under the MA 2 project of the Agency's work programme [95].

Data were thus made available from 91 monitoring sites (in 49 cities) from nine EU Member States. In order to extend this fairly limited database, available national reports on air quality of 1993 were also used. This extended the database to six more European countries. As an example, fig E.4:3 shows SO_2.

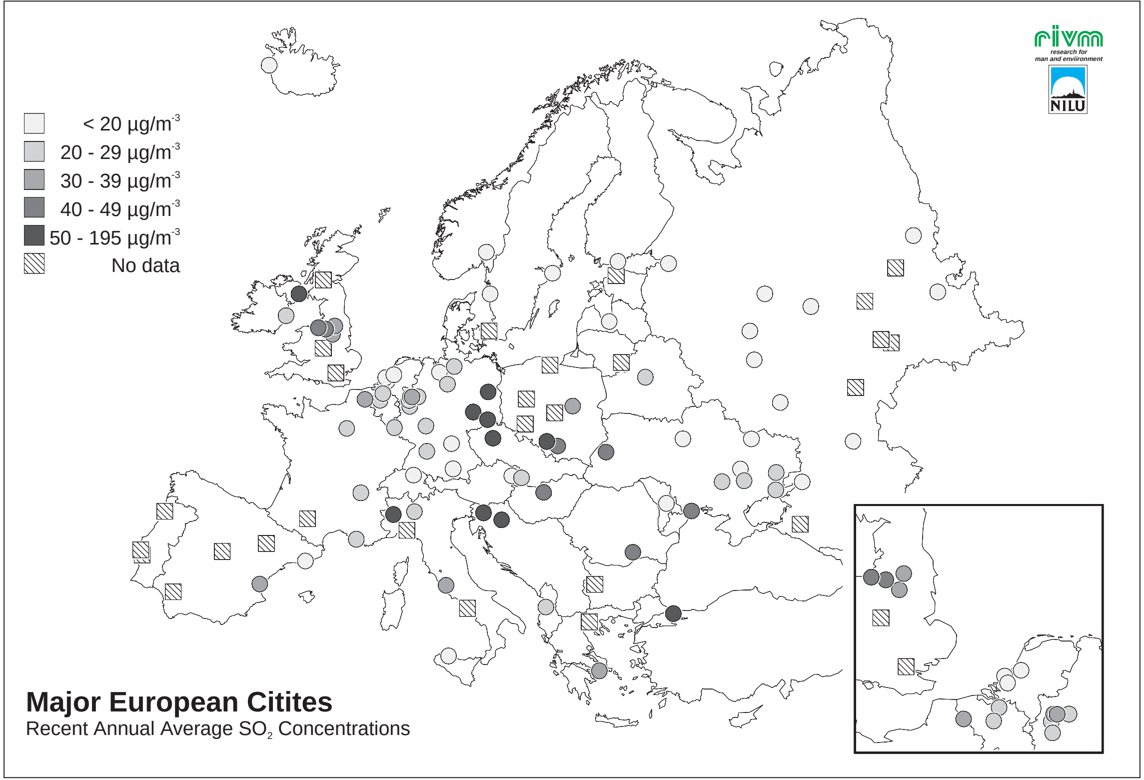
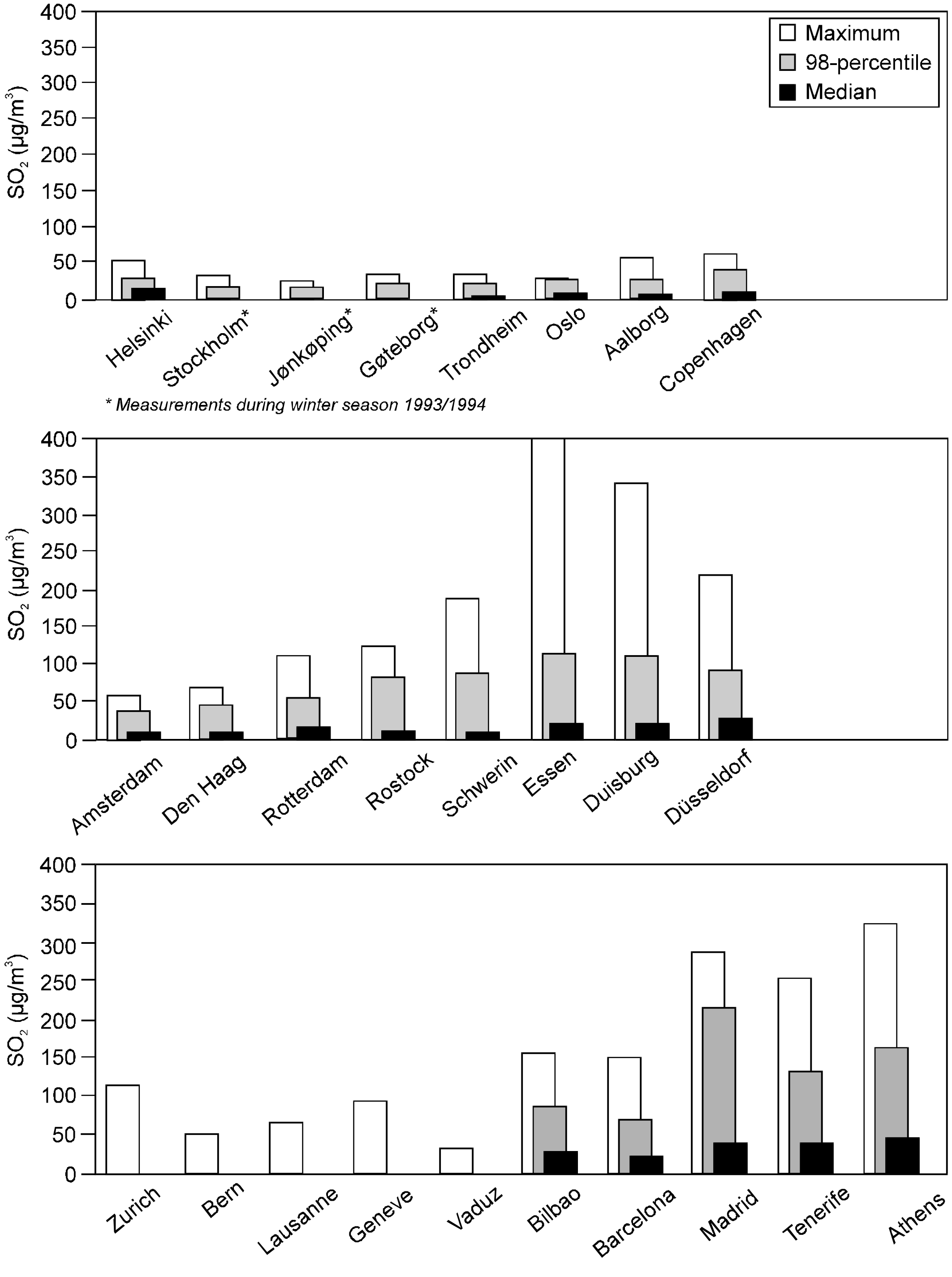


Fig E.4:2 SO_2 concentrations (annual average) in European cities, recent years (mostly 1985) [94]



** Measurements during winter season 1993/1994*

Fig E.4:3 24 h maximum, 98 percentile and median SO$_2$ values for 1993 for selected stations and cities (µg/m^3)

One of the problems encountered when using data from such varied sources, was that data presentations and statistics parameters in the data sources were not harmonised. The status description was based on presentation of averaging times and percentiles of concentrations as they appear in EU Directives and WHO Air Quality Guidelines. These parameters were not, in general, available from the national reports.

For *ozone* specifically, the ETC-AQ in 1995 produced for the European Commission a report summarising the 1994 pollution situation regarding ozone in EU Member States [96].

The report aimed to summarise the state of the ozone situation relative to the EU Ozone Directive. Ozone data were collected through a dedicated procedure, where Member States transferred the data to the Commission according to the requirements of the Directive, from which they were made available to the ETC.-AQ. Data were thus made available from 461 monitoring sites in 14 Member States, for 1994.

The various ozone threshold values as defined in the Directive are shown in Table E.4:1.

Table E.4:1 Thresholds for ozone concentrations in air

Thresholds	Mean values
Health protection threshold	110 μg/m^3 over 8 h
Vegetation protection threshold	200 μg/m^3 over 1 h 65 μg/m^3 over 24 h
Population information threshold	180 μg/m^3 over 1 h
Population warning threshold	360 μg/m^3 over 1 h

As an example, fig E.4:4 shows, based on hourly values, the number of days when the 180 μg/m^3-threshold value was exceeded. If ozone threshold values for damage to building materials could be established, which seems likely from the UN ECE dose-response functions (see section C.4), the necessary data will be easily accessible from these public sources. In this context it is quoted from Kucera [66]: “For the purpose of the mapping manual, an acceptable level of ozone of $20 \cdot 10^{-9}$ (24 μg/m^3) as an annual mean has been chosen for direct effects on organic materials. The value, which is based on a few available damage functions for organic materials, has to be considered as very provisional.”

E.4.4 European air quality monitoring networks

As part of the MA 1 project of the Agency’s work programme, the ETC.-AQ in 1995 summarised the state of the air pollution-monitoring situation in Europe (the MA 1-2 sub project). The information on monitoring networks and sites was collected through a questionnaire, and additional information was provided through a similar activity carried out by the WHO collaborating centre on Air Quality Management and Air Pollution Control, the Institute for Water, Soil and Air Hygiene (WABOLU) in Berlin.

The MA 1-2 report provides detailed country-wise tables on networks, sites, compounds, reporting etc., summarised into country reports, and again condensed into summary tables covering all the 29 countries from which data were available.

On the *regional scale*, the MA 1-2 activity revealed that there is extensive monitoring in addition to the EMEP network, and that 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, while monitoring of ozone precursors is so far limited to seven countries.

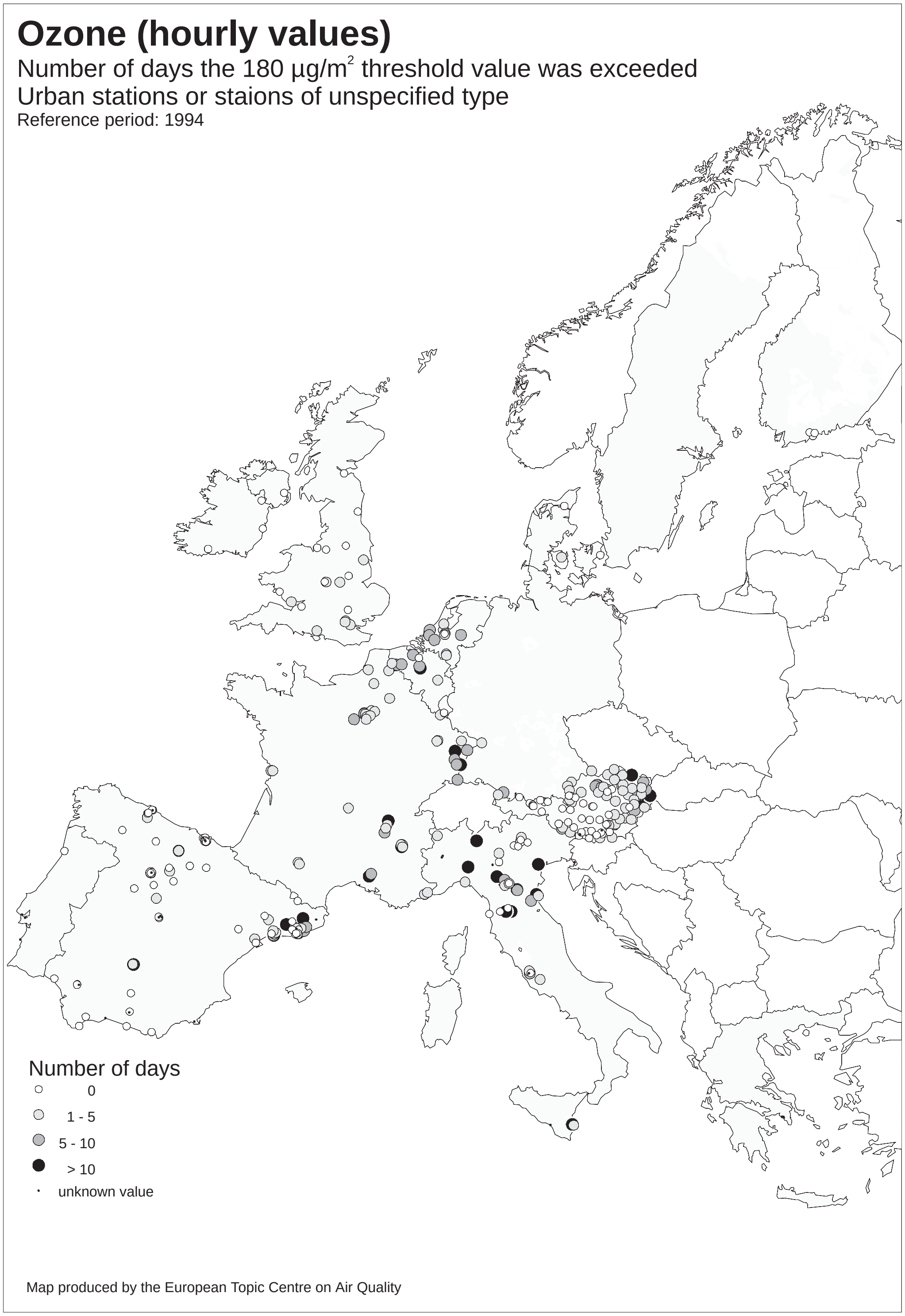


Fig E.4:4 Number of days the 180 µg/m^3 ozone threshold value was exceeded in 1994 [97]

On the *local/urban scale*, monitoring is carried out at a very large number of sites in Europe, totalling close to 5000 sites according to the information made available to the Topic Centre (see fig E.4:5). Most of these sites seem to be general urban background sites, while hot-spot sites (traffic, industry) are less well represented. Re. compounds, the compounds of the EU Directives (SO_2, particles, NO_2, ozone, lead) are extensively covered.

These 5000 local/urban monitoring sites are contained in a large number of networks operated by local, regional or national authorities. We can only assume that operating practices, quality control procedures, data availability and reporting vary considerably from network to network.

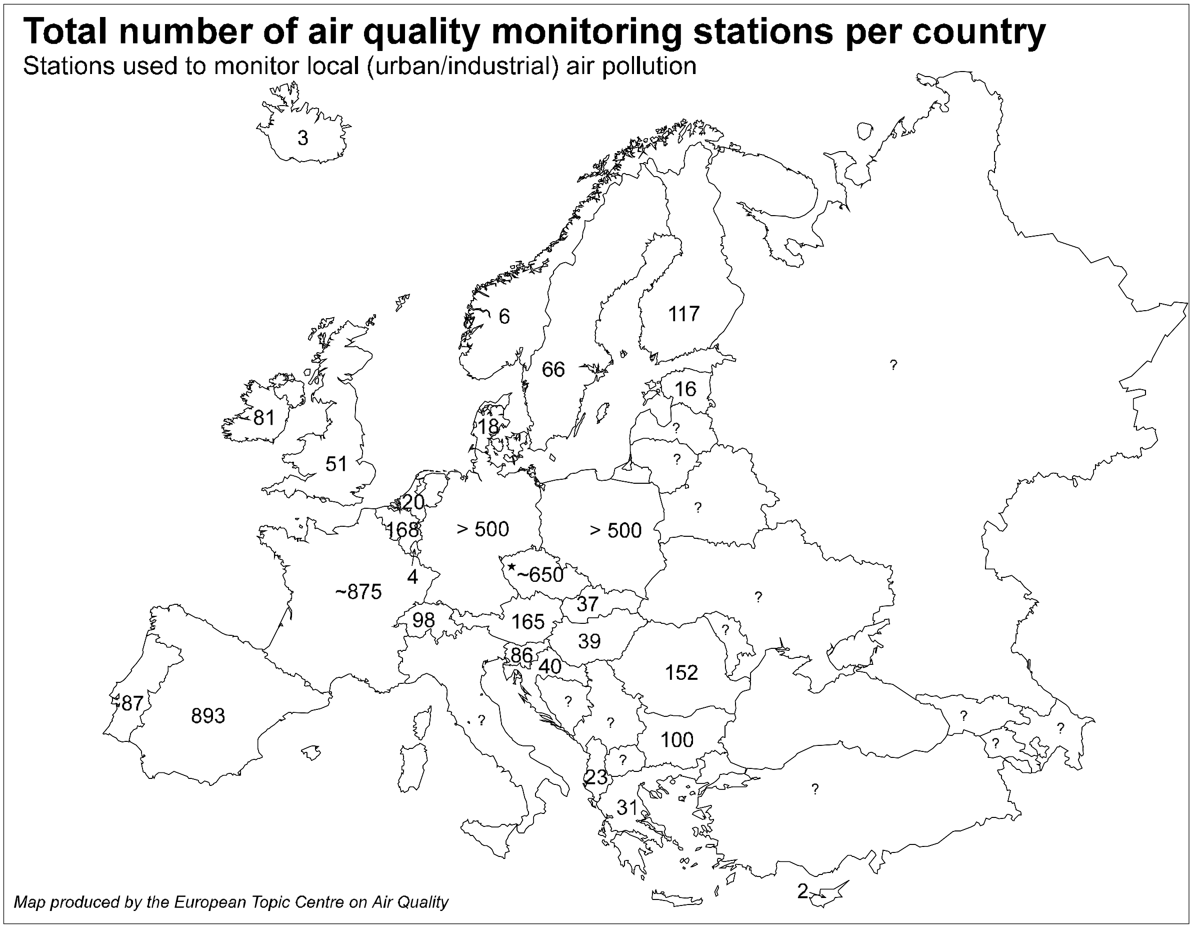


Fig E.4:5: Number of sites per country for the monitoring of urban/local industrial air pollution

E.5 INTERNATIONAL STANDARDS FOR AIR QUALITY

The European Committee for Standardisation (CEN) has assumed responsibility to its technical committee TC 264 Air Quality for working out European standards for the monitoring of emissions, loads and levels of pollutions. This work was started in 1991, and no standards are yet published (March 1996). However, many standards have been elaborated within the ISO/TC 146 "AIR QUALITY", as follows from Table E.5:1.

Table E.5:1 ISO/TC 146 "Air quality"

ISO/TC 146 "AIR QUALITY"

ISO 7708:1995

SC 1 "Stationary source emissions"	**SC 2 "Workplace atmospheres"**	**SC 3 "Ambient atmospheres"**
ISO 7934:1989 Stationary source emissions - Determination of the mass concentration of sulphur dioxide - Hydrogen peroxide/barium perchlorate/Thorin method	ISO 8518:1990 Workplace air - Determination of particulate lead and lead compounds - Flame atomic absorption spectrometric method	ISO 4219-1979 Qualité de l'air - Détermination des composés soufrés gazeux dans l'air ambiant - Appareillage d'échantillonnage
ISO 7935:1992 Stationary source emissions - Determination of the mass concentration of sulphur dioxide - Performance characteristics of automated measuring methods	ISO 8672:1993 Qualité de l'air - Détermination de la concentration en nombre de fibres inorganiques en suspension dans l'air par microscopie optique en contraste de phase - Méthode du filtre à membrane	ISO 4220-1983 Ambiant air - Determination of a gaseous acid air pollution index - Titrimetric method with indicator or potentiometric end-point detection
ISO 9096:1992 Stationary source emissions - Determination of concentration and mass flow rate of particulate material in gas-carrying ducts - Manual gravimetric method	ISO 8760:1990 Workplace air - Determination of mass concentration of carbon monoxide - Method using detector tubes for short-term sampling with direct indication	ISO 4221-1980 Air quality - Determination of mass concentration of sulphur dioxide in ambient air - Thorin spectrophotometric method
ISO 10155:1995 Stationary source emissions - Automated monitoring of mass concentrations of particles - Performance characteristics, test methods and specifications	ISO 8761:1989 Workplace air - Determination of mass concentration of nitrogen dioxide - Method using detector tubes for short-term sampling with direct indication	ISO/DIS 4222.2: 1980 Air quality - Measurement of atmospheric dustfall - Horizontal deposit gauge method
ISO 10396:1993 Stationary source emissions - Sampling for the automated determination of gas concentrations	ISO 8762:1988 Workplace air - Determination of vinyl chloride - Charcoal tube/gas chromatographic method	ISO 6767:1990 Ambient air - Determination of the mass concentration of sulphur dioxide - Tetrachloromercurate (TCM)/pararosaniline method
ISO 10397:1993 Stationary source emissions - Determination of asbestos plant emissions - Method by fibre count measurement	ISO 9486:1991 Workplace air - Determination of vaporous chlorinated hydrocarbons - Charcoal tube/solvent description/gas chromatographic method	ISO 6768-1985 Ambient air - Determination of the mass concentration of nitrogen dioxide - Modified Griess-Saltzman method
ISO 10780:1994 Stationary source emissions - Measurement of velocity and volume flow-rate of gas streams in ducts	ISO 9487:1991 Workplace air - Determination of vaporous aromatic hydrocarbons - Charcoal tube/solvent desorption/gas chromatographic method	ISO/DIS 6769:1983 Ambient air - Determination of the mass concentration of hydrogen sulphide in ambient air - Methylene blue spectrometric method

ISO/TC 146 "AIR QUALITY"

ISO 7708:1995

SC 1 "Stationary source emissions"	**SC 2 "Workplace atmospheres"**	**SC 3 "Ambient atmospheres"**
ISO/DIS 10849.2:1994 Stationary source emissions - Determination of the mass concentration of nitrogen oxides - Performance characteristics of automated measuring systems	ISO/DIS 9976:1994 Workplace air - Determination of concentrations of C_3 to C_{10} hydrocarbons - Sorbent tube/thermal desorption/capillary gas chromatographic method	ISO 7996-1985 Ambient air - Determination of the mass concentration of nitrogen oxides - Chemiluminescence method
	ISO/DIS 9977:1994 Workplace air - Determination of acrylonitrile - Pumped sorbent tube/thermal desorption/gas chromatographic method	ISO 8186:1989 Ambient air - Determination of the mass concentration of carbon monoxide - Gas chromatographic method
	ISO/DIS 11041:1994 Workplace air - Determination of particulate arsenic, arsenic compounds and arsenic trioxide vapour - Method by hydride generation and atomic absorption spectrometry	ISO 9835:1993 Ambient air - Determination of a black smoke index
		ISO 10313:1993 Ambient air - Determination of the mass concentration of ozone - Chemiluminescence method
		ISO/DIS 10529:1993 Ambient air - Determination of the mass concentration of gaseous and soluble particulate fluorine-containing compounds - Method by filter sampling and ion-selective electrode analysis
	ISO/DIS 11174:1994 Workplace air - Determination of particulate cadmium and cadmium compounds - Flame and electrothermal atomic absorption spectrometric method	ISO 9855:1993 Ambient air - Determination of the particulate lead content of aerosols collected on filters - Atomic absorption spectrometric method
		ISO 10312:1995 Ambient air - Determination of asbestos fibres - Direct-transfer transmission electron microscopy method

Table E.5:1 cont.

ISO/TC 146 "AIR QUALITY" *ISO 7708:1995*		
SC 4 "General aspects"	**SC 5 "Meteorology"**	**SC 6 "Indoor air"**
ISO 4225:1994 Air quality - General aspects - Vocabulary ISO 4226:1993 Air quality - General aspects - Units of measurement ISO/TR 4227:1989 Planning of ambient air quality monitoring ISO 6879-1983 Air quality - Performance characteristics and related concepts for air quality measuring methods ISO/DIS 6879.2:1994 Air quality - Performance characteristics and related concepts for air quality measuring methods ISO 7168-1985 Air quality - Presentation of ambient air quality data in alphanumerical form ISO 8756:1994 Air quality - Handling of temperature, pressure and humidity data ISO 9169:1994 Air quality - Determination of performance characteristics of measurement methods ISO 9359:1989 Air quality - Stratified sampling method for assessment of ambient air quality		

E.6 MEASURING DEVICES FOR CONTINUOUS EMISSION MONITORING

Continuous monitoring of emissions from plants requiring an operating licence is increasingly being entrusted to systems that can measure concentrations of several substances successively. This represents a major saving in investment and operating costs compared to devices that measure concentrations of only one substance.

The list of suitable devices currently available (Table E.6:1) shows that they mostly depend on optical measuring principles spread across the part of the electromagnetic spectrum between infrared and ultraviolet.

Table E.6:1 Multi-component measuring systems used for continuous emission monitoring

Measuring principle	Substances	Instrument type	Method
NDIR	CO, NO, SO_2, O_2	URAS 10 E	Electrochemical cell
non-dispersive infrared absorption	CO, NO, SO_2	URAS 10 P	
	CO, NO, O_2	ENDA 1000	O_2 magnetopneumatic
	CO, NO, SO_2, O_2	ENDA 1000	
	HCl, H_2O	MCS 100 HW	Single-beam bifrequency
VIS-FIR	NH_3, CO_2		Gas correlation
visible spectrum absorption	HCl, CO, NO, NO_2	MCS 100 HW	Single-beam bifrequency
	SO_2 , H_2O		
	CO, NO, NO_2, SO_2	MCS 100 HD	Gas filter correlation
NDUV	NO, SO_2	RADAS 2	
non-dispersive ultraviolet absorption	NO, SO_2 , dust concentration	GM 30	
	NO, SO_2	GM 30-2	
DOAS	SO_2, NO, NO_2		
differential absorption spectroscopy	SO_2, NO, NO_2	OPSIS	
	NH_3, H_2O		
	Phenols, formaldehyde, Hg(O)		
Electrical conductivity	HCl, SO_2	Microgas, HCl/SO_2 Air TI	
Pressure loss/ differential pressure	Dust concentration, exhaust gas volume	LPS-E	
Electrochemical monitoring points	CO, NO, O_2	MSI 5600 heating oil EL F	

The most frequently used techniques are non-dispersive infrared absorption (NDIR), visible spectrum absorption (VIS), non-dispersive ultraviolet absorption (NDUV) and differential absorption spectroscopy (DOAS) in the UV to IR range, while electrical conductivity and electrochemical cells are used only sporadically.

The DOAS principle has proved particularly versatile, since it is able to measure both traditional components such as sulphur dioxide, nitrogen monoxide, nitrogen dioxide, carbon monoxide and ammonia in the exhaust gas of plants fitted with catalytic denitrification devices, and metallic mercury in the cleaned gas from waste incineration plant. The DOAS system is also suitable for measuring phenol and formaldehyde emissions in the manufacture of rock wool.

E.7 MEASURING DEVICES FOR CONTINUOUS MONITORING OF AMBIENT AIR QUALITY

The levels of air pollutants can nowadays be monitored with automatic instruments that allow continuous recording and the identification of peak loads. Table E.7:1 gives an overview of parameters and available monitor principles and specifications.

Table E.7:1 Specification of instruments for continuous monitoring of air pollutants

Parameter	Measuring parameters	Unit	Span	Detection limit	Accuracy	Period
NO	Chemiluminescence	µg/m^3	$0\text{-}2\cdot10^{-6}$	$2\cdot10^{-9}$	1 %	10 s
NO_2		µg/m^3	$0\text{-}2\cdot10^{-6}$	$2\cdot10^{-9}$	1,4 %	10 s
NO_x		µg/m^3	$0\text{-}2\cdot10^{-6}$	$2\cdot10^{-9}$	1 %	10 s
SO_2	UV-fluorescence	µg/m^3	$0\text{-}0{,}5\cdot10^{-6}$	$5\cdot10^{-9}$	1 %	10 s
CO	IR-absorption	mg/m^2	$0\text{-}100\cdot10^{-6}$	$100\cdot10^{-9}$	1 %	10 s
O_3	UV-absorption	µg/m^3	$0\text{-}250\cdot10^{-9}$	$0{,}6\cdot10^{-9}$	$1\cdot10^{-9}$	10 s
Particles	TEOM resonance frequency	µg/m^3	0-2 g/m^3	5 µg/m^3	5 µg/m^3	10 s
CH_4	Flame chromatography	$\times10^{-6}$	$0\text{-}10\cdot10^{-6}$	$0{,}05\cdot10^{-6}$		10 s
VOC		$\times10^{-6}$	$0\text{-}10\cdot10^{-6}$	$0{,}05\cdot10^{-6}$		10 s
PAH	Photoionisation	$\times10^{-9}$	$0\text{-}2\cdot10^{-6}$	$1\cdot10^{-9}$		10 s

Passive samplers are a useful alternative for non-continuous sampling and, more importantly, provide a simple and inexpensive guide to average loads throughout an investigation area. These samplers are also very useful for the mapping of micro-environmental loading, for example on a building. They are therefore described in more detail in section G.

E.8 AIR QUALITY INFORMATION AND MANAGEMENT SYSTEMS

Surveillance and management of air quality can now be facilitated and performed via total information systems. The Air Quality Information System, AirQUIS, represents the air pollution part of a modern Environmental Surveillance and Information System, ENSIS, developed and demonstrated during the Winter Olympic Games, 1994 in Lillehammer [98].

The AirQUIS system was developed by institutions dealing with air pollution, information technology and geographical information systems (GIS). The combination of on-line data collection, statistical evaluations and numerical modelling enable the user to obtain information and carry out forecasting and future planning of air quality. The system can be used for monitoring and to estimate environmental impacts from planned measures to reduce air pollution.

The AirQUIS system contains the following modules, see fig E.8:1:

- On-line monitoring;
- Data handling and quality control;
- Modern data bases;
- Emission inventories;
- Numerical dispersion models;
- Generation of wind fields;
- Presentations based upon a geographical information system (GIS); and
- Exposure models for materials and human health.

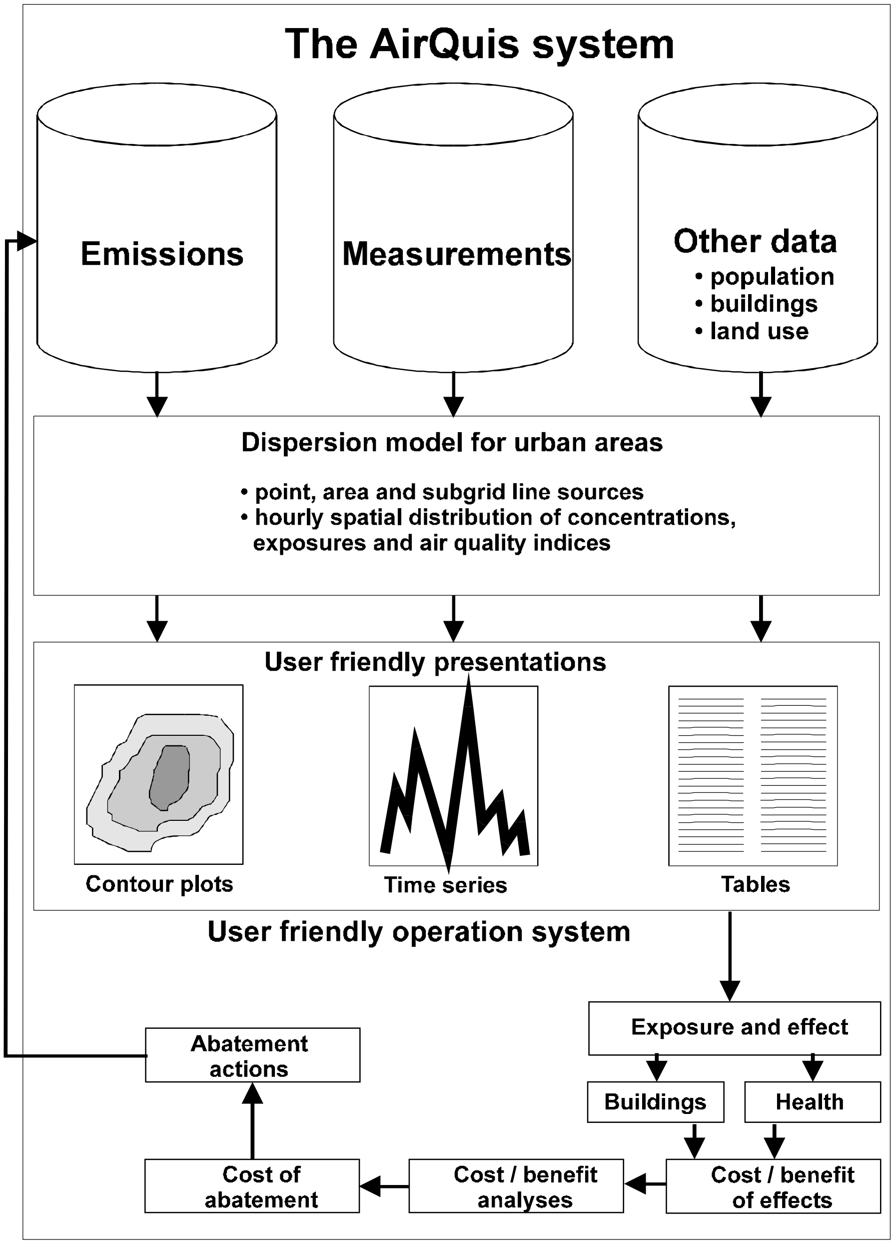


Fig E.8:1 The AirQUIS system

E.8.1 On-line measurement system

A measurement system of modern on-line sensors for selected air pollution indicators can be designed specifically for the area concerned. An appropriately designed data logger for meteorology and air quality is included in the system. The measurements are automatically transferred from the monitoring sites to a central database for quality control.

Data quality control is performed at different levels throughout the data collection process, and by approval at the final storage database, where simple statistics and data graphics are used to check the validity and representatively of the data.

E.8.2 The emission inventory data base

A modern database for the air pollution emission inventory has been developed. The emission module is a flexible system containing a user friendly map-oriented interface to treat the main sources for emission to air, such as industry, traffic, energy (consumption of fossil fuels) and emissions related to other mobile sources such as airport and harbour activities.

The industry emission module allows the user to select sources related to specific activities or areas. The time variation of emissions can be entered specifically for each source or for groups of sources. Based on emission factors, emissions can be calculated from consumption data.

The traffic module, which is the most complex part of the emission module, includes road types and facades, vehicle type distribution, traffic time variation and emission factors dependent on parameters such as vehicle type, traffic speed and road type.

The emission inventory database is developed for applications in databases such as Access on PC as well as Sybase and Oracle on Unix workstations.

E.8.3 Statistical and numerical models

The models included in the AirQUIS system cover air pollution on all scales; traffic in street canyons and along roads, industrial emissions, pollution from household etc. within the urban areas and on a regional scale.

The NILU-developed source-oriented numerical dispersion model EPISODE calculates spatial distribution of hourly concentrations of SO_2, NO_x, NO_2 and suspended particles. The NILU models ROADAIR and CONTILENK are used to estimate sub grid concentrations close to roads within the square grid. This is of relevance for estimating the microenvironment exposure on buildings and it will therefore be described more in dept in section G. A puff-trajectory model is used to calculate the influence of point sources.

All model results are displayed using GIS, such as ArcInfo/ArcView. The models are running on a Unix workstation. The presentation of results can be carried out both on PC and Unix platforms.

E.8.4 The geographical information system

A geographical information system based upon ArcInfo and ArcView is used as a platform to integrate the presentation of measurements, emission inventory and results from model estimates. The geographical information system is directly linked to the databases, by which statistical evaluations, graphical presentations and spatial distributions of emissions and model results can be presented.

E.8.5 Effect modules and air quality planning

One main application of the AirQUIS system will be as an effective tool for air quality abatement strategy. The contribution of air pollution from different source categories such as traffic, household and industry to the population and building exposure in an urban area can be calculated based upon data on emissions, dispersion and distribution of buildings and population. Different recommended measures to reduce air pollution can be evaluated with the respect to exposure of population and buildings and cost-benefit/efficiency analyses. A priority list of the selected measures can be developed, taking into account air pollution exposure, health aspects and related costs.

The module for modelling and calculating buildings degradation, service lives and maintenance costs is called CorrCost and is further described in an application used in Oslo [99] (see section F.2).

F Classification and mapping of environmental degradation factors and corrosivity

F.1 MAINTENANCE AND ENVIRONMENTAL ASPECTS IN CORROSIVITY MAPPING

The classification employs *two* approaches: classification in terms of the corrosion-determining *environmental* parameters and/or classification based on *corrosion rate measurements of standard samples* exposed in the environment concerned, see fig F.1:1.

The *standard specimen* approach was dominant until the last decade due to little available knowledge of and data on dose-response functions and degradation factors. Originally, this corrosivity mapping was undertaken responding to the needs for better maintenance, and in order to design the best protective systems for constructions. Most of the various national classification and mapping systems served this objective and used this approach. Since this method does not characterise and map the degradation factors, the approach is strictly valid only for the period of the measurements, i.e. it is not explanatory and predicative. The standard specimen approach will therefore only be briefly referred to here.

The standards ISO 9223-26 [18] also have improved maintenance as a prime objective. These standards represent a huge step forward as they for the first time in this context describes a system for quantitative characterisation and classification of the important environmental degradation factors. However, the practical use has so far been impeded by the lack of data and user-friendly IT tools. The approach can also be used for other types of materials, and such standards are beginning to emerge.

Today, when data are becoming more known from the environmental research area, efforts have been undertaken to categorise, classify and map the degradation factors and corrosivity of the exposure environment.

The environmental concern and the strive for sustainable development in the recent years have also generated a need for mapping the corrosivity of the exposure environment, in order to provide input to environmental regulations. This is done in *two* separate ways. The first one uses corrosivity mapping for proper cost-benefit analysis of corrosion costs [22], while the other is based on the “critical-load/level approach” (see F.4). This methodology, which is elaborated within the UN ECE Working Group on Effects, gives tools for predicting and mapping material degradation rates, and eventually service lives, related to their exposure environment. In co-operation, it could turn out to be very useful in converging the needs of the environmental issue and the building society, thereby ensuring a more durable sustainable built environment.

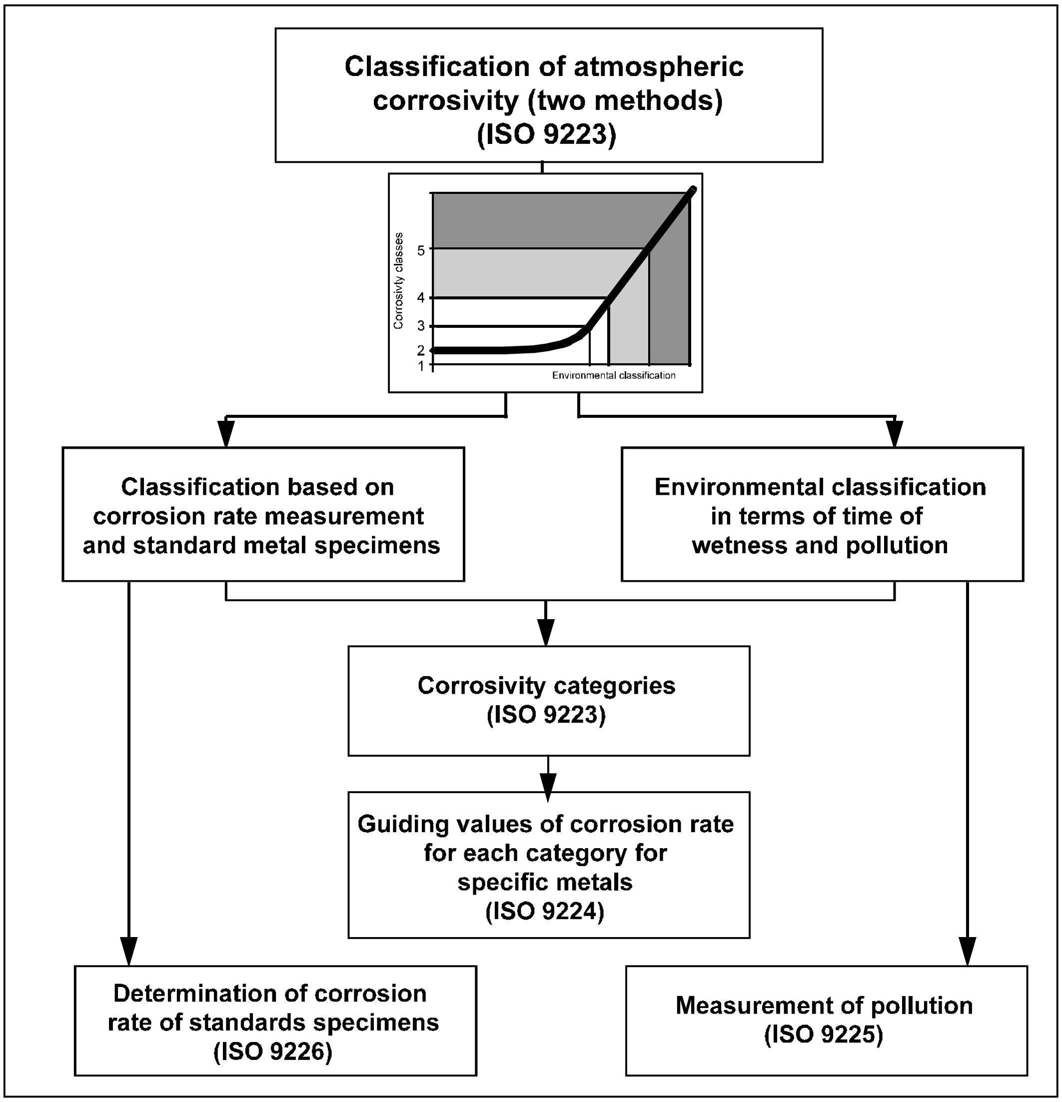


Fig F.1:1 ISO 9223-26 – the two ways of classifying atmospheric corrosivity

F.2 ISO 9223-26 CLASSIFICATION OF ATMOSPHERIC CORROSIVITY FOR METALS

The standards ISO 9223–9226 *Corrosion of metals and alloys – Corrosivity of atmospheres* have been developed for the classification of atmospheric corrosivity of metals and alloys [18]. The development was based on the Czech classification philosophy, where the approach was used as early as 1975 to map the atmospheric corrosivity for the North Bohemian region [23]. 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 approach of corrosion rates, as shown in fig F.1:1.

The series of standards is:

- ISO 9223 – Classification

Scope: Specifies 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).* Corrosivity categories (C) are defined on basis of these three factors and used for the classification of atmospheres for the metals/alloys-unalloyed steel, zinc and copper, and aluminium. TOW is described in five classes, classification of SO_2 and chloride is made in four classes, and the corrosivity in five classes. 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.

- ISO 9224 – Guiding values for the corrosivity categories

Scope: Specifies guiding corrosion values and the characteristics of corrosion for the corrosivity categories defined in ISO 9223. This can be used to predict the service life for metals/alloys in atmospheres corresponding to different corrosivity categories. These values also provide a technical basis for determining the need for protective measures and other engineering purposes. The values are based on evaluation of a large number of site exposures and in-service performance.

- ISO 9225 – Measurements of pollution

Scope: Specifies three methods for measuring the deposition rates of sulphur dioxide and airborne salinity. It does not cover concentration measurements, which are covered by ISO 4221:1980. The methods apply for characterisation of the test site, and as such are also suitable for characterisation of the microenvironment on buildings, see section G. The sulphur dioxide methods are based on lead dioxide (PbO_2) and alkaline surfaces, and for chloride deposition the wet candle method is specified.

- ISO 9226 – Determination of corrosion rate of standard specimens for the evaluation of corrosivity.

Scope: Specifies the methods for determination of corrosivity by standard specimens (flat plate or helix), including characterisation of materials.

The ISO 9223 approach has since the mid 80's been used by many researchers to classify and map the atmospheric corrosivity [10,11,15,32].

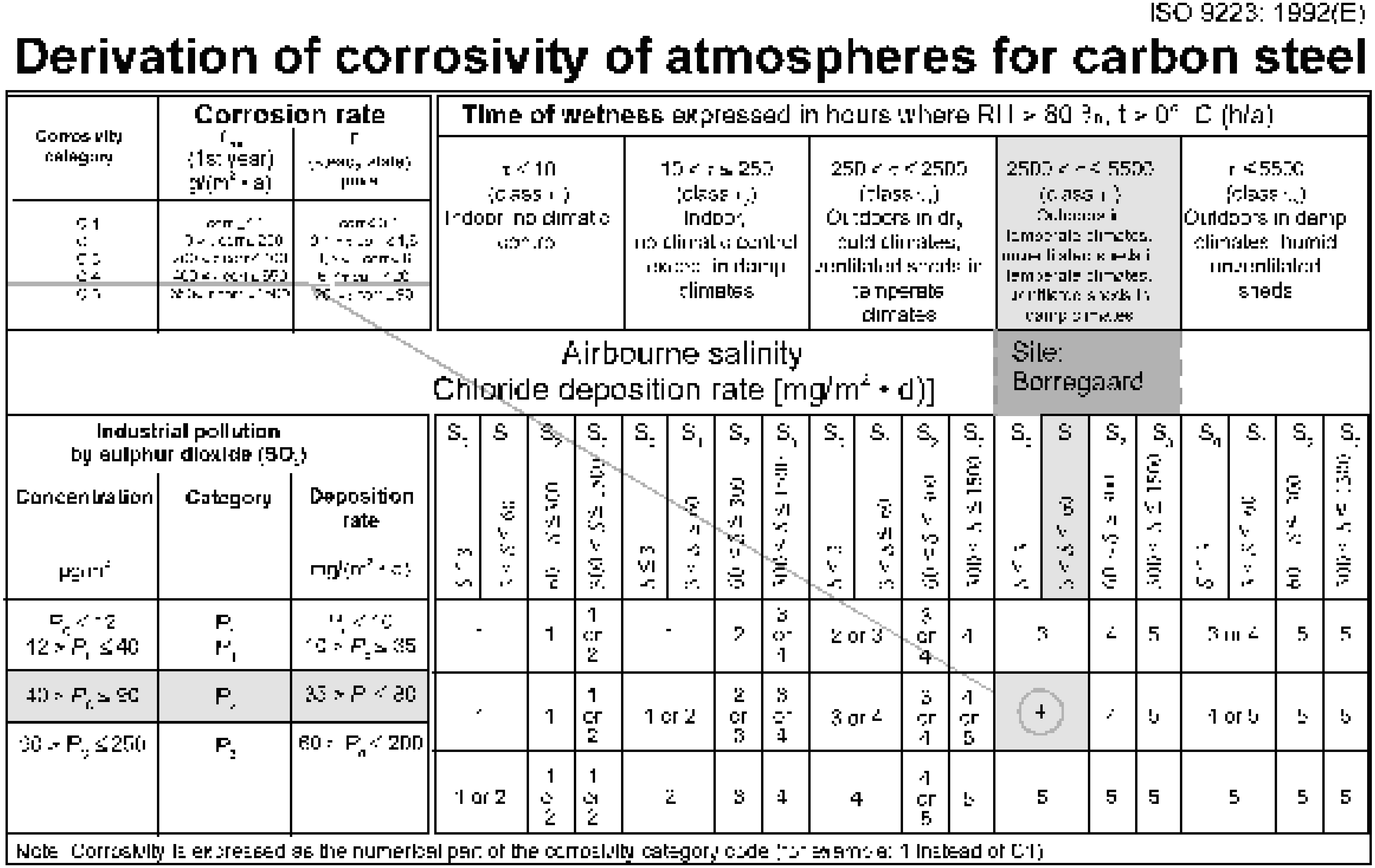
Table F.2:1 shows an example of practical use of the standard. In this Table all the classes/and values for the environmental variables of TOW, SO_2 and Cl are put together with the corrosivity categories/rates, and the use for classifying the corrosion test site Borregaard in Norway by both approaches are shown. This site is one of the most aggressive in the UN ECE ICP network.

F.3 OTHER CLASSIFICATION STANDARDS

The above-mentioned standard approach for classification of the atmospheric corrosivity is primarily applicable to the corrosion of metal and alloys and cannot, without complementary amendments, be used for an appropriate description of the degradation environment for non-metals.

The ISO/FDIS 12944-2 [100] deals with the classification of the principal environments to which steel structures are exposed, and the corrosivity of these environments. It defines atmospheric corrosivity categories, based on ISO 9223 and ISO 9226 [18].

Table F.2:1 Example of the use of ISO 9223-26 to classify the atmospheric corrosion test site Borregaard in Norway



ISO 9223: 1992(E)

Derivation of corrosivity of atmospheres for carbon steel

Corrosivity category	Corrosion rate r_{corr} (1st year) g/(m² • a)	Corrosion rate r (steady state) µm/a
C1	$r_{corr} \le 10$	$r_{corr} \le 0,1$
C2	$10 < r_{corr} \le 200$	$0,1 < r_{corr} \le 1,5$
C3	$200 < r_{corr} \le 400$	$1,5 < r_{corr} \le 6$
C4	$400 < r_{corr} \le 650$	$6 < r_{corr} \le 20$
C5	$650 < r_{corr} \le 1500$	$20 < r_{corr} \le 90$

Time of wetness expressed in hours where RH > 80 %, t > 0° C (h/a)

$\tau \le 10$ (class τ_1)	$10 < \tau \le 250$ (class τ_2)	$250 < \tau \le 2500$ (class τ_3)	$2500 < \tau \le 5500$ (class τ_4)	$\tau > 5500$ (class τ_5)
Indoor no climate control	Indoor, no climate control except in damp climates	Outdoors in dry, cold climates, ventilated sheds in temperate climates	Outdoors in temperate climates, unventilated sheds in temperate climates, ventilated sheds in damp climates	Outdoors in damp climates, humid unventilated sheds

Site: Borregaard

Airbourne salinity
Chloride deposition rate [mg/(m² • d)]

Industrial pollution by sulphur dioxide (SO₂)

Concentration µg/m³	Category	Deposition rate mg/(m² • d)	τ_1 S_0 $S \le 3$	τ_1 S_1 $3 < S \le 60$	τ_1 S_2 $60 < S \le 300$	τ_1 S_3 $300 < S \le 1500$	τ_2 S_0 $S \le 3$	τ_2 S_1 $3 < S \le 60$	τ_2 S_2 $60 < S \le 300$	τ_2 S_3 $300 < S \le 1500$	τ_3 S_0 $S \le 3$	τ_3 S_1 $3 < S \le 60$	τ_3 S_2 $60 < S \le 300$	τ_3 S_3 $300 < S \le 1500$	τ_4 S_0 $S \le 3$	τ_4 S_1 $3 < S \le 60$	τ_4 S_2 $60 < S \le 300$	τ_4 S_3 $300 < S \le 1500$	τ_5 S_0 $S \le 3$	τ_5 S_1 $3 < S \le 60$	τ_5 S_2 $60 < S \le 300$	τ_5 S_3 $300 < S \le 1500$
$P_c \le 12$ $12 < P_c \le 40$	P_0 P_1	$P_d \le 10$ $10 < P_d \le 35$	-		1	1 or 2	-		2	3 or 4	2 or 3		3 or 4	4	3		4	5	3 or 4		5	5
$40 < P_c \le 90$	P_2	$35 < P_d \le 80$	1		1	1 or 2	1 or 2		2 or 3	3 or 4	3 or 4		3 or 4	4 or 5	⊕		4	5	4 or 5		5	5
$90 < P_c \le 250$	P_3	$80 < P_d \le 200$	1 or 2		1 or 2	1 or 2	2		3	4	4		4 or 5	5	5		5	5	5		5	5

Note Corrosivity is expressed as the numerical part of the corrosivity category code (for example: 1 instead of C1)

F.4 MAPPING ACCORDING TO SERVICE LIFE (AND ISO 9223) APPROACH

F.4.1 Japan

Solar UV rays, water and *heat* are common chemical and mechanical deterioration factors for many building materials used in outdoor environments. Tomiita [14,101,102] has, as described in section D, estimated the intensity or index of these factors from meteorological data. He has classified and mapped the following factors:

- Solar UV energy;
- Time of wetness (see fig F.4: 1);
- Wet-dry cycle;
- Thermal degradation based on BPT; and
- Daily BPT difference.

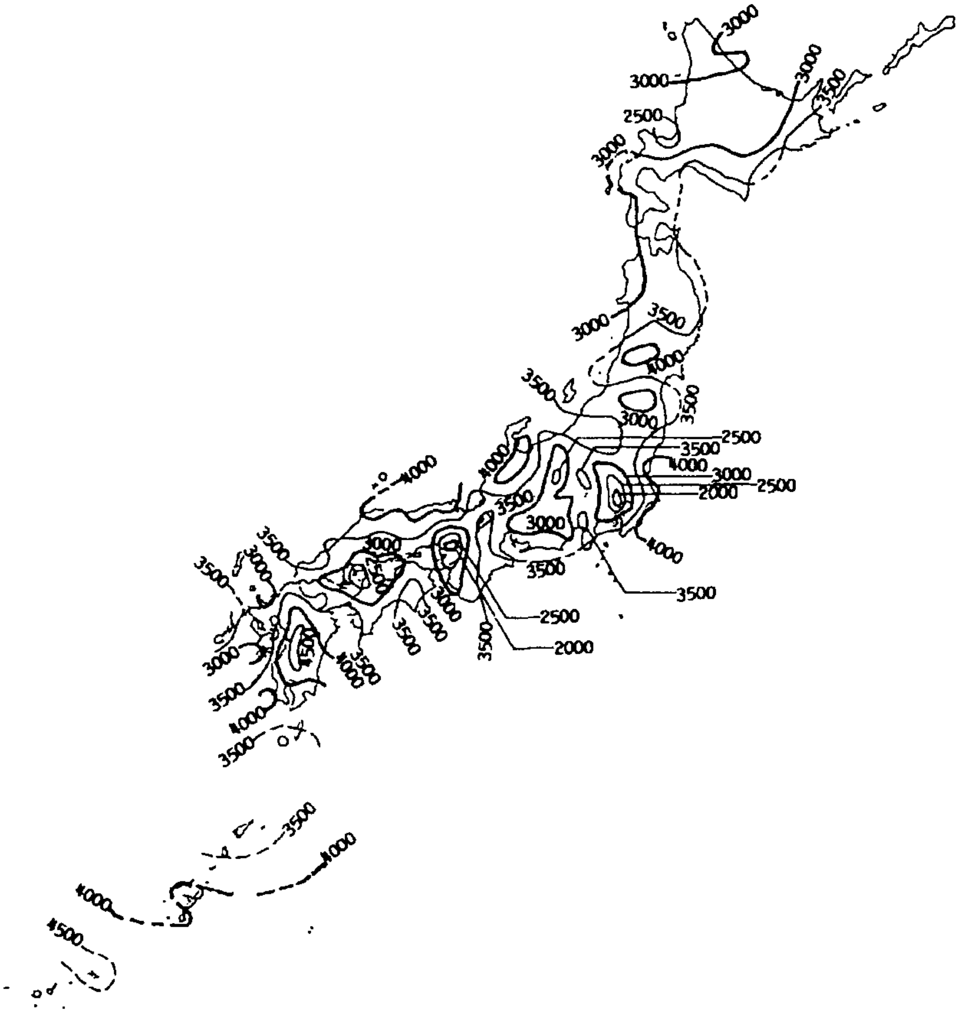


Fig F.4:1 Time of wetness (h/year) according to ISO criterion [14]

The local differences in the ways or rate of the deterioration of building materials can generally be explained by citing the maps presented, of which time of wetness is shown in fig F.4:1.

There are synergistic effects arising from multiple degradation factors. Tomiita and Kashino [58,59] have also estimated and mapped the temperature-modified time of wetness/wet-dry cycle.

F.4.2 UK – Atmospheric corrosivity values

The Ministry of Agriculture, Fisheries and Food publishes a map every five years illustrating the average atmospheric corrosivity rate for 10 km grid squares of the UK [103]. This map is based on a zinc reference database but corresponding corrosivity rates for other metals (steel, aluminium, copper and brass) can be approximately assessed using dose-response function relationships. It should also be remembered that the local microclimate could have a greater effect on durability of building materials than the average corrosivity within a section of a 10 km by 10 km square.

F.4.3 Spain – Morcillo and Feliu

Probably the most extensive corrosivity classification and mapping for any country has been performed for Spain by Morcillo and Feliu [57]. They have used both ISO classification approaches extensively (see fig F.1:1) and produced detailed classification maps for the whole country and for the various areas and cities.

They have also initiated an extension of the national programme to a large programme for South-America, called MICAT, starting in 1988, totally involving 14 countries, 70 institutions and 71 test sites.

F.4.4 The ISO-CORRAG programme

The international corrosivity-mapping programme ISO-CORRAG was prepared in the framework of activity of ISO/TC 156 WG 4 Classification of corrosivity of atmospheres, who elaborated the ISO 9223-26 standards. The programme started in autumn 1986/summer 1987, and the aim is to verify the methodological and technical aspects of the procedures included in ISO 9223-26 (at that time DIS).

The programme comprises the same type of materials and environmental variables as in the standard, and involves 46 test sites in 12 countries in Europe, Russia and North America.

Results with regression analyses for period 1987–1992 were recently reported [104].

There is a clear connection and co-operation between the three international programmes of UN ECE ICP, MICAT and ISOCORRAG, in the context of both scientific content and scientists involved.

F.4.5 USA – Scheffer’s index for wood

There are great differences in temperatures and precipitation regimes on the macro scale. This results in equally great differences in potential wood decay rates. A great need exists for mapping these risk factors. Based on Scheffer’s climatic index (see section D.5), the US Weather Bureau developed a climate index map for the US (fig F.4:3).

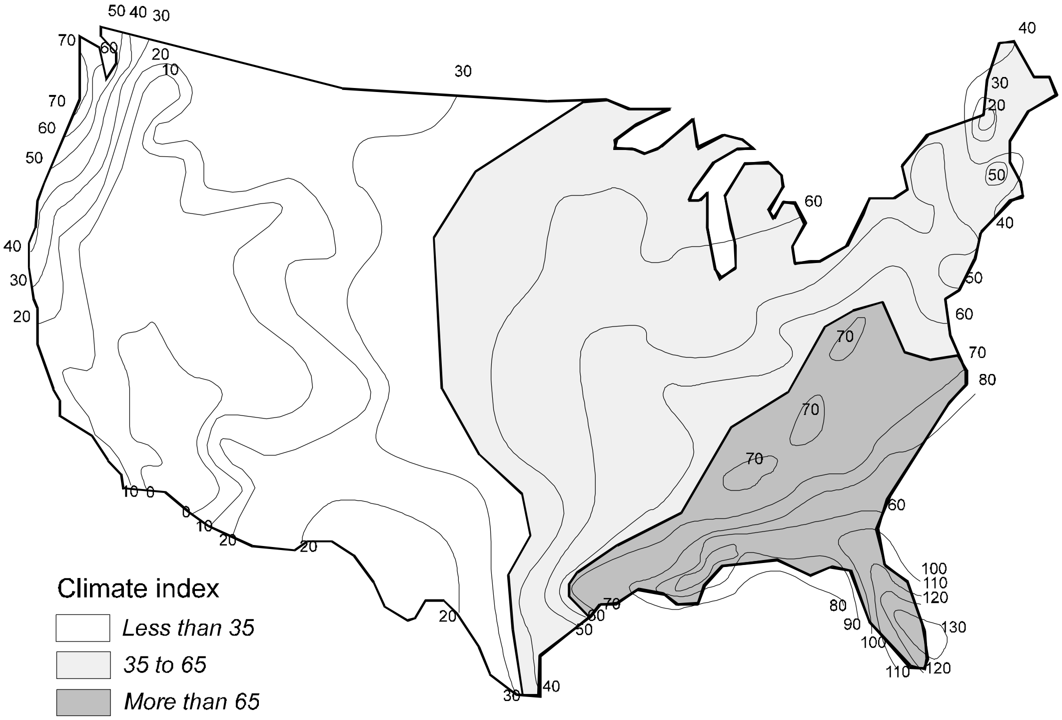


Fig F.4:3 Mapping of Scheffer’s climate index for wood decay in US produced from US Weather Bureau data [84]

F.5 ENVIRONMENTAL RESEARCH MAPPING APPROACH (UN ECE)

F.5.1 Dose-response functions

To evaluate the effect of airborne acidifying pollutants on corrosion of materials, the Executive Body for Convention on Long-Range Trans-boundary Air Pollutants decided to launch the UN ECE ICP. The programme and four-years dose-response (D-R) functions have been described in section C.4 (Table C.4:1). For unsheltered exposure most of the D-R-functions have the same form

$$ML \text{ or } MI = a + bTOW\,[SO_2][O_3] + cRain[H^+] \qquad (2)$$

According to this D-R function, the corrosion adds up contributions from natural corrosion, *a*, dry pollution corrosion, $bTOW\,[SO_2][O_3]$, and from wet pollution corrosion, $cRain[H^+]$.

From such D-R functions, mapping procedures based on the "critical/ acceptable load" concept have been developed.

F.5.2 Mapping procedure

The concept of critical load or level has been developed within the convention work in UN ECE and proved to be a useful tool in formulating abatement strategies for natural ecosystems. A critical load has been defined as "the highest deposition of a compound that will not cause chemical changes leading to harmful effects on ecosystems structure and function" [105].

As atmospheric corrosion and deterioration of materials is a cumulative, irreversible process which proceeds even in the absence of pollutants, this concept has to be modified as even the lowest concentration of pollutants cause an increase in the deterioration rate. This has resulted in the concept of *acceptable rate of deterioration*, based on technical and economic considerations. Acceptable levels of atmospheric pollutants can then be defined if dose-response relations are available, giving the relationship between the corrosion rate and the level or load of pollutants in combination with climatic parameters. The acceptable corrosion rate K_a should be related in a simple way to the background corrosion K_b, i.e. the corrosion in areas with "background" pollution.

$$K_a = nK_b \qquad (50)$$

The background corrosion rate has been defined as the lower 10 percentile (K_{10}) of the observed corrosion rate for each material in the ongoing ICP Materials exposure programme.

Table F.5:1 Four-years background corrosion rates for materials in the UN ECE ICP

Material	Four-years background corrosion rate (g/m^2)	Average background corrosion rate (μm/year)
Weathering steel	204	6,5
Zinc	17,1	0,6
Aluminium	0,67	0,06
Copper	14,5	0,4
Bronze	11,6	0,3
Limestone	44,0	3,9
Sandstone	40,0	3,9

The following concepts and maps are defined at ***level 1 mapping***:

- *Corrosion rate and acceptable corrosion rate.* Corrosion rates are mapped from dose-response functions, and *acceptable corrosion rates* are defined as being a percentage increase in the background corrosion rate. At present it is recommended that values of *n* between 1.2 and 2.0 be used.

- *Acceptable exposure load/level* is the concentration or load that does not lead to an unacceptable increase in the rate of corrosion. From this definition it is possible to calculate the acceptable pollution level from the acceptable corrosion rate and dose-response functions.

- ***Areas of exceeding of threshold values*** where the corrosion rate and the pollution level/ load are unacceptable.

- *Environmental degradation parameters*. In addition to these maps it is also recommended to map the geographical distribution of some of the most important environmental degradation parameters and their combinations.

Level 2 mapping is similar to level 1, but includes data on the *stock of material* in each mapping unit and the *economic costs* associated with corrosion of these materials (e.g. replacement or repair costs) (see F.6.2).

F.5.3 The environmental assessment level approach in UK

Butlin et al. [21] have on contract for the Department of Environment (DoE) carried out a study in accordance with DoE’s aim to establish a methodology by which it can assess the effects of a wide range of emissions on a range of receptors. In the BRE study the receptors were buildings.

The objectives of the programme were to:

1. Provide a listing of current dose-response functions for a range of substances and their relevant interactions with buildings and building components (see also section C.5);
2. Assess practical options for setting Action Levels (ALs) and Environmental Assessment Levels (EALs) based on current definitions of ALs and EALs;
3. Discuss options for further developing EALs; and
4. Recommend procedures for further developments necessary in the assessment of the effect of pollutants on buildings.

The critical-load approach needs a practical way to assess the level at which pollutants are considered harmful. In the DoE approach the assessment levels and concepts are defined as shown in fig F.5:1.

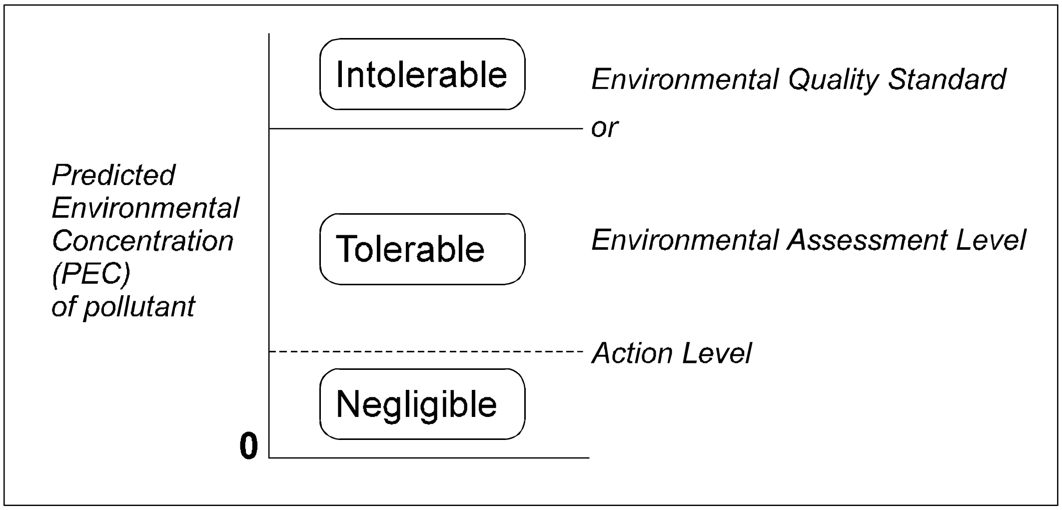


Fig F.5:1 The tolerability of releases [21]

The dose-response functions surveyed have been combined with the current thinking of this concept to provide provisional values for both ALs and EALs. The final part of the report discusses future options for the further development of the Regulatory Framework, in particular the incorporation of spatial variation and the stock-at-risk into the setting of EALs.

The development of a rational basis for determining the interim EALs should eventually lead to the establishment of “damage functions” which determine the Predicted Environmental Concentration (PEC) at which harm becomes intolerable.

Quoting from the report: “Five practical indicators have been identified as a basis for determining the EAL and which seek to introduce rational, rather than arbitrary, judgement into the determination:

1. *Threshold Indicator*. The EAL might intuitively be found where damage is measured at the turning point in a dose-response curve that is increasing at an exponential rate.
2. *Catastrophic Indicator*. The EAL might be found where damage clearly becomes catastrophic to property owners; e.g. characterised by building collapse.
3. *Social Indicator*. The EAL should be determined, not on the basis of harm to modern buildings, but on the harm caused by pollution to historic buildings and monuments. Since historic buildings are, by definition, those structures which society judges to be most deserving of protection and preservation, harm to the structure could be deemed intolerable.
4. *Liability Indicator*. The EAL might be based on a risk assessment of damage that reflected standards in liability law and in building regulations. Liability could be assessed by examining the detrimental effects of the pollution on the material itself, and/or by considering the secondary impacts on human health.
5. *Present Worst Case Indicator*. When other indicators can not be applied, the EAL could be determined by the present worst pollutant concentrations, on the basis that regulations do not envisage allowing environmental damage to increase above existing levels.”

F.5.4 Implication and use for building society

Maps of the corrosion rates/dose-response functions can easily be transformed into maps for service life and maintenance intervals if the performance requirement is defined for the material in question. In that context the UK DoE project is a very interesting and forward looking approach by the building and environmental society in UK. It will have a huge impact in the process to come.

Such maps could then serve as a tool for maintenance planning for individual users etc. In that respect the question of transformation and validation of the dose-response functions to the microenvironment on the building surface have to be addressed. The mapping would be of great value for the standardisation work going on within CEN and ISO/TC59 WG9 “Design life of buildings” on durability of building materials and components.

F.6 EXAMPLES OF MAPPING ACCORDING TO ENVIRONMENTAL RESEARCH APPROACH

F.6.1 Degradation factors for Europe

Reference is simply given to sections E.3 and E.4 and figs E.3:2-3 and E.4:2-4.

F.6.2 GIS-based level 1 and 2 mapping for urban Oslo

By using the UN ECE dose-response functions and mapping procedures together with the available air quality data and dispersion models, the corrosion is modelled and mapped for Oslo for the years 1985 and 1993. The corrosion modelling is presented as a new AirQUIS CorrCost module (see section E.8). Integrated in the AirQUIS CorrCost module is also information from the recently completed digitised building register GAB, containing all buildings in Norway [22].

For the corrosion and cost assessment mapping of Oslo the following data and models have been used:

For *1985***,** modelling has been performed for grid size 1 km by 1 km. Calculations of SO_2 and NO_2 concentrations in the grid are based on information of emission sources (industry and heating, traffic in main and local roads) and modelling of the concentration by use of the NILU model system “KILDER” [106]. For the corrosion mapping the existing grid winter concentrations were reduced to yearly average values by multiplying the grid values with 0,5 for SO_2 and 0,3 for NO_2, respectively.

The O_3 grid values were formed by using the UN/ECE-ICP established relation between O_3 and NO_2:

$$[\mathrm{O_3}] = 60{,}5e^{-0{,}014[\mathrm{NO_2}]} \qquad (46)$$

For TOW, Rain and the $[H^+]$ concentration the yearly values measured at the Oslo UN/ECE-ICP exposure site were used.

The 1993 corrosion mapping was conducted using the 1985 values for TOW, Rain and

[H^+] and a new inventory for the SO_2 and NO_x emissions. A new model-system "EPISODES" with a grid square of 500 m × 500 m was used for calculation of the SO_2 and NO_x concentrations. The values were transformed to yearly average values in the same way as for the 1985 data.

GAB Building register. For cost assessments (level 2) information on the stock of materials and buildings at risk must be included. The Norwegian building register, GAB, with parcels, properties, buildings and addresses, was established in 1978 and completed in 1995 with digitised information on almost all four million buildings in Norway. Information from GAB is integrated into the AIRQUIS CorrCost module for Oslo.

F.6.2.1 Level 1 mapping

The *environmental degradation factors* NO_2, SO_2, O_3 and the combined factor $SO_2 \times O_3$, have been modelled and mapped for 1985 and 1993, showing the considerable decrease in the SO_2 exposure levels from 1985 to 1993 due to regulations.

Corrosion maps have been produced for weathering steel, zinc, aluminium, copper and bronze. As an example, fig F.6:1 shows the map for zinc corrosion year 1985, where the highest four years corrosion of zinc in the centre is shown to be >32 g/m^2 (= 4,5 µm).

According to the dose-response relationship, the corrosion adds up from contributions from natural, wet pollution and dry pollution corrosion. These contributions can be calculated and displayed for each grid in the GIS-system. As shown, the corrosion of 23 g/m^2 adds up from; background 14,5 (63 %); wet pollution 1,4 (6 %); and dry pollution corrosion 7,1 g/m^2 (21 %), while for corrosion of 32 g/m^2 the figures are: background 14,5 (45 %); wet 1,4 (5 %); and dry pollution 16,1 g/m^2 (50 %).

Fig F.6:2 shows the zinc corrosion mapping for year 1993 and the results from the UN/ECE ICP test site.

For zinc and the other materials, maps are produced showing areas with exceeding of acceptable corrosion rates. Fig F.6:3 shows exceeding areas for acceptable zinc corrosion at levels $1,3K_{10}$ and $1,5K_{10}$, i.e. 30 % and 50 %, respectively, above background corrosion, base year 1985.

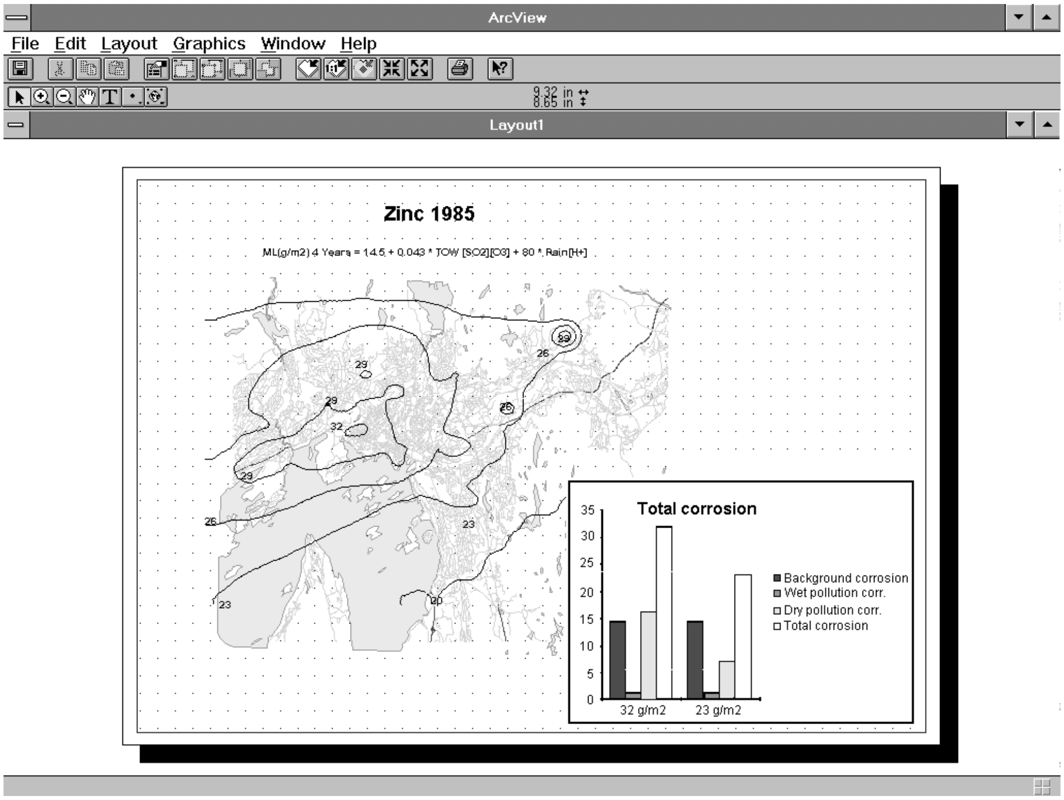
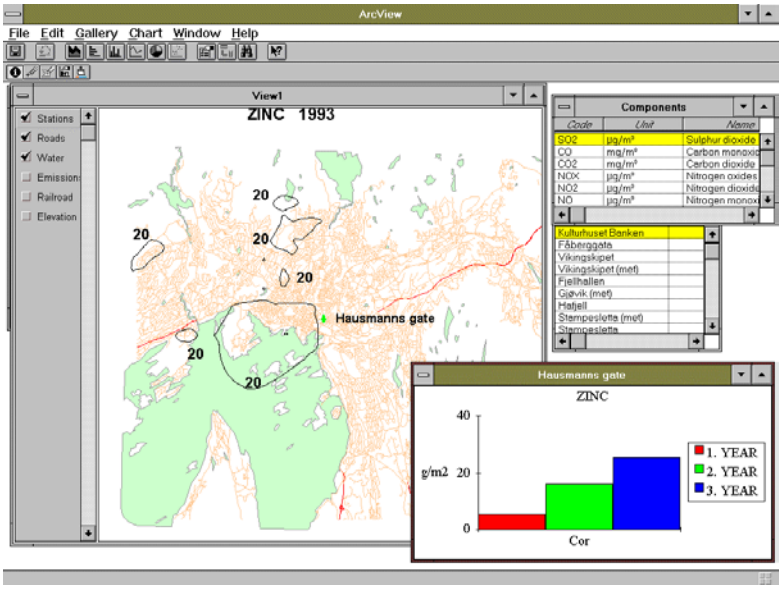


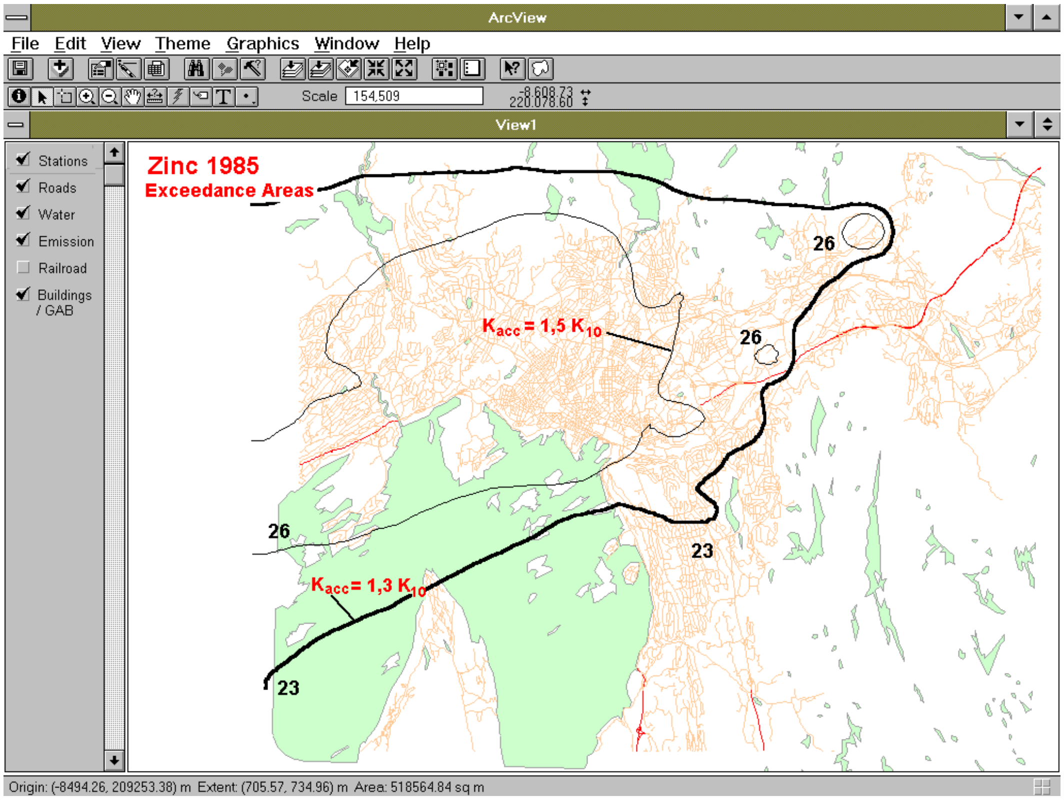
Fig F.6:1 Mapping of zinc corrosion in the Oslo area from UN ECE ICP dose-response function and available data and models for Oslo. Mapping year 1985 and grid size 1 km × 1 km, where the window shows contribution from various terms in the dose-response function at corrosion rates 23 g/m^2 and 32 g/m^2, respectively



Code	Unit	Name
SO2	µg/m³	Sulphur dioxide
CO	mg/m³	Carbon monoxi
CO2	mg/m³	Carbon dioxide
NOX	µg/m³	Nitrogen oxides
NO2	µg/m³	Nitrogen dioxid
NO	µg/m³	Nitrogen monoxi

Kulturhuset Banken
Fåberggata
Vikingskipet
Vikingskipet (met)
Fjellhallen
Gjøvik (met)
Hafjell
Stampesletta (met)
Stampesletta

Fig F.6:2 Mapping of zinc corrosion in the Oslo area from UN ECE ICP dose-response function and available data and models for Oslo displayed in the ArcView AIRQUIS CorrCost. Mapping year 1993 and grid size 0,5 km × 0,5 km. Windows show components, measuring sites and zinc corrosion results from the UN ECE ICP test site in Oslo



☑ Stations
☑ Roads
☑ Water
☑ Emission
☐ Railroad
☑ Buildings / GAB

Origin: (-8494.26, 209253.38) m Extent: (705.57, 734.96) m Area: 518564.84 sq m

Fig F.6:3 Mapping of areas of exceeding of acceptable zinc corrosion at levels 1,3·K_{10} and 1,5·K_{10}, i.e. 30 % and 50 %, respectively, above background corrosion in the Oslo area from ECE ICP dose-response function and available data and models for Oslo. Mapping year 1985 displayed in the ArcView AIRQUIS CorrCost

F.6.2.2 Level 2 mapping – assessment of corrosion costs

By use of the tools available in the ArcView AirQUIS CorrCost system a cost assessment study for Norway was carried out [99]. It allows counting different types of buildings and building materials with different economic value for each grid. Maps have been produced showing exposure environment and the stock of materials and buildings at risk by integrating an information layer on the building register GAB. Fig F.6:4 shows the location and number of different types of buildings in grid no. 20-22 (500 m × 500 m), together with the environmental characteristics of that grid [22]. By use of the materials distribution factors and the environmental dependent service life and maintenance intervals from the MOBAK study, the costs can be assessed for each material and building type within each grid. Such calculations have been performed for 1586 grids of the greater Oslo [99].

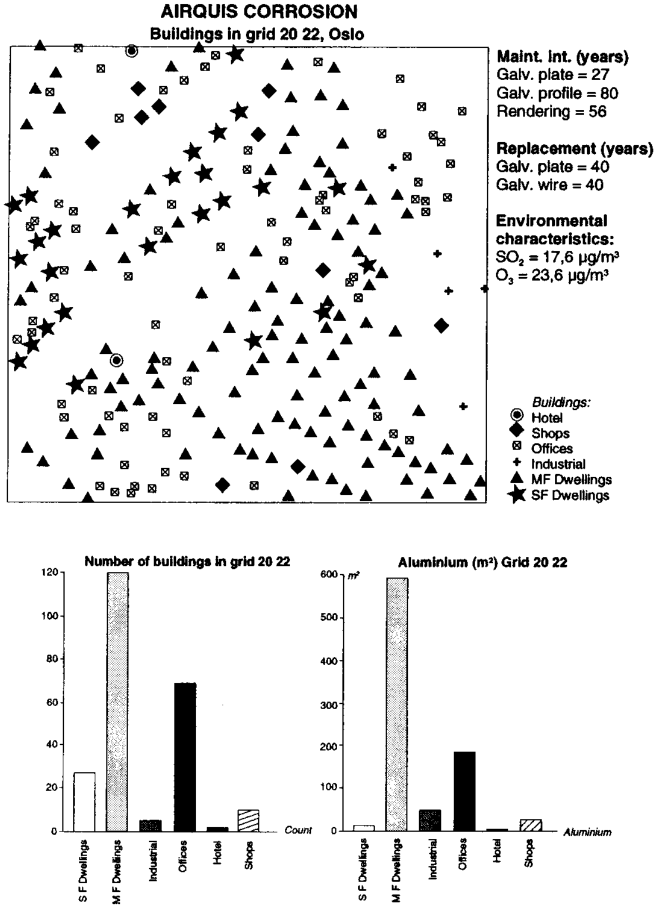


Fig F.6:4 Characteristics of buildings and exposure environment in one of 1584 grids (500 m × 500 m) in Oslo, displayed in AirQUIS CorrCost [22]

F.6.3 UK – Mapping of critical loads and levels to materials damage

From Butlin et al. [107] the following is quoted: “The United Kingdom National Materials Exposure programme was initiated in 1986 to study the effects of acid deposition on building materials [108]. The output data in the form of empirical dose-response equations (described elsewhere) have been incorporated into a geographical information system (GIS). In addition, data for the stock at risk of building materials has also been used. The dose-response relations indicate a dominance of dry deposition of sulphur dioxide in the decay process. Critical level/load maps have been determined for a number of materials. General pollution and meteorological data sets are also included in the mapping process. Maps give “exceeding squares” on a 20 km square grid basis, indicating the unprotected areas or those still at risk for a given scenario for SO_2 reduction in the context of the UN ECE protocol for sulphur. In order to derive maps of areas sensitive to pollutants in the future a model, HARM 7,2, is used for the prediction of distribution of emissions of pollutants in the UK. A series of maps has now been produced for different materials at 70 % and 80 % scenarios for the reduction of SO_2. Studies of the sensitivity of the maps of exceeding to the accuracy or variation of the components in the dose-response equations have been undertaken. Results from the mapping programme and the sensitivity analysis are presented together with discussion of the concept of critical-load of materials.”

G Micro-environmental characterisation

G.1 MONITORING SYSTEMS

G.1.1 Temperature and wetness – the WETCORR instrument

The WETCORR instrument is designed for recording of the wetness and temperature condition in the microenvironment of constructions, see fig G.1:1.

The measuring principle makes use of the electrochemical nature of the corrosion processes by measuring the current flow in an electrochemical cell as a function of the thickness of the humidity film bridging the electrodes surface. The principle was proposed by professor Thomashov as early as in 1950ies and has later been adapted and modified by other research groups [60]. By selecting various current levels, the time above certain humidity levels can be monitored.

The sensor developed so far consists of a small gold cell for measuring time of wetness, TOW^{100}, defined as the time with 100 % RH and/or rain, condensation etc., and a temperature sensor for recording the surface temperature.

To ensure that the temperature sensor follows the surface temperature, the cell backing is made of aluminium oxide with good thermal conductivity. The gold wetness sensor is made of two gold electrodes arranged in a finger pattern. A fixed voltage, usually selected in the range between 100 mV and 200 mV is applied and the current is measured. To prevent polarisation of the electrodes the direction of the voltage is changed every 30 s.

Work is going on to develop other types of sensors, such as sensors for measuring resulting moisture uptake within wood (EU-project ENV-CT95-0110 (DG 12-ESCY)). Fig G.1:1 shows the set-up of such measurements, illustrating also the measurement principle.

The instrument consists of the following modules:

System Controller (SC):

- Control unit for communication with the Sensor Adapters (SA);
- Unit for the external communication with the Sensor Adapters;
- Power source for the sensor; and sampling unit for all data.

16 Sensor Adapters - each being the control and recording unit for four sensors. The data are recorded as average values over one minute. The SA will convert all data recorded to digital signals before transferring the data to the SC. This is done to reduce the influence of stray currents and radio signals on measured data. The cables between the sensor and the SA are kept short, less than 2 m, while the cables for the digital signals between SA and SC can still be long, up to 250 m in two directions. The SA can be placed outdoors in all weather conditions.

64 Sensors - the set-up of parameters cell voltage and sampling period must be the same for all four sensors connected to one adapter, while each adapter in a system can be configured individually for temperature and humidity.

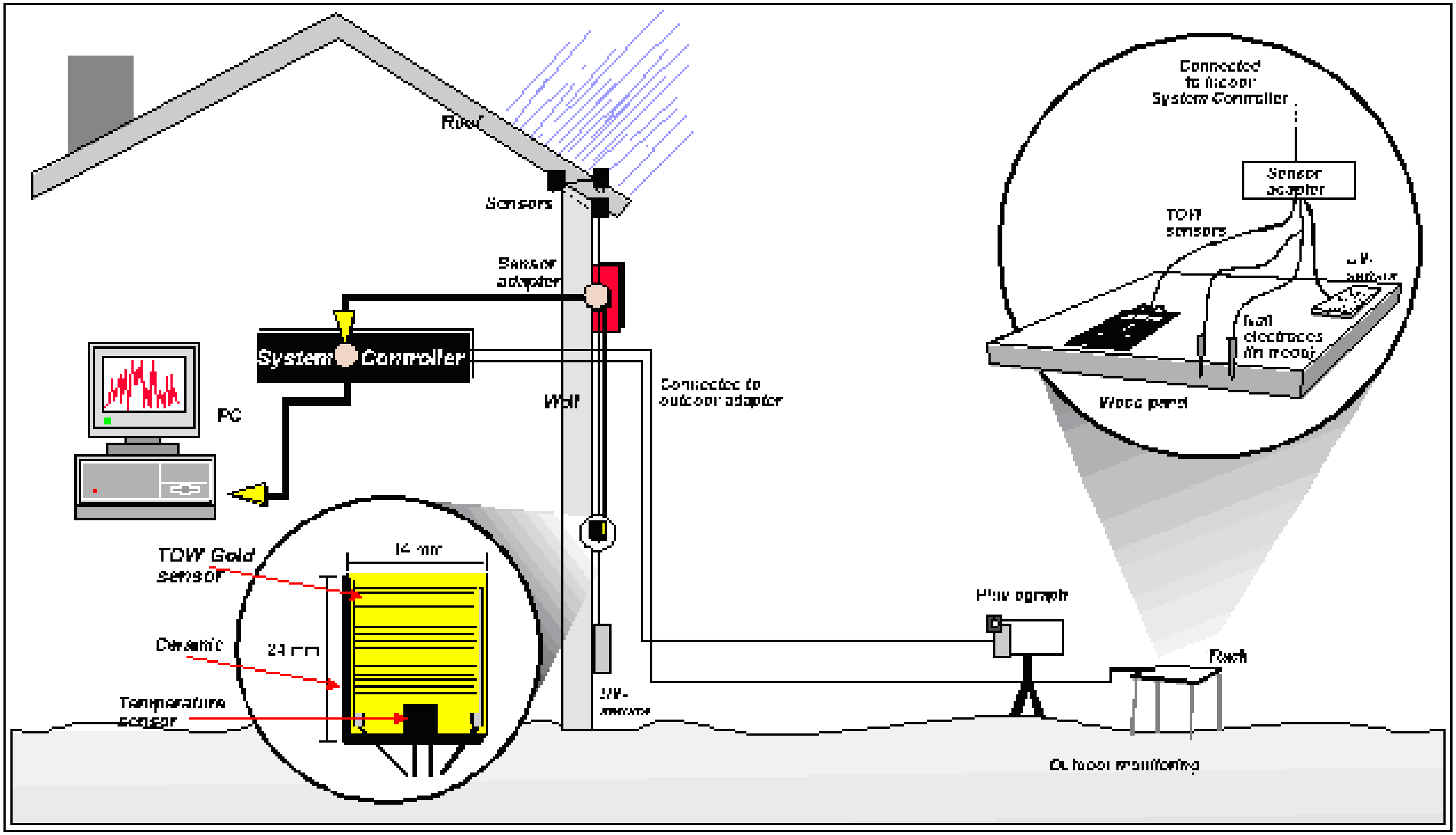


Fig G.1:1 WETCORR measurements applied on and within building materials

The amount of data has always been a problem in this type of measurements, and great emphasis is put on solving this problem through user-friendly database design and management, and data presentation tools. The instrument is therefore delivered with one Windows based software package for communication, parameter setting and recording and presentation of data. Presentations within geographical information systems (GIS) are also available.

The following data presentation forms are available:

- Currents and temperatures versus time;
- Total current output from the sensors in the period;
- Per cent of time with current above the selected TOW_{crit};
- Per cent time with the temperature above or below selected temperatures;
- Frequencies for TOW_{crit} and temperature above selected values;
- Minimum, maximum and mean values for current and temperature; and
- Minimum, maximum and mean values for TOW_{crit}.

G.1.2 UV

G.1.2.1 General

There are, in general, three main classes of detectors for measuring UV-radiation: physical, chemical and biological. With *biological methods* one measures, for instance, the amount of erythema or pigmentation of the skin in response to UV radiation. *Chemical methods* include the IG Farben dosimeter and Landsberg's glass dosimeter. The *physical methods* are the most common and include the Eppley ultraviolet pyranometer and the CSIRO ultraviolet pyranometer [109].

However, these methods demand an extensive and fairly expensive instrumentation, which is partly why a few measurements have been made of the UV environment on and around buildings. To meet the need for cheap and still reliable UV measurement techniques, photochemical dosimeters have been developed [110].

Recently NILU has developed a multi-channel UV-detecting system, basically for studies of biological effective UV-doses and total ozone abundance [111]. This UV sensor can be adopted to the WETCORR instrument and used on buildings.

G.1.2.2 Photochemical dosimeters

Results are reported from measurements in Nordic environments using photochemical dosimeters made from thin films of polyphenylene oxide (PPO) [110]. The results show, in generalised terms, a certain order of UV doses for different facades and shadowed eaves: South > East > West > North > Eaves/ south > Eaves/east. It was found, however, that the difference in values for south and north facades was relatively small, emphasising the fact that the diffuse component of the UV radiation is large. The measurements showed the diffuse component to be between 68 % and 88 % of the total radiation in the UV band.

G.1.2.3 NILU's UV-radiometer

The intensity of solar ultraviolet radiation reaching the ground is controlled by several factors, such as solar elevation, cloud cover, total ozone amount and ground reflection. A decrease in total ozone abundance is expected to lead to an increase in harmful UV radiation if all other factors are kept unchanged.

Cloudiness is among the largest sources for the variation of the daily integrated UV irradiance at the surface. Therefore, in order to detect trends in solar UV-B radiation due to trends in total ozone amount, a UV monitoring network should measure both UV-B radiation, total ozone amount and cloud transmission.

NILU has developed a multi-channel radiometer for measurement of solar UV radiation. The instrument measures the irradiance (direct plus diffuse radiation) in the UV-B region (280 nm–315 nm) and in the UV-A region (315 nm–400 nm) in five channels. The instrument contains a built-in data logger that can store up to three weeks of one-minute average readings from all channels, as well as the temperature, which is measured close to the detectors. The instrument can also be set to store a one-minute average data every 15 minutes only. In this mode the storage capacity of the data logger is one year.

Changes in the UV radiation due to variations in total ozone abundance and cloud cover can be recorded with the instrument. The UV-B channels are sensitive to variations in total ozone and variations in cloudiness. UV-A channels, however, are sensitive to variations in cloudiness but not sensitive to ozone variations.

Measurements from 29 and 30 March, 1995 at NILU illustrate the sensitivity to ozone variations, fig G.1:2. The integrated ozone amount was 417 Dobson Units (DU) on 29 March and 329 DU on 30 March, i.e. a 20 % decrease. The measured increase in UV radiation from 29 March to 30 March was 100 %, 50 % and 27 % in the 305 nm, 308 nm, and 313 nm channels, respectively. No changes were observed in the UV-A channels since the sky was clear on both days and since UV-A intensity is known to be insensitive to ozone.

By employing a radiative transfer model combined with such irradiance measurements, biologically effective UV doses, total ozone amount and cloud transmission can be determined.

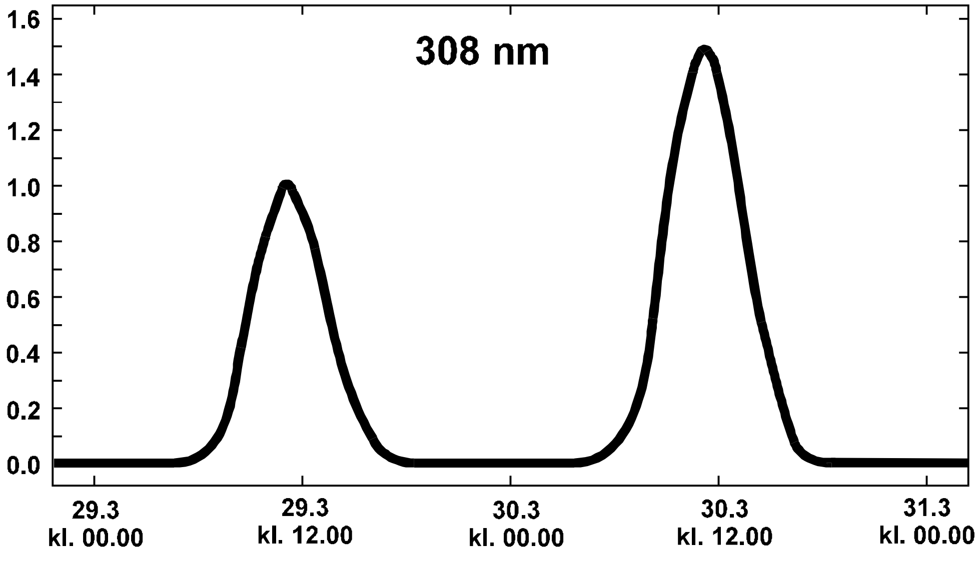


Fig G.1:2 Measurements at NILU show the sensitivity to ozone variations. The 308 nm level (UV-B) increased by 50 % from 29 March to 30 March due to a 20 % decrease in the total ozone abundance. The sky was clear on both days and no change was observed in the UV-A channels

G.2 THE PASSIVE SO_2 AND NO_2 SAMPLERS

G.2.1 Description

A simple sampler for surveillance of time integrated SO_2 and NO_2 concentration distributions has been developed. The sampler is inexpensive in use, simple to handle and has a good overall precision and accuracy. This method has been used in industrial areas, in urban areas and for studies of indoor/ outdoor exposures.

The sampler was developed by the Swedish Environmental Research Institute (IVL) and has been used in several cases by NILU. The sampler includes an impregnated filter inside a small plastic tube, fig G.2:1. To avoid turbulent diffusion inside the sampler, the inlet is covered by a thin porous membrane filter. Gases are transported and collected by molecular diffusion. The uptake rate is only dependent upon the diffusion rate of the gas. The collection rate is 31 *l*/24 h for SO_2 and 36 *l*/24 h for NO_2. Also NH_3 can be collected at a rate of 59 *l*/24 h.

For SO_2 concentration the measuring range is approximately $0{,}1–80 \cdot 10^{-9}$ (0,3–213 µg/m^3 for a sampling period of one month. The corresponding range for NO_2 is $0{,}02–40 \cdot 10^{-9}$ (0,04–80 µg/m^3). The passive samplers are assembled at NILU and made ready for use. After exposure the samplers are usually returned to NILU, where the SO_2 concentration is determined from the level of sulphate formed by means of ion chromatography. NO_2 and NH_3 concentrations are deduced by means of spectrophotometry.

Some examples of applications are shown in figs G.2:2-4.

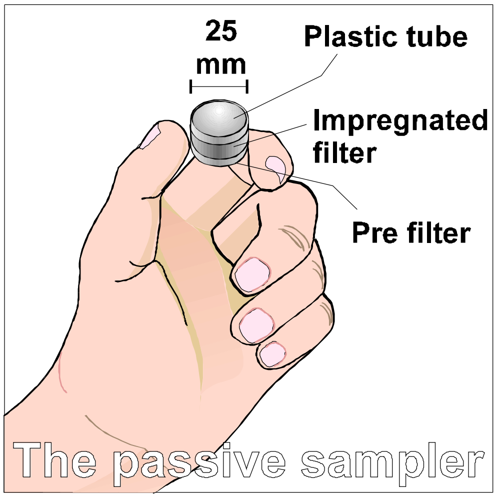


Fig G.2:1 The passive sampler

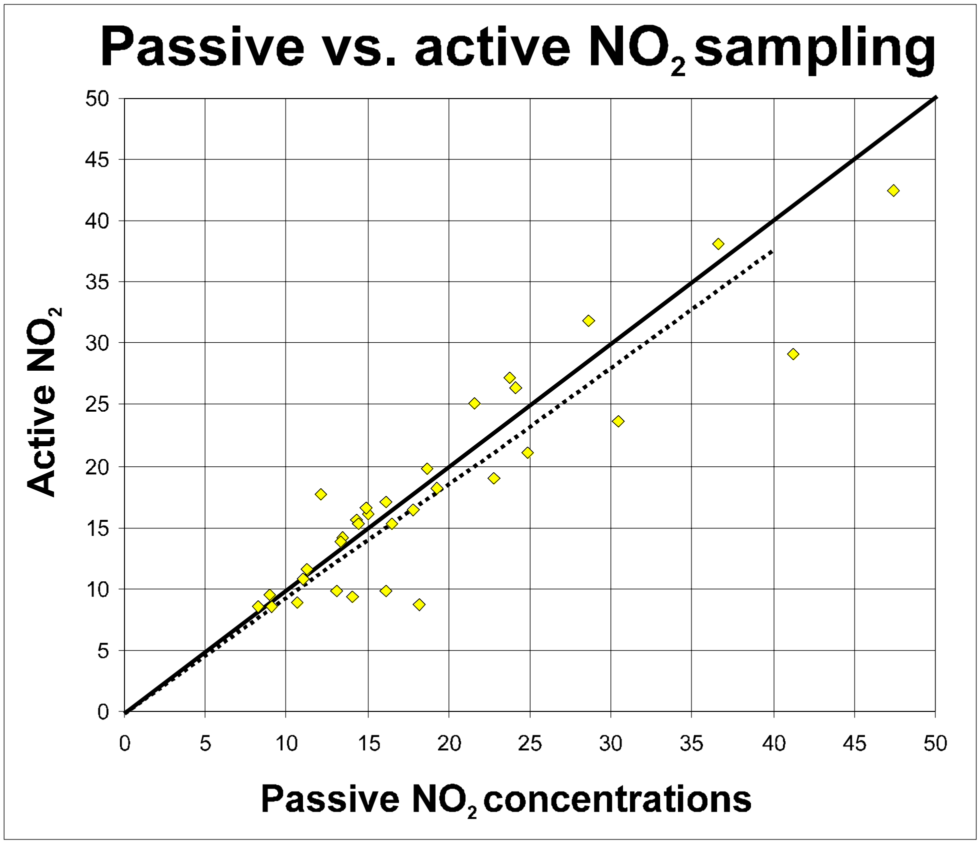


Fig G.2:2 The integrated passive sampling of SO_2 and NO_2 is well correlated to the available active sampling methods

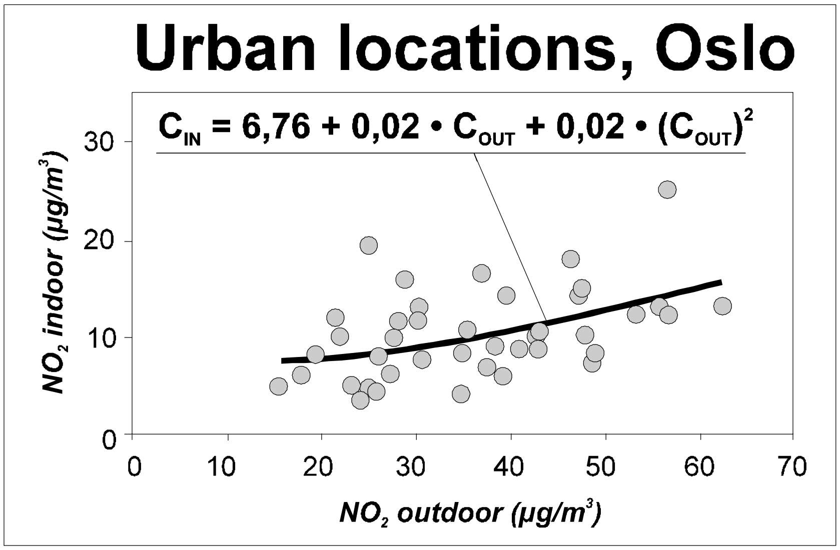


Fig G.2:3 Several investigations have been performed to study the relation between indoor and outdoor NO_2 pollution levels. In urban locations in Oslo indoor NO_2 concentrations were considerably lower and varied less than outdoor concentrations. At 40 µg/m^3 ambient NO_2 the indoor NO_2 concentration was on the average about 10 µg/m^3

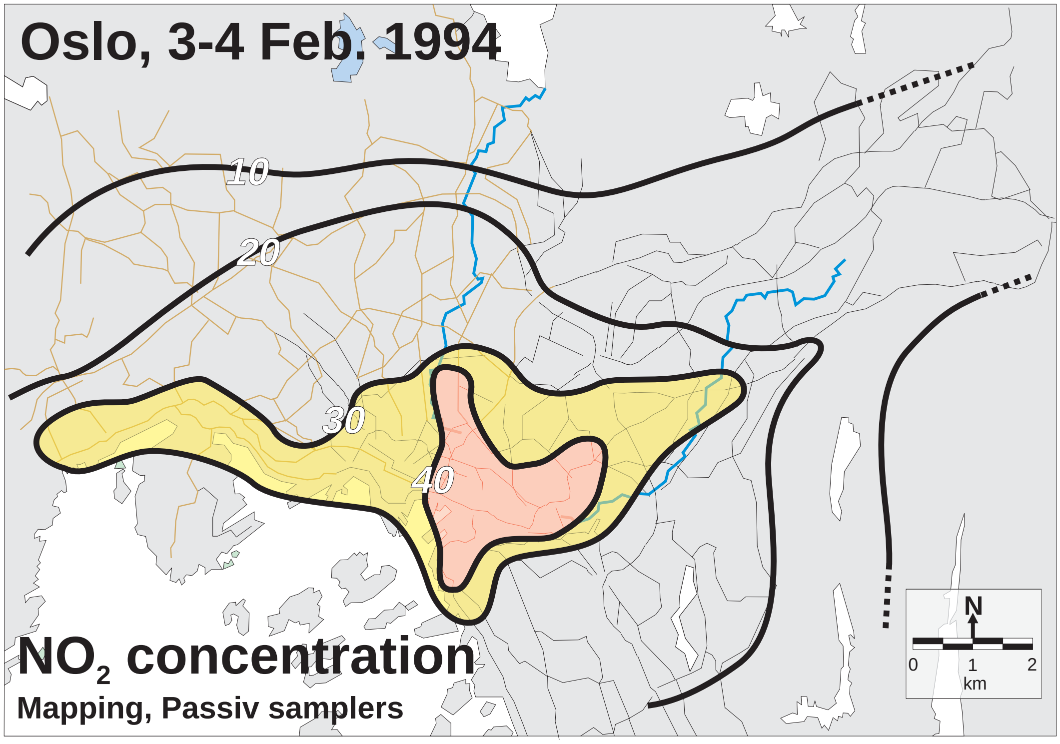


Fig G.2:4 The 24 h average NO_2 concentration distribution, measured in Oslo on 3–4 February 1994 with 20 passive samplers, shows that the highest concentrations occurred along the main road systems and in the central parts of Oslo down-town. This 24 h average distribution may change considerably from one day to the next depending on the meteorological conditions in the Oslo airshed

G.3 MODELLING

G.3.1 Road network emission and dispersion model

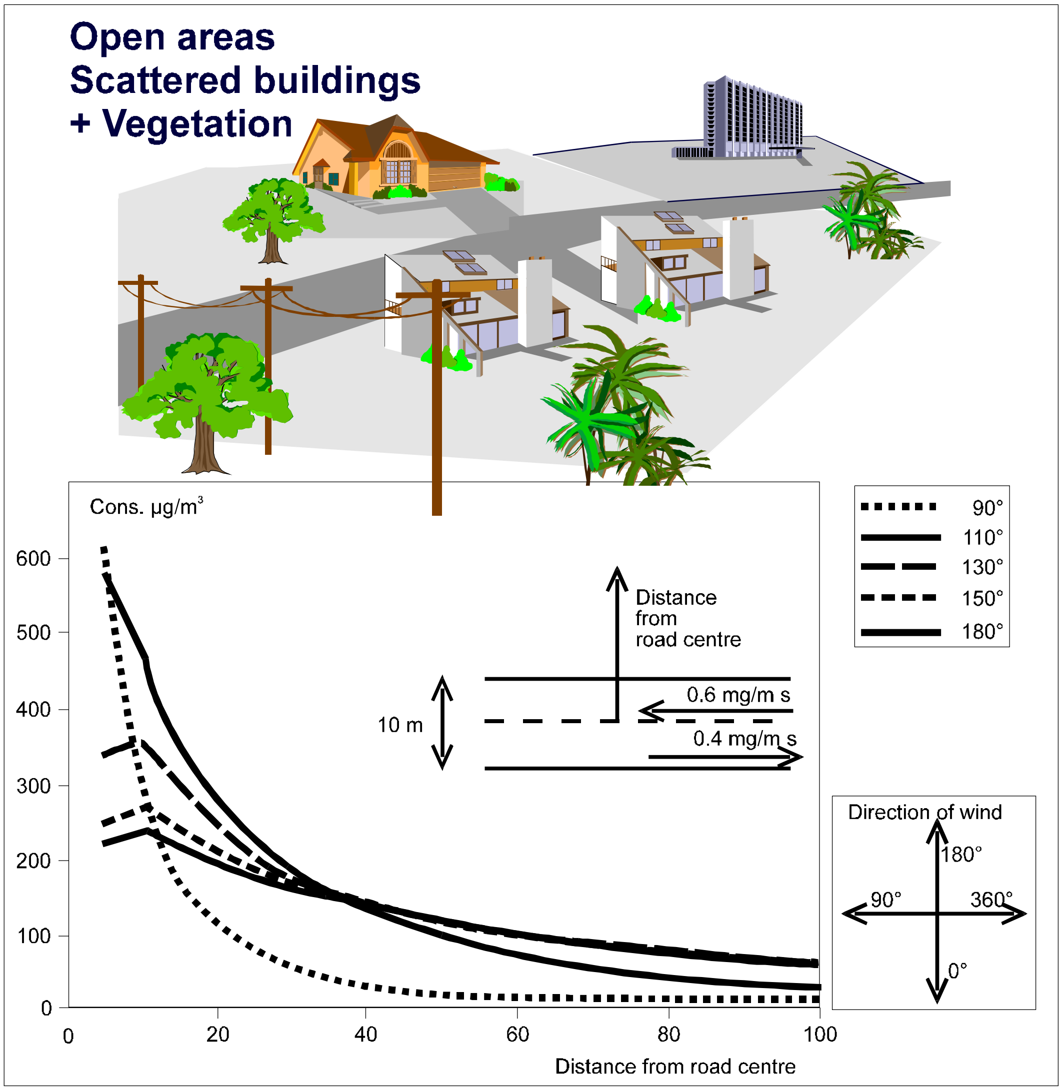
The effect of road traffic pollution on urban populations is expected to increase during the next few years. Some measures to reduce this effect are:

- Reducing the total amount of traffic, via economic instruments, town planning, legislation or physical restriction;
- Moving traffic away from residential areas, e.g. by constructing ring roads;
- Improving traffic flow/avoiding congestion, by increasing the capacity of the road network; and
- Improving vehicle technology, by applying stricter emission regulations and increasing the renewal rate of the vehicle fleet.

Traffic planners are often in need of practical tools for studying the effect of such measures on the environment. Quite a few air dispersion models exist and can be used for this purpose. NILU has developed a personal computer-based model *RoadAir*, for quantitative descriptions of air pollution along road networks. RoadAir calculates total emissions, concentrations along each road segment and the air pollution exposure of the population and *buildings* along each road, see fig G.3:1. Calculations can be carried out for road networks, defined by road and traffic data. The model was primarily developed for conditions in Scandinavia, but can easily be adapted to conditions in other parts of the world. RoadAir is incorporated into the AirQUIS system (see section E.8).

The pollution concentrations near a road with a given volume of traffic are closely related to the local dispersion conditions. The model calculates maximum concentrations that occur in periods with poor dispersion conditions (e.g. during winter).

The contribution from other pollution sources is accounted for by adding a city background concentration to each road link. Different background concentrations can be added to different parts of the calculation area.



S

Fig G.3:1 Dispersion modelling of CO concentration profile across and along roads

G.3.1.1 Input data

The geometry of each road link must be described by its length, width, steepness and whether it is characterised as a street, canyon or an open area.

The traffic data are given on a daily basis for calculation of total emissions, and on an hourly basis for concentration calculation during rush hours. If no rush hour traffic data are available, the model assigns standard values depending on the road type.

The vehicle fleet must be described by fractions of light and heavy-duty vehicles on each road. The model assigns standard distributions of technology types within classes of vehicles. The emission calculations take into account vehicular age distribution, effects of cold starts, road steepness and time of observation.

The distribution of population and buildings are described by the position of private homes relative to the road links. An average number of residents per home must be given.

Modelling of the traffic patterns that often represent the input to RoadAir, has to be performed prior to the application of RoadAir.

G.3.1.1 Output data

- Total emissions of NO_x, CO_2 and CO for the road network and emissions for each road link;
- Concentrations of NO_2 and CO at a chosen distance from the curb for each road link;
- Tables with number of road links and length of road with concentrations above chosen levels;
- Tables with number of people and *buildings* exposed to various concentration levels of NO_2 and CO for each road link, and in total along the road network; and
- Data on suspended-particle (PM_{10}) concentrations for which calculations have been included in RoadAir for areas where studded tyres are in use.

H Needs for R&D and Standards

Concerning the characterisation of environmental degradation factors the following R&D and corresponding standardisation needs are listed:

1. There is a *lack of systematic knowledge* on the right type and form of degradation factors to be characterised. In order to produce this knowledge, extensive service life research based on damage functions approach has to be carried out. It should involve models for degradation mechanisms and should be tested out in well-monitored relatively short exposures in the field and in laboratory.
2. Damage function based research and proper characterisation of degradation factors both in the field and laboratory is one absolute criterion for relevant coupling of *field and laboratory tests*.
3. Some *methods* for automatic and continuous monitoring of important degradation factors in the microenvironment on buildings exist, but testing and further development of methods are strongly needed.
4. Quite a *few functions* exist today after extensive research in the environmental research area. However, these functions have to be tested and validated in the *microenvironment* on buildings. Measuring and modelling methods for micro environmental loading and materials degradation have to be developed and extensive measurements carried out.
5. The dose-response functions produced so far often stems from the environmental research area. They are therefore limited in terms of choice of degradation indicators. In addition, they do not contain identified *limit states*, and are therefore not damage or service life functions.
6. Interdisciplinary co-operation between the building and environmental research community is a must.

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Joint CIB W080 / RILEM TC 140 – Prediction of Service Life of Building Materials and Components

Guide and Bibliography
to
Service Life and Durability Research for Buildings and Components

PART I – Service Life and Durability Research

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PART I SERVICE LIFE AND DURABILITY RESEARCH

P. Jernberg[1], C. Sjöström[2], M.A. Lacasse[3], E. Brandt[4] and T. Siemes[5]

A Definitions

A.1 SERVICE LIFE AND LONG-TERM PERFORMANCE – AN INTEGRATED APPROACH

In fig A.1:1 is shown some hypothetical performance-over-time functions for a component in a certain service environment. The functions describe how the measured values of some chosen performance characteristics are expected to vary with time. In reality, though, a performance-over-time function is not as simple as outlined below, but describes a statistical distribution of a performance characteristic. Performance characteristics are measurable, physical quantities corresponding to the critical properties identified for the component in its application. With performance-over-time functions established for the range of in-use conditions considered and agreed performance criteria, all essentials are known to make a service life prediction.

Service life analysis aims at establishing and explaining the performance-over-time functions.

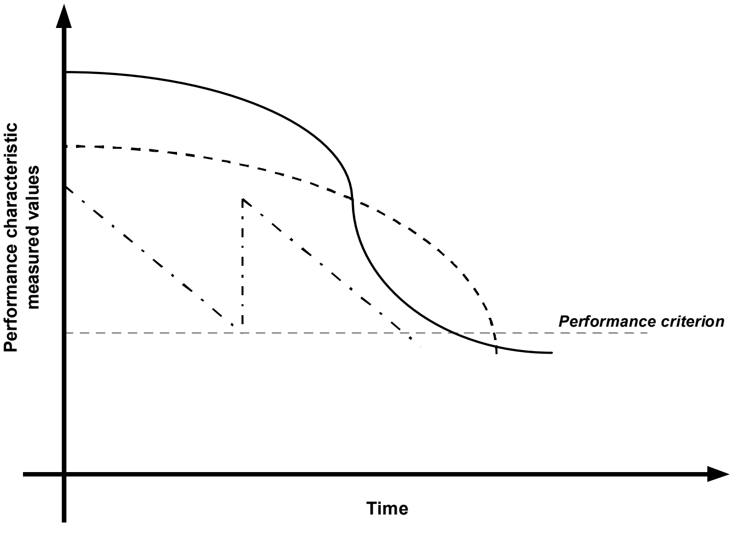

Fig A.1:1 Hypothetical performance over time functions

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Fig A.1:2 shows a service life concept. To establish the performance-over-time functions a thorough knowledge of the degradation environment is necessary.

Fig A.1:2 Relation between degradation environment, materials or products durability and performance over time

Durability, defined as the “capability of a building or a building part to perform its required function over a specified period of time under the influence of the agents anticipated in service”, is seen as the capability of a material or a product to withstand degradation in the given environment.

Service life is defined as “the period of time after installation during which all conditions of a building or a building part meet or exceed the performance requirements”.

In clause A.1.1 below is described the general concepts in a service life prediction process. The description is essentially that given in the draft standard ISO/WD 15686-2.3 “Service Life Planning – Part 2: Service Life prediction Principles”, which in its turn primarily is based on the RILEM Technical Recommendation “Systematic Methodology for Service Life Prediction of Building Materials and Components”, developed by RILEM[6] TC 71-PSL working jointly with CIB[7] W80. Finally, in A.1.2, the factor method to be used to estimate the service life for planning and design purposes is briefly described. Details can be found in ISO/CD 15686-1.4 “Service Life Planning – Part 1: General Principles”. The two standards referred to, part 1 and 2, represent the top and bottom, respectively, of “Geoff’s diamond” shown in fig A.1:3, describing the structure of a system of standards regarding service life planning of buildings.

A.1.1 Service life prediction

The systematic approach or methodology for service life prediction outlined includes the identification of needed information, the selection or development of test procedures (exposure programmes and evaluation methods), testing (exposure and evaluation), interpretation of data, and reporting of results. The methodology employs an iterative approach that permits improved predictions to be made as the base of knowledge grows.

The methodology is developed to be generic, i.e. applicable to all types of building components, and is meant to serve as a guide to all kinds of prediction processes. The essential steps in a prediction process are shown in fig A.1:4. It is of the greatest importance to account for all assumptions and judgements made. The methodology may be used in the planning of service life prediction studies regarding new and innovative components of which the knowledge of their performance is limited, or be the guiding document in the assessment of already performed investigations in order to appraise their value as knowledge bases for service life prediction and reveal where complimentary studies are necessary.

It has to be emphasised that in practice most quantities involved in a service life prediction process are statistically distributed and should be treated by appropriate, statistical methods.

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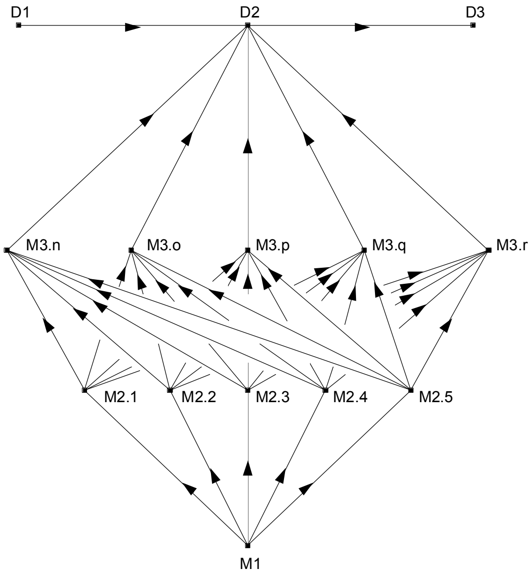
[6] The International Union of Testing and Research Laboratories for Materials and Structures

[7] International Council for Building Research, Studies and Documentation



D1 - D3	Standards for design life of buildings
M3.x	Material specific or components specific standards (product standards) for service life prediction
M2.x	Generic standards for aspects of the service life prediction of materials and components
M1	Generic standard for service life prediction of materials and components

Fig A.1:3 ”Geoff‘s Diamond” – Relationships among standards for service life prediction of materials and components and for design life of buildings

A.1.1.1 Range of the service life prediction and problem definition

First of all an analysis of the problem under study including an identification or specification of the essential data and delimitation of the range of the study should be carried out. The study may be anything from limited to a single, specific application – a specific study – to cover any possible application – a general study. Furthermore, the component to be tested should be characterised in terms of structure and chemical composition, and consideration and outlining of a critical review of the study should be carried out.

A.1.1.1.1 Definition of a specific study

A reference building context (ref. ISO 6241 and CIB Master List) and a set of performance requirements should be identified according to the information given on the specific case. The building context will account for the specific use of the component, covering the design consequences, and comprise a description of the climate at the site where a building is planned to be located. A description on the effects of occupancy (such as water vapour, heat or abrasion) and the principles on which the building is operated (e.g. high or low thermal inertia) should also be included if appropriate.

A.1.1.1.2 Definition of a general study

The types of environments (i.e. in-use conditions) where it is intended a component be used should be specified. The various types of environments may be grouped into a discrete number of classes; each class being representative for certain ranges of agent intensities. Care should be taken on the effect of various usages and positions of the component, while this can strongly govern the in-use conditions. The actual in-use condition relevant to materials degradation is the microclimate, i.e. the prevailing environmental condition in a layer adjacent to a component surface.

The set of performance requirements should be qualitatively specified in terms of critical properties. Then, in order to limit the performance range to be covered by the service life analysis, the lowest reasonable set of performance criteria for the component should be quantitatively specified. The set of performance criteria may include specifications on, for example, strength, optical transmission, acoustical insulation and aesthetic qualities.

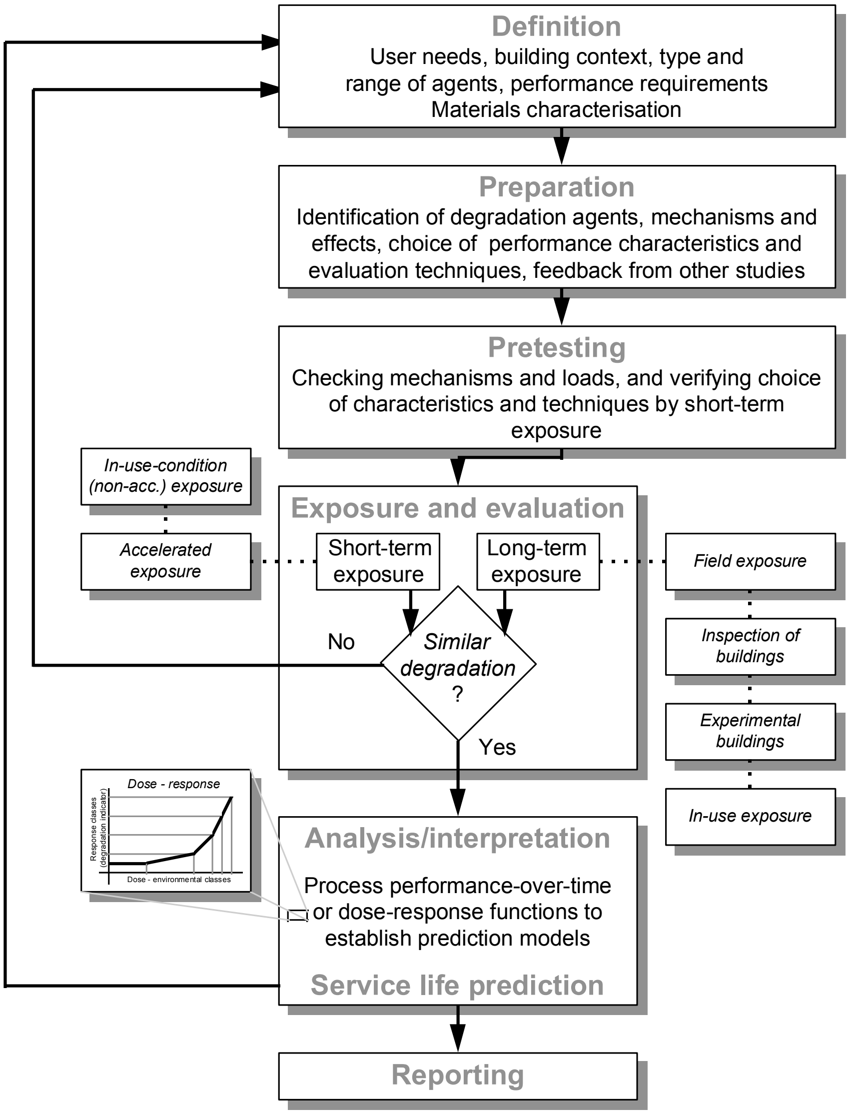


Fig A.1:4 Systematic methodology for service life prediction of building materials and components

A.1.1.1.3 Characterisation of the material or components

Characterise the material or component to be evaluated as thoroughly as possible in terms of structure, chemical composition and performance values corresponding to the performance criteria identified in section A.1.1.1 and A.1.1.2.

A.1.1.2 Preparation

The preparation step aims to identify or postulate degradation factors, possible degradation mechanisms and how degradation can be accelerated or induced within ageing exposure programmes.

A.1.1.2.1 Identification of degradation factors
The type and intensity distribution of the expected degradation agents should be identified, where one or several reference environments are considered, which and the number depending on the range of the study. A list of relevant degradation agents is presented in Table A.1:1, condensed from ISO 6241:1984. The agents are classified according to their *nature*. In general, external to the building the *origin* of the agents is either the atmosphere or the ground, whereas internal to the building the origin is related to occupancy or design and installations. However, although not stated in ISO 6241:1984, an agent acting externally while originating as a design consequence can also occur. Furthermore, the influence of agents originating from the atmosphere to internal degradation should not be disregarded.

Table A.1:1 Degradation factors affecting the service life of building materials and components

Nature	***Class***
Mechanical agents	*Gravitation* *Forces and imposed or restrained deformations* *Kinetic energy* *Vibrations and noises*
Electromagnetic agents	*Radiation* *Electricity* *Magnetism*
Thermal agents	*Extreme levels or fast alterations of temperature*
Chemical agents	*Water and solvents* *Oxidising agents* *Reducing agents* *Acids* *Bases* *Salts* *Chemically neutral*
Biological agents	*Vegetable and microbial* *Animal*

The most relevant agents within ageing exposure programmes on building components usually originate from the atmosphere, that is:

- Freeze-thaw and wind, (mechanical);

- Solar radiation, (electromagnetic);
- Temperature – elevated, depressed and pulsed, (thermal);
- Precipitation – solid, liquid, vapour, (chemical);
- Normal air constituents, (chemical); and
- Air contaminants – gases, aerosols, particulates, (chemical).

Some quantitative information on agents originating from the atmosphere is available from published weather and climatological data and from data published on air pollution. Classification of the environment in terms of its corrosivity on a number of standard materials, mainly metals, is also carried out for wide areas. Dose-response functions (see A.1.1.5.2), relating the dose of agents, mainly of chemical nature, to the degradation of materials, are available for certain classes of materials.

A.1.1.2.1.1 Combined agents and combination of agents

Some agents are combined by more than one co-existing factor, e.g. an imposed force such as freeze-thaw stress is due to cycling temperature and to the presence of water. At the same time, water solely is a chemical agent. Temperature itself is the thermal agent, while for many chemical reactions the temperature is decisive for the reaction rate. Other agents of the same or different nature can give rise to significant synergistic effects, e.g. sulphur dioxide together with nitric oxides, and UV radiation together with oxygen (photo oxidation).

A.1.1.2.1.2 Chemical and physical incompatibility between dissimilar components

Incompatibility between components is considered as an agent related to the design. By considering each component separately, incompatibility to another component can be treated analogous to, for instance, an atmospheric agent, both in general terms being parts of the environment of the component, in accordance with ISO 6241:1984, 6.4.1. Incompatibility includes, for example, corrosion caused by contact between dissimilar metals or stress caused by different thermal expansion coefficients of rigidly connected dissimilar components.

A.1.1.2.1.3 Upper limits to certain agents

It can be difficult to quantify the in-service intensity of biological agents and agents originating from the occupancy, but upper limits within the normal range can usually be established by engineering judgement. When selecting or designing ageing exposure programmes, consider each of these certain degradation agents that can affect the performance of a building component.

A.1.1.2.2 Identification of possible degradation mechanisms and effects

All reasonable possible mechanisms by which the identified degradation agents are known or believed to induce changes in the properties of the component should be identified. The mechanisms may be identified at various levels. If the chemistry of the component is well documented, it can be possible to identify mechanisms based upon specific chemical reactions, such as hydrolysis and photo-oxidation.

If little is known about the chemical reactions of the component, mechanisms may be defined in more general terms as, for example, thermal decomposition, volatilisation of constituents, constituent diffusion, corrosion, and shrinking/swelling.

Limitations of the knowledge available will always exist. However, it is important to identify as many conceivable degradation mechanisms as possible. This reduces the possibility for error and improves the basis for establishing that those mechanisms induced in an exposure programme, in particular accelerated short-term exposure, are representative of those that occur in service.

Also the possible effects of degradation on the performance characteristics of the material or component should be identified.

A.1.1.2.3 Choice of performance characteristics and performance evaluation techniques
The critical properties, corresponding to the set of performance requirements identified or specified, should be interpreted in terms of any of the performance characteristics found to be afflicted with degradation. For each of the performance characteristics selected, appropriate measurement and/or inspection techniques[8] should be chosen; to realise a service life prediction, it is a prerequisite that quantitative data are obtained. Finally the initial values of performance characteristics selected should be determined before the ageing exposure programme starts.

A.1.1.2.4 Feedback from other studies
Information from other studies, concluded or on going, should always be sought. Useful information can run from general knowledge of similar components, measurement techniques and exposure programme design to detailed data on performance-over-time functions of cases closely related to the case to be studied. In the latter case, and in favourable circumstances, this can reduce the volume of testing test and/or considerably shorten the test period.

A.1.1.2.5 Postulations regarding ageing tests
With the information obtained above, postulations can be made regarding specific procedures for inducing the identified mechanisms of degradation using the degradation agents identified. When accelerated short-term exposure is used it is necessary to ensure that extreme intensity levels of degradation agents do not result in degradation mechanisms that would not be experienced in service.

The postulations that are made in this step lay the groundwork for selecting or designing preliminary exposure programmes.

A.1.1.3 Pre-testing

A pre-test shall provide for the selected performance characteristics to be evaluated before and after exposure to the degradation agents to which the component will be exposed in service, or at least to all degradation agents suspected to be of any significance. This will, when properly performed

- Establish the primary degradation agents and their order of importance,
- Support or rule out the previously identified mechanisms by which property changes occur,

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[8] Numerous of measurement and inspection techniques exist of various degree of sophistication. Standardised techniques may always be recommended just because they are standardised. However, apart from generally being the simplest ones (and thus too simple in certain cases), many of the standardised inspection techniques suffer from the fact that they are relying upon subjective judgements of the practitioners. This makes comparisons between studies from different practitioners less reliable, which in fact counteracts the purpose of standardisation. Accordingly, there is a need for further development in this field and a standardisation of existing, more sophisticated techniques.

- Establish the agent intensity levels needed to induce property changes and demonstrate how rapid changes in the selected performance characteristics can be induced by exposure to extreme intensities,
- Contribute to a better understanding of the nature of the primary degradation agents leading to property changes and indicate additional or substitute property changes that are likely to be relevant and useful as performance characteristics, and
- Verify the adaptability of the measurement and inspection techniques chosen for the performance evaluation.

A.1.1.3.1 Intensities of degradation agents used in pre-tests

Intensities should be established at a level relative to the quantitative in-service distributions or ranges identified. As an example, weather and climatological data for the most extreme climates in which the component will be used can form the basis for the choice of intensities of these agents in the pre-tests.

A.1.1.3.2 Biological and incompatibility factors

These may not be important unless combined with the extreme values of other agents. For example, fungi and bacteria are most active in warm, moist locations; chemical incompatibility may only be important as long as liquid water is present between joined components; physical incompatibility may not be important unless there are large temperature changes. Accordingly, the effects of biological and incompatibility agents can usually be estimated along with pre-tests intended to determine the effect of the relevant main agents.

A.1.1.3.3 Agents related to occupancy

Although agents related to occupancy are not often included in ageing exposure programmes, they can affect the service life of building components and should be evaluated if deemed critical. However, abuse is usually considered beyond the scope of test methods. Installation and maintenance practices are normally assumed to be provided as recommended by the manufacturer.

A.1.1.4 Ageing exposure programmes

The full exposure programme must be carefully designed to cope with the range and aim of the study, considering the information and data obtained by the procedures described above.

A.1.1.4.1 Design and performance of exposure programmes

Component properties as well as environmental characteristics are stochastic variables, i.e. they are represented by statistical distributions. Therefore, irrespective of the type, the exposure programme should, if feasible, be design to comprise a multiplicity of specimens or test objects, enabling a statistical treatment of test data. This recommendation, though, may be difficult to follow in some cases when dealing with experimental building and in-use-exposure or when tests are very costly. In such cases distribution widths or ranges should, if possible, be estimated from other sources of information. For all kind of exposure programmes the conditions have to be recorded continuously or at sufficiently short intervals. Depending partially on the type of exposure program, the reasons for this are to:

- Enable establishment of performance-over-time or dose-response functions

- Provide relationship between different exposure periods and sites (for exposure programmes with uncontrolled conditions)
- Check that the actual environmental conditions is representative for the environmental type (for exposure programmes with uncontrolled conditions);
- Verify that the intended degradation agent intensities are achieved (for exposure programmes with controlled conditions).

The sources of recording may vary from official environmental databases to detailed measurements of degradation agent intensities at or in the vicinity of the test samples. In the field of environmental characterisation a strong development occurs towards, e.g. standardised measurement techniques and improved dispersion models, hosted in GIS environments, facilitating mapping of environmental data from meso/local levels to micro levels.

A.1.1.4.1.1 Long term ageing exposures under in-use conditions

The exposure programme may consist of an actual in-service exposure of a complete system in which feedback information on the performance of the components included is obtained over time, or it may involve exposure of selected components. It is essential to design an exposure programme so that all agents of importance are considered. Even at a specific study, preferably the exposure should take place in more than one type of service environment.

The different ways of generating data from long term ageing under in-use conditions are described in four categories.

Field exposure

Standardised ways of performing atmospheric field exposures have been in operation for some time. An example is the ASTM method G7-89 “Standard Practice for Atmospheric Environmental Exposure Testing of Non-metallic Materials”. The standard ISO 9226 “Determination of corrosion rate of standard specimens for the evaluation of corrosivity” is, it is true, a standard for classification of the environment from exposure of standard samples, but the exposure practice advised may be employed for any specimens.

National and international atmospheric field exposure programmes on a vast of materials have been performed, of which many still are ongoing. Evaluations of these have resulted in establishment of performance-over-time and dose-response functions, which should be taken into account as an important source of information at service life prediction. The most extensive exposure programme is the UN ECE ICP programme, aiming at classifying the effect of airborne acidifying pollutants on corrosion of materials. The programme involves field exposure at 39 sites in 12 European countries and in the United States as well as Canada.

It is essential to note that:

- The results of a field exposure relate to the specific exposure site and that the transformation of data to relate to another geographic location requires knowledge of performance-over-time or dose-response and environmental characteristics,
- One has to be careful when drawing conclusions from one exposure period to another, especially if the time of exposure is short, and
- Exposing component samples to the environment may be regarded as an accelerated exposure – for instance at exposure racks with inclination 45° and directed towards the sun – with the degree of acceleration varying with the type of component under exposure.

Inspection of buildings

The service life of building components may be evaluated through inspection of buildings. As many objects as feasible should be included in the study by means of statistical sampling methods. By data processing, service life distributions for the components studied can be found.

Evaluating the service life of building components through inspection of buildings involves several obstacles that have to be considered:

- It may be difficult to obtain data of the history of the inspected components, i.e. data on the original performance values, information on installation and performed maintenance etc.; and
- Normally the service environment cannot be controlled and may be difficult to monitor or measure and describe.

Feedback from practice of durability data by inspection of buildings has the advantage, when the investigations are properly designed, to give direct correlation between the state of components, exposure environment and the building use.

Experimental buildings

Durability evaluations of building components may be carried out by exposing the component to load and environmental effects when incorporated within a test facility such as one dedicated to experimental evaluations. If the exposure programme and the test building are properly designed according to the aim of the study, this approach for establishing data from long term ageing under conditions close to an in-use situation has several advantages. The most important asset is that the building component may be tested in full scale (for example is it possible to measure the effective sound insulation performance).

The difficulties of generalising the results as were pointed out for field exposure, may also apply to investigations involving the use of experimental buildings.

In-use exposure

By in-use exposure is meant an intentional use of a component in a full-scale building or structure under normal use, in order to evaluate the service life of the component. The aim of the approach is to create an experimental situation as well observed as possible, where the component tested is under the influence of the full range of degradation agents of the in-service situation. There are several similarities between the use of experimental buildings and the in-use exposure approach.

The limitations of the in-use exposure approach are basically the same as for experimental buildings. The in-service environment affecting the component tested is normally more difficult or not at all possible to control, and may also be difficult to measure and describe. In-use exposure is a necessary experimental approach in many cases when building components are to be subjected to degradation agents directly related to user action and behaviour.

A.1.1.4.1.2 Short term exposures

Accelerated short term exposures

The goal of an accelerated short-term exposure is to provide a relatively rapid mean of measuring the rate of property changes typical of those that occur in long term ageing exposures under in-use conditions. Short-term exposures can be structured into three major groups:

1. **Reference components / Comparative exposures.** In the simplest form such exposure programmes can be used to rank and classify new components in reference to existing. However, when the degradation mechanisms are understood, by designing an exposure programme carefully and combining results from detailed long term exposures on the same type of or similar (reference) components, a service life prediction is achievable.
2. **Simulated and accelerated environmental exposures.** Exposure programmes attempted to simulate and accelerate the entire service environment, or at least the intensity of the agents to which the component is expected to be susceptible. The level of acceleration is normally estimated from relative intensities of the degradation agents and some degradation model.
3. **Acceptance exposures.** Usually accelerated exposures designed with a pass or fall performance criterion, relying on experience of the performance of similar components in specific environments. This type can at the best give a crude quantitative indication of the service life and is discouraged.

Accelerated short-term exposures should normally be designed from information obtained in pre-tests and/or from long-term exposure of the same or similar components. In general the intensity of agents in these exposure programmes should be less than in pre-tests to reduce the likelihood of causing degradation by a mechanisms that are not encountered in-service. The properties measured before and after ageing should be those that have been identified as most useful or most important for indicating degradation. The possibility of synergistic effects between degradation agents should also be considered.

It is essential to confirm that degradation mechanisms and the relative reaction rates induced by accelerated short-term exposures are the same or at least similar as those observed in service. Thus, accelerated short-term exposures are by advantage combined with in-use conditions and short-term exposures (see below). By measuring degradation at early stages at both exposure situations, a quick comparison in this respect can be made.

Accelerated short-term exposure, in combination with modelling, is particularly well suited to assess the effects of mechanical and thermal agents, static as well as cyclic.

Short term in-use exposures

Short-term exposures are usually, but not always, based upon accelerated ageing. In cases when property changes leading to degradation can be detected at early stages (typically by means of high-sensitive surface analysis instruments), an exposure set-up employing in-use conditions, i.e. similar designs as for long term exposures, can be utilised.

A.1.1.4.2 Performance evaluation

During exposure the performance should be evaluated in terms of the selected performance characteristics by means of the measurement and inspection techniques chosen. The evaluation should take place at sufficiently narrow intervals, in accordance with the range and aim of the study. Particularly, in order to verify that the degradation mechanisms do not change with time of exposure, the exposure programme should permit the most important degradation mechanisms to be identified in a relatively short period of time.

For certain agents, empirical and semi-empirical models have been developed which allows for simulation of exposure. In such cases the evaluation may be based mainly on computing with moderate exposure and measurements efforts only. This is particularly the case for agents of mechanical and thermal nature. However, models on the effects of chemical agents exist and develop strongly and should be utilised when possible.

A.1.1.4.3 Comparison of types of degradation

The types and range of degradation obtained at short-term exposures should be compared with the degradation at in-use conditions. If mechanisms are induced, not being representative of those obtained under the in-use conditions, alter the ageing exposure programmes after reassessing the information obtained under Definition, Preparation and Pre-testing.

A.1.1.5 Interpretation and discussion

The interpretation of data obtained from the performance evaluation aims to find the predicted service life distribution sought for (or to compare the relative performance of components).

An essential part of data interpretation is always a judgement by experts. Despite efforts to quantify results and to base decisions upon scientific information, judgement is ever present and should be accounted for.

A.1.1.5.1 Performance-over-time functions

In general a performance-over-time function, possibly expressed in terms of a degradation indicator, is a complicated, non-linear, multivariate function of time as well as of agent intensities or combinations of such agent intensities. Moreover, such a function is not single-valued but describes a statistical distribution of a performance characteristic.

In the variable range of interest, though, it is normally possible to approximate the function by a considerably simplified expression, often linear in some or all variables, see below. A performance-over-time function should be evaluated, for instance by means of regression analysis, from quantitative data, possibly by the aid of physical-mathematical models, established semi-empirically or from first principles.

A.1.1.5.2 Dose-response functions

For many classes of components it has been possible, based on field exposures, to establish approximate, descriptive dose-response functions, each valid for an entire class of component. These functions, which relate the doses of the most important degradation agents to one or a few degradation indicators of the components, are by necessity, simplified relationships, frequently of linear, sometimes of non-linear character. The degradation indicator is characterised in a summarised and simplified way, often as weight-loss or weight-gain[9].

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[9] Emerging from the medical field, the use of dose-response functions has been adopted and developed within the field of atmospheric corrosion of components and the environmental research area.

Essentially, dose-response functions describe the same courses of events, as do performance-over-time functions, however simpler in form and character and with the time and intensity variables merged into variables of doses, at least formally [10)].

A.1.1.5.3 Service life prediction

Degradation models can be establish and the predicted service life distribution can be found by processing the results of the evaluations carried out at various ageing exposure programmes – long term exposures, short term exposures or combinations thereof. By synthesising, modelling and/or interpolating/extrapolating the performance-over-time or dose-response functions established from these evaluations, a performance-over-time or dose-response function for a hypothetical case sought for can be found.

A predicted service life distribution can be resolved from established performance-over-time or dose-response functions by inserting the quantitative set of performance criteria to be fulfilled by the component tested, expressed in terms of the performance characteristics or degradation indicators employed in the exposure programmes. The predicted service life distribution is determined by the performance-over-time function of the critical property found to be the dimensioning critical property. (When dealing with proper dose-response functions the dose variable(s) has to be separated into time and intensity variables in order to obtain the time dimension and, finally, the service life in an explicit form.)

Already established relationships between performance-over-time functions obtained from long term and short term can be used to predict service lives of related components by means of additional short-term exposures exclusively. If more than one service environment is employed in an exposure programme, the service life of the tested component, serving in another environment than one of the exposure environments, may be predicted by means of interpolation or extrapolation techniques. Extra caution shall be taken at extrapolation.

At a specific study the analysis can be limited to the predicted service life distribution at the specified conditions and its sensitivity to moderate variations in the service environment and the set of performance criteria, typically expressed as partial derivatives.

An alternative to actually predicting service life is to compare the relative performance of several components that have been tested in a similar manner. Such comparisons are often made to rank components.

A.1.1.5.4 Iterative process

To make sure that the predictions are based on the best knowledge available, it is recommended that an iterative research or decision-making process be employed for the service life prediction. This is illustrated by the outermost loop in fig A.1:4. In addition, while not illustrated, sub-loops between steps within a cycle may be necessary.

A.1.1.6 Reporting

The results of the study should be fairly and accurately reported to the intended audience. The findings of all analyses, data, methods, assumptions and limitations should be transparent and presented in a sufficient detail to allow the reader to assess the quality of the information.

[10)] Functions referred to as dose-response functions where time and intensities (or related quantities) are explicit variables can be found in the literature. Although this is not in accordance with a strict definition, it has become an established designation, especially within the environmental research area.

The report should also allow the results and interpretation to be used in a manner consistent with the goals of the study.

All results, where relevant, should be reported together with measured, calculated or estimated statistical uncertainties. Uncertainties can, for example, be expressed in terms of distributions, standard deviations or levels of confidence. Short-term ageing exposures typically involve a significant degree of uncertainty and the results have to be considered with care.

A.1.2 The factor method for estimating service life

The method described here 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 and a series of modifying factors that relate to the specific conditions of the case. For example, if the reference service life of a window is 20 years, a modifying factor of 0,8 might be used to estimate the window's service life in a particular position where a severer environmental condition prevails than what was presumed at the establishment of the reference service life. The estimated service life would then be 20×0.8 = 16 years. But if a particularly rigorous inspection and maintenance regime was to be applied to ensure that minor defects did not develop into more serious problems, then a further modifying factor of 1,4 might be applied. The estimated service life would then be 16×1.4 = 22.4 years. Note that factors of less than unity reduce the estimated service life and factors of more than unity increase it. The factor 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 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:

Estimated service life (ESL) = Reference service life (RSL)×A×B×C×D×E×F

A.1.2.1 Use of the factor method

The factor method is a way of bringing together the consideration of each of the variables that is likely to affect service life. It can be used to make a systematic assessment even when test data is lacking or does not fully match the anticipated conditions of use. Thus its use can also bring together the experience of designers, observations, intentions of managers and manufacturers' assurances. Not all components will need forecasts based on a factored estimate, and the project team and the building owner should agree which components are to be assessed on the basis of their criticality to use and cost of the building.

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. When data, especially the reference service life, largely are based on results from studies carried out according the service life prediction methodology described above, of course the reliability improves. In any case, the information taken into account should be recorded, so that it is clear whether the estimate is particularly robust or not. Certain parties involved in building projects may be concerned about liability for estimating future performance. For the avoidance of doubt, the recommendations given here are not intended to implement contractual liabilities and the expectation is that "best efforts" will be applied, but that estimates cannot be expected to always be accurate.

It may be desirable to consider the consequences of failure when estimating service lives using the factor method. The method may be used as a guide both to those components that should be included in estimates (it is not necessary to estimate every component's service life) and in deciding those where failure would be most critical. The method itself does not indicate the seriousness of failures, but interpretation of results may suggest components that are too "risky" to be used without either enhancing the specification or providing for regular condition monitoring. For these it may be necessary to be extra cautious, either by critically reviewing (and possibly changing) the design life (the "target") or typically by being particularly rigorous in assessing the value of factors. The factor method can be applied to both components and assemblies. When applied to assemblies it is necessary to consider the interfaces (for instance joints) between components, as well as the components themselves. For example, different external environment and maintenance factors may apply to a whole assembly that relies on sealants to weatherproof the joints between factory-made cladding units, than those which would apply to the each of the individual cladding units.

A.1.2.2 The reference service life

The starting point of the factor method is the reference service life. It is a documented period in years that the component or assembly can be expected to last in a reference case under certain service conditions. It may be based on:

- Data provided by means of the service life prediction methodology or another test carried out by a manufacturer or any other interested party (this may be a single figure or, preferably, a distribution of typical performance);
- Previous experience or observation of similar construction or materials or in similar conditions;
- Building codes may give a typical service life for components;
- Some books are available which include typical service lives; and
- Assessments of durability in the certificates of Boards of Agrement in the EC states.

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 minimising the inaccuracy introduced by the factors.

When the reference service life is provided by a manufacturer or other organisation it may be necessary to establish what conditions it is based on – for example if figures for a steel bath are based on domestic use they may not apply under more intensive use in an institution. Those who provide guidance on the reference service life of a category or class of component or of a type of construction should say in what conditions the figures apply. It is important to know whether the reference service life is pessimistic (e.g., based on time to first failure) or moderate (e.g., based on mean time to failure).

A.1.2.2 The modifying factors

A.1.2.2.1 Choice of values

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, e.g., the service life of carpet in an entrance hall may have been found to be 25% less than the carpet in general circulation areas. This observation would justify a reduction of the indoor environment factor (D) of 75%.

A.1.2.2.2 Assessing the overall contribution of individual factors

A combination of small modifying factors can have a significant effect overall. It is therefore essential to keep the overall picture in view when applying a series of these factors. 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. This is especially important when degradation is affected by a combination of factors, for instance poor workmanship and exposure to driving rain – separately these factors may have little impact on the service life but taken together they are likely to lead to failures. Where two or more agents acting together have a negative or enhancing effect disproportional to either on their own, it is necessary to consider whether both will tend to lead to the same type of failure or protection against failure, for example embrittlement of plastic rainwater drainage pipes due to UV degradation and likelihood of impacts during maintenance access may both lead to breakage while both sheltering overhanging elements and painted protective coatings reduce this risk.

It is important not to “double-count” negative or enhancing factors under more than one heading. This can give an unduly cautious estimated service life. For example, if it is considered unlikely that site mixes of concrete will match the specification this should be allowed for under either the materials factor or the workmanship factor, but not both.

A.1.2.2.3 Data from which factors can be derived

Modifying factors will often be based on known actions of the environment on specific materials, for instance increased corrosion in salt atmospheres, or on known effects of poor workmanship and maintenance. Manufacturers should give details of things that are likely to reduce or extend service life. Wherever possible the information should be given in a way that allows a figure to be selected as a modifying factor, for example the service life of a smoke alarm can be increased from five to seven years by regular vacuum cleaning (which can be expressed as a modifying factor (F) of 1,4 when a strict maintenance routine is operated). Each factor represents the deviation from the assumed conditions when the reference service life was established.

A.1.2.2.4 Description of the factors

Factor A: Quality of components

This factor represents the component as supplied to site. It is a measure of the quality of the design of the component itself (e.g. the specific softwood species together with timber preservation, jointing and applied coatings for a softwood window where the reference value was just for any softwood window).

Factor B: Design level

This factor reflects the component’s installation in the building and is based on whether the design of the building provides the installed component(s) above average shelter and protection from agents or confronts them (e.g. an overhanging roof can provide extra protection to the wall below, a reflective coating can prevent cladding exposed to sunlight reaching a critically high temperature).

Factor C: Work execution level
The factor represents the level of skill and control likely in site work. It is based on whether the site work is likely to be in accordance with manufacturers' recommendations and tightly controlled. The assessment should be based on the likelihood of achieving the designed level of workmanship, including issues such as storage, protection during installation, ease of installation, number of trades required for each activity, site applied coatings etc.

Factor D: Indoor environment
This factor indicates the environmental assessment, the exposure to agents of degradation and their severity. The general use of the building should be taken into account within this factor, together with relevant local aspects, for instance locations subject to wetting, such as kitchens and bathrooms.[11,12]

Factor E: Outdoor environment
A meso or local level designation may be adequate (e.g. coastal, polluted) for this factor. But for detailed design the microenvironment should be taken into account (e.g., southern elevation, wind suction or uplift at high-level, salt-spray zone). Also the specific use of the building may constitute an effect outdoors, and should then be taken into account. A combination of the agents can have a critical effect, for example a combination of wetting and freezing. Note also that components may be both exposed to external weathering and below ground water.

Factor F: Maintenance level
Even though the estimation is made on the basis of a planned level of maintenance, the likelihood of that being achieved for the type of building under consideration should be included, for instance may certain components be inaccessible or require special equipment for access. The expertise of cleaning should also be taken into account, as this may introduce agents not normally found (alkalis, bleaching powder etc).

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[11] The EOTA publication “Assessment of working life of products” contains tables of both indoor and outdoor environments, together with degradation agents to be taken into account for various materials.
[12] Internal and external environment are separated and for most components only one would apply, but certain components (e.g., those embedded in the building envelope) may be subject to degradation from both indoor and outdoor agents.

B Requirements for service life prediction

B.1 AGENTS CAUSING DEGRADATION

B.1.1 Climatic ranges

It has been stated [1] that due to the great variability of the environment and of the intensity and predictability of the agents, it is difficult, if not impossible, to make an exhaustive listing of the agents originating from the environmental constraints to which materials in service are subjected. Hence, every attempt in practice to measure and describe the degradation environment is an approximation and simplification.

One frequently used basis of classification for climate, and environment, is the division into macro, meso and microclimate (see e.g. [2,3]). This division means a definition of different scales describing the variations in the meteorological variables. There exist no common and exact definitions of the different scales.

By macroclimate is normally meant the gross meteorological conditions described in terms like polar climate, subtropical climate and tropical climate. The descriptions are based on measurement of meteorological agents such as air temperature, precipitation etc.

When describing meso climate, the effects of the terrain and of the built environment are taken into account. The climatological description is still based on the standard meteorological measurements.

The microclimate describes the meteorological variables in the absolute proximity of a material surface. The microclimate or microenvironment is crucial to understanding and estimating material degradation. The most important variables describing microclimate include relative humidity, surface moisture, surface temperature, irradiation and deposition of air pollutants.

B.1.2 Degradation agents

Degradation of exterior building materials mostly involves complicated chemical and/or physical processes governed by a great number of degradation agents. The standard ISO 6241:1984 (E) [4] classifies these agents relevant to building performance according to their *nature*, i.e. under the headings Mechanical, Electromagnetic, Thermal, Chemical and Biological agents.

In general, external to the building the *origin* of the agents is either the atmosphere or the ground, whereas internal to the building the origin is related to occupancy or design and installations. However, although not stated in ISO 6241:1984 (E), an agent acting externally while originating as a design consequence can also occur. Furthermore, the influence of agents originating from the atmosphere to internal degradation should not be disregarded.

Examples of mechanical agents are static and dynamic loads from wind, ice and traffic, whereas temperature and solar radiation mainly represent thermal and electromagnetic agents, respectively.

Water and solvents, oxidising and reducing agents, gaseous and particulate pollutants may serve as examples of categories of chemical degradation agents.

Living organisms such as fungi and bacteria are important environmental agents taking part in the deterioration of primarily organic materials. Biological agents may also contribute to degradation of inorganic materials, such as limestone and sandstone, by increasing the time of wetness and by causing the production of acid compounds.

A list of relevant degradation agents is presented in Table B.1:1, condensed from ISO 6241:1984 (E) [4].

Table B.1:1 Degradation factors affecting the service life of building materials and components

Nature	***Class***
Mechanical agents	*Gravitation* *Forces and imposed or restrained deformations* *Kinetic energy* *Vibrations and noises*
Electromagnetic agents	*Radiation* *Electricity* *Magnetism*
Thermal agents	*Extreme levels or fast alterations of temperature*
Chemical agents	*Water and solvents* *Oxidising agents* *Reducing agents* *Acids* *Bases* *Salts* *Chemically neutral*
Biological agents	*Vegetable and microbial* *Animal*

Some quantitative information on agents originating from the atmosphere is available from published weather and climatological data and from data published on air pollution.

B.1.2.1 Combined agents and combination of agents

Some agents are combined by more than one co-existing factor, e.g. an imposed force such as freeze-thaw stress is due to cycling temperature and to the presence of water. At the same time, water solely is a chemical agent. Temperature itself is the thermal agent, while for many chemical reactions the temperature is decisive for the reaction rate. Other agents of the same or different nature can give rise to significant synergistic effects, e.g. sulphur dioxide together with nitric oxides, and UV radiation together with oxygen (photo oxidation).

B.1.2.2 Chemical and physical incompatibility between dissimilar components

Incompatibility between components is considered as an agent related to the design, and occur normally only under certain conditions. By considering each component separately, incompatibility between a certain component and a neighbouring component can be treated analogous to, for instance, the case when the component is subjected to an agent originating from the atmosphere. That is, in general terms both the neighbouring component and the agent originating from the atmosphere are parts of the environment of the component, in accordance with ISO 6241:1984 (E), clause 6.4.1 [4]. Incompatibility includes, for example, corrosion caused by contact between dissimilar metals when moisture is present, or stress caused by different thermal expansion coefficients of rigidly connected dissimilar components at extreme temperature conditions.

B.1.3 Classification of degradation environment

Classification of the environment in terms of its corrosivity on a number of standard materials, mainly metals, has been carried out for wide areas, normally using a series of International standards specifying how to classify atmospheric corrosivity. Dose-response functions relating the dose of agents, mainly of chemical nature, to the degradation of materials, are also available from this work for certain classes of materials.

B.1.3.1 Atmospheric corrosivity for metals

The ISO standards ISO 9223 - 9226 [5] have been developed for the classification of atmospheric corrosivity and are specifically adapted to the corrosion of metals:

- ISO 9223:1992, Corrosion of metals and alloys – Corrosivity of atmospheres – Classification
- ISO 9224:1992, Guiding values for the corrosivity categories.
- ISO 9225:1992, Measurements of pollution.
- ISO 9226:1992, Corrosion of metals and alloys – Corrosivity of atmospheres – Determination of corrosion rate of standard specimens for the evaluation of corrosivity.
-

Classification employs two approaches ([6], sec.12.1), either environmental classification in terms of the corrosion-determining parameters Time of Wetness (TOW) and pollution, or classification based on corrosion rate measurements of standard metals exposed in the micro environment concerned.

With the first approach, the environment is classified with respect to the variables TOW, SO_2, and chloride. TOW is described in five classes, T_1 - T_5 ([6], Table 9). Classification of SO_2 and chloride is done in four classes, P_0 - P_3 and S_0 - S_3, respectively, ([6], tables 10-11). The corrosion categories ([6], Table 12) are determined from the fixed TOW and pollution classes.

In Table 13 [6], the corrosion categories for the four standard metals are described qualitatively. The categories may also, as mentioned, be directly determined by one-year exposure of the standard metals.

B.1.3.2 Description of the degradation environment for non-metals

The above mentioned standard approach for classification of the atmospheric corrosivity is primarily applicable to the corrosion of metals and cannot, without complementary amendments, be used for an appropriate description of the degradation environment for non-metals. However, in the series IEC 60721, *Classification of environmental conditions* [7], intensities of agents affecting the performance of electric components are classified. Where appropriate, these standards may also be applied to the environment of building components, non-metals as well as metals.

Degradation of non-metals is generally caused by other mechanisms than those responsible for the corrosion of metals, even if oxidation processes are important in most cases. For example, a generic classification of degradation environments for polymeric materials in outdoor use calls among other things for improved knowledge concerning moisture periods (Time of Wetness), surface temperatures, UV-radiation and the influence of air pollutants. There is a need for more knowledge about deposition mechanisms and concentrations of air pollutants deposited on all various types of material surfaces.

B.1.4 References

1 Eurin, P., Maréchal, J. and Copé, R. (1985) Barriers to the prediction of service life of polymeric materials, in *Problems in Service Life Prediction of Building and Construction Materials*, (ed. L.W. Masters), Martinus Nijhoff Publishers, Dordrecht, pp. 21-40.

2 International Council for Building Research Studies and Documentation (CIB), (1990) *Feedback from practice of durability data: inspection of buildings*, (ed. C. Sjöström), CIB Publication 127, Rotterdam.

3 Sjöström, C. and Brandt, E. (1990) Collection of in-service performance data: state of the art and approach by CIB W80/ RILEM 100-TSL, in *Proceedings of the 5th International Conference on Durability of Building Materials and Components, Brighton, 1990*, (eds. J.M: Baker, P.J. Nixon, A.J. Majumdar and H. Davies), E & FN Spon, U.K., 7-9 November, pp. 287-98.

4 International Organization for Standardization (1984) *Performance Standards in Buildings – Principles for Their Preparation and Factors to be Considered*, ISO, Geneva, ISO 6241-1984 (E).

5 International Organization for Standardization (1992) *Corrosion on Metals and Alloys. Corrosivity of Atmospheres*, ISO, Geneva, ISO 9223-6:1992.

6 Statens institut för byggnadsforskning (1987) *Påverkan på fasadytor från yttre miljö*, (Eds. C. Sjöström and J. Henriksen), SIB, Gävle, Sweden, (in Swedish).

7 International Electro technical Commission (1982) *Classification of Environmental Conditions. Part 2: Environmental Conditions Appearing in Nature*, IEC, Geneva, IEC 60721-2

C.1 LONG TERM STUDIES

C.1.1 Introduction

Traditional building products with a long successful use history can normally continue to be used without problems provided that the way of using them is not changed too radical. The experience gained over many years may even in some cases be extended to new materials of the same type, however, the way of using the product and the in-use environment is limited to the one covered by the experience gained with the original product.

When a new material, a new way of using an old material and/or the use in a new and different exposure environment is desired, it is no longer possible to rely on experience and an assessment of the service life is therefore necessary. Such assessments will most often be based upon a comprehensive series of tests including Field Testing as well as Short-Term Accelerated Tests. Neither of the approaches can be dispensed from. The short-term test methods, which are necessary to provide quick information, are normally considered more uncertain, i.e. they are more open to misleading results and misinterpretation if not designed and performed carefully. Short-term test methods will not be dealt with in this paper.

Long-term ageing tests are necessary to give confidence and to give correlation between results from short-term test and the behaviour under in-use conditions. Normally, long-term ageing does not need reference materials and is normally considered more reliable than short-term methods. On the other hand, field tests are slower, which is in conflict with the commercial interests of the producers. As can be seen, great attention must be paid to the planning and selection of methods for long-term ageing. The following gives an introduction to and lists the advantages and drawbacks that will facilitate the choice of test type.

C.1.2 Long term ageing — general considerations

Studies to generate data from long-term ageing under in-use conditions should be designed so as to give close simulation of the real building context, and ought to aim at improving the knowledge base for designing short-term test methods. Only a good agreement between the results of long-term experiments under in-use conditions and short-term test methods will allow for a later correlation of the degradation during the two different paths, thereby making prediction of service life possible. Experiments should always be designed to account for all the factors which might affect the durability of the product under investigation. If a product is meant for use in a specific environment, an evaluation of the degradation factors in that specific environment is satisfactory. For global use, a more thorough test program with a wider range of degradation factors so as to account for every imaginable use situation might be necessary.

Data from long-term ageing are achievable in different ways either from tests/experiments based on long-term tests on special test sites or from in-situ monitoring of buildings.

It should be remembered that the results obtained relate to the specific investigation site. There may be great difficulties in transferring results from one geographic location to another, as the present individual degradation factors may vary considerably and with them the synergistic effects. This problem may to some extent be overcome by using more test sites/objects — i.e. by intentionally selecting a variety of exposure sites or by statistical sampling of objects when inspecting buildings.

As weather does not repeat itself — i.e. every year is not a standard year — one has to be cautious in drawing conclusions from one exposure period to another. This is especially the case if the exposure period is less than a full year. The longer the test period the more reliable the results, as stochastic variations are more likely to be levelled out. It is necessary to account for the exposure environment (and that statement is valid not only for field tests but also for short-term accelerated tests).

In order to obtain sufficiently reliable results many test specimens may be necessary, as the response of for instance a coating system or of concrete specimens may vary considerably.

Even though test results are often used for the prediction of the service life of a product, it should be pointed out that in many cases long term tests may be used only to determine the environment which exists in a certain type of building structure, in order to evaluate whether or not a material will get a long service life. For example, it is well known that a major degradation factor for untreated wood is dry rot, which is normally only present at high humidity levels. A prediction of a long service life might therefore be based upon knowledge that the environment is sufficiently dry to avoid dry rot (presupposing of course that other degradation mechanisms may be considered of less importance).

The different ways of generating data from long-term ageing are:

- Field exposure testing
- Experimental buildings
- In-use testing
- Inspection of buildings

The demands and needs regarding evaluation techniques etc. are basically the same for all four approaches; however, there are also differences that are reflected in the advantages and drawbacks of each type of long-term ageing tests.

In the present third working period the committee has continued its work related to achieving data from long term ageing under in-use conditions but this time with emphasis on field exposure tests, experimental buildings and in use testing.

C.1.3 General considerations

Before discussing the long term ageing tests in detail there are some common considerations that are valid for all types of long term ageing tests and which should always be kept in mind irrespective of the type of experiment:

- The results obtained relate to the specific investigation site. There may be great difficulties in transferring results from one geographic location to another, as the present individual degradation factors may vary considerably and with them the synergistic effects. This problem may to some extent be overcome by using more test sites — i.e. by intentionally selecting a variety of exposure sites or by statistic sampling of objects in inspection of buildings.
- As weather does not repeat itself — i.e. every year is **not** a standard year — one has to be cautious in drawing conclusions from one exposure period to another. This is especially the case if the exposure period is less than a full year. The longer the test period the more reliable the results, as stochastic variations are more likely to be

levelled out — it is necessary to characterise the weather for as well long term in use conditions as for accelerated tests.

- In order to obtain sufficiently reliable results it may be necessary with many test specimens, as the response e.g. of a coating system or of concrete specimens may vary considerably.

Even though it is the task of the joint CIB/RILEM group to give advice concerning ways of predicting the service life, it should be pointed out that in many cases long term tests may be used only to determine the environment which exists in a certain type of building structure, in order to evaluate whether or not a material will get a long service life. For example it is well known that a major degradation factor for wood is dry rot, which is normally only present at high humidity levels. A prediction of a long service life might therefore be based upon the knowledge that the environment is sufficiently dry to avoid dry rot (presupposed that other degradation mechanisms may be considered of less importance).

C.1.4 Field exposure testing

C.1.4.1 General

Field exposure tests (or exposure site testing) are since long in operation and a deep and widespread knowledge exists among materials researchers about the applicability and usefulness of field exposure testing of different types of materials. Standardized ways and methods of performing field exposure tests are thus available from different national and international organizations like ASTM and ISO. Two examples are the: ASTM G7-89 "Standard Practice for Atmospheric Environmental Exposure Testing of Non-metallic Materials" and ASTM G 90-91 "Practice for performing Accelerated Outdoor Weathering of Nonmetallic Materials Using Concentrated Natural Sunlight".

Field exposure testing may range from small tests performed by a materials producer to international co-operation in large programmes, e.g. the International Materials Exposure Programme (IMEP) co-ordinated by the United Nations Economic Commission for Europe (UNECE). Some of the programmes performed have led to the formation of databases concerning environmental measurements and materials degradation factors. The majority of these programmes have formulated dose response relationships relating the environmental factors to materials degradation.

The location of the test sites normally coincides with an environment where one or more of the commonly recognised major weathering factors are at a high level. Florida and Arizona in the U.S. and Bandol in the South of France, for instance, have high levels of sunlight and test sites are set up here to test materials vulnerable to UV-light. Similarly, other test sites are set up to meet other requirements, e.g. acid rain or freeze thaw cycles under humid conditions. Commercial test sites are often used by the industrial sector and some of them are set up to meet standards specifically written to meet industry's needs.

Besides "natural" exposure some of the test sites use "semi-natural" testing where one of the stresses in the environment is increased or an additional factor is added.

An example of increasing a stress factor is the above-mentioned ASTM standard G 90-91 "Practice for Performing Accelerated Outdoor Weathering of Non-metallic Materials Using Concentrated Natural Sunlight". In this method highly effective reflectors are used to focus multiple reflections of sunlight onto the surface of the specimen. The light might be increased to about 5 times its natural level. It should be noted that also the temperature increases during the test and this factor must be taken into consideration.

An example of an additional factor is the testing of building products in a combination of natural weathering and cyclic movements. This approach has been used in several countries and for several materials. The additional factor might be considered compensation for stresses subjected to components under use. As an example, the cyclic movement might compensate for the movements and stresses anticipated in a sealant in an actual building structure. The frequency and the magnitude of the movements must be designed according to the expected movements under in-use conditions.

C.1.4.2 Advantages

1. It is often possible to use many specimens as these are usually of limited size. This opens up for the possibilities of different exposure times, different exposure environments etc.

2. At field exposure testing the weathering is genuine, i.e. there are no problems regarding the size and distribution of the degradation factors (except maybe for accelerated tests).

3. The environment is well defined and should be recorded in detail during the test, in order to account for variations in the weather and to get the best possible picture of the actual exposure conditions.

4. Even though it is natural ageing, some acceleration is possible, e.g. using a 45° inclination towards the sun or by selecting special locations with high levels of one or more important degradation factors, e.g. UV-light.

5. It is often possible to use (sophisticated) laboratory investigations of specimens, thereby increasing the possibilities of detecting (early) signs of degradation.

6. Some acceleration is possible using special devices to increase one of the stress factors.

C.1.4.3 Drawbacks

1. Usually the specimens are not full size, which may limit the use of the results.

2. Normally, the exposure is not a simulation of the in-use conditions in all respects, e.g. user-induced conditions like the influence from in-door climate will not be simulated.

3. In many cases there will be no variability of exposure conditions (only one test site), which may limit the use of the results.

4. The interpretation of results from exposure site to use site may be rather difficult especially as regards sites that differ considerably in climate, e.g. from Arizona's dry desert to Copenhagen's humid, temperate climate.

5. For accelerated tests, the results might differ from "pure" natural weather tests.

C.1.5 Experimental buildings

C.1.5.1 General

Durability testing of building products may be carried out by testing the product in special experimental buildings — made specifically for testing purposes.

There are — to the author's knowledge — no standards describing the use of experimental buildings. Nevertheless experimental buildings (or full-scale test houses) have been used quite extensively.

Test houses can range from very simple houses built for one specific purpose to advanced constructions with interchangeable roofs and walls that can turn with the sun. In its most simple form the test house is used only for exposing full-scale components or elements to the weather. In more advanced experimental buildings there may be controlled in-door climate simulating the expected in-use conditions and equipment to monitor the exposure environment as well as properties etc. of the tested product. Experimental buildings are used for studies by manufacturers as well as by test laboratories. An example of its use is the study of moisture transmission and the risk of condensation in roof constructions that might eventually lead to degradation and thereby decreased service life.

In a Nordic research programme information about design and use of 8 full-scale test houses in the Nordic countries were compiled [5].

C.1.5.2 Advantages

1. For experimental buildings the weathering is genuine, i.e. there are no problems regarding the size and distribution of the degradation factors.
2. The environment is well defined and should be recorded in detail during the test, in order to account for variations in the weather and to get the best possible picture of the actual exposure conditions.
3. The tests are run under nearly in-use conditions, as the normal user related influences to some extent may be simulated, e.g. by keeping the in-door temperature and relative humidity at levels normally expected in households.
4. The specimens are full scale thereby making it possible to find for instance deflections of components or the influence from adjoining materials.
5. It is often possible to cut out small samples and use (sophisticated) laboratory investigations of specimens, thereby increasing the possibilities of detecting (early) signs of degradation.
6. Even though it is still difficult, translation of results is easier than for field exposure testing.

C.1.5.3 Drawbacks

1. Acceleration is difficult except for the selection of exposure site (and here the possibilities are normally very restricted).
2. As this is a full scale testing the number of specimens are usually rather limited.
3. Only very rarely exposure will take place at more than one location, i.e. there will be a lack of variability in the exposure environment.

C.1.6 In-use testing

C.1.6.1 General

By in-use testing is meant an intentional use of new produts in a normal building or structure under the intended use conditions, in order to evaluate the service life of the product. The product might be a new material or component or it might be a well known material used in a new way. The aim of the approach is to create an experimental situation as controlled as possible, where the specimen is under the influence of the full range of degradation factors of the actual in-service situation.

There is — to the author’s knowledge — no standards describing in-use testing even though it is a very normal procedure for industry as well as for research laboratories. In-use testing is often the last step — used to provide industry with its final proof of serviceability and durability — before a product is put on the market on a larger scale.

C.1.6.2 Advantages

1. With in-use testing the weathering is genuine, i.e. there are no problems regarding the size and distribution of the degradation factors.
2. Also the rest of the in-use conditions are genuine, e.g. user-induced stresses etc. will occur as the tests take place in real buildings and structures.
3. Provided it is possible to use more exposure sites a variety of exposure environments may be used (depending on economy/number of tests).
4. The specimens are full scale thereby making it possible to find e.g. deflections of components or the influence from adjoining materials.
5. The interpretation of test results is easy provided a sufficient number of test specimens (and range of exposure sites) are available.

C.1.6.3 Drawbacks

1. It may be difficult to keep track of the exposure environment, as recording normally is not performed on the exposure site.
2. The use of destructive testing or sophisticated lab test methods might be a problem, as the buildings normally should be left unharmed.
3. It is not possible to perform accelerated test, except the limited possibility of choosing a severe location.

C.1.7 Inspection of buildings

C.1.7.1 General

Inspection of buildings has always been used to gather data about long-term service — this is in fact what our experience is based upon. Only recently, however, the generation of data has been performed in a more methodological manner in order to obtain the best results.

Most field inspections of buildings that include an assessment of the performance of materials and components are descriptive. The purpose of the inspections may be to describe the in-service state of materials for single buildings, e.g. as part of a maintenance routine or for a whole well defined population of buildings when general information about service life of for instance materials in the building envelope is wanted.

Depending on the purpose the investigation may be non-statistical or statistical. If the goal is to generalise, an inspection of a statistical sample of objects from the population is an effective way of performing the investigation.

Inspection of buildings was the subject of the work of the previous joint CIB W80/RILEM 100 TSL committee. The work of this committee was reported in two CIB reports — references [3] and [4], where more detailed information about inspection of buildings is given including the use of statistical sampling for generalising.

C.1.7.2 Advantages

1. The weathering is genuine, i.e. no problems regarding the size and distribution of the degradation factors exist.
2. Also the rest of the in-use conditions are genuine, e.g. user-induced influences, as the tests take place in real buildings and structures.
3. It is possible to use as many objects as wanted (provided economy allows it) in order to get the most reliable result.
4. Through statistical sampling it is possible to reach reliable results and to generalise.
5. The specimens are full scale thereby making it possible for instance to find deflections of components or the influence from adjoining materials.
6. The interpretation of test results is easy, provided a sufficient number of test specimens (and range of exposure sites) are available.

7. It is always possible to take up studies of new objects, thereby "immediately" getting knowledge of long-term in-use behaviour provided that it is possible to find a sufficient number of objects to study, and it is of course not possible to study materials or components of a more recent date.

C.1.7.3 Drawbacks

1. It is difficult to keep track of the history of the materials and components under investigation.

2. It is difficult to measure and describe the in-use environment and it is normally impossible to find a sufficient record of the actual exposure conditions.

3. The use of destructive testing or sophisticated lab test methods might be a problem, as the buildings normally should be left unharmed.

4. It is only possible to study objects of some age, and built-in instruments normally cannot be used.

C.1.8 Measurements and evaluation — general considerations

It is necessary to plan and perform the measurements and evaluation of results in a systematic manner in order to get the most out of the tests. There are, however, differences between the methodology used for inspection of buildings and the other three ways of generating long-term data.

For inspection of buildings there is a wide range of methods especially suited for this purpose. These methods were treated in the work of the previous CIB/RILEM group.

The methods applicable for testing and evaluation of materials and components tested at field exposure testing, experimental buildings, or in use testing are far more numerous as they range from the methods used for inspection of buildings to sophisticated laboratory test methods. Therefore the possibilities for testing are often better than for inspection of buildings. The more sophisticated laboratory test methods are currently treated in a sub-group of the present CIB W80/RILEM 140 TSL committee. For the three last-mentioned methods it is from the very beginning the intention that measurements are to be performed at a later stage. This is one of the reasons why more sophisticated methods can be used, and besides it gives much better possibilities for planning, which is essential to a good result. The planning should at least encompass:

- What shall be done (what are we looking for)?
- When shall it be done (e.g. when can changes etc. be expected)?
- How shall it be done (which methods are we going to use)?
- Can special measure devices be incorporated — and if yes how?
- How do we best assure the usefulness of the results (choice of object, location etc.)?

C.1.9 How is degradation evaluated?

The information sought in a specific investigation depends on the purpose of the investigation. Usually information is requested about how properties are changing in time and for bigger test specimens in supplement, the presence and extent of defects in conjunction with exposure time are of major interest. It is also of the utmost importance to find the degradation mechanisms, i.e. what caused the failure — the decisive degradation factor — and what was the effect — what actually happened to the material/component because of the influence of the decisive factor.

Depending on the purpose of the test, the result may be the evaluation of a single property — e.g. energy to break of a concrete test-specimen — or a more general evaluation of several properties of a building component. In some cases, it may be practical to distinguish between different levels when assessing a specimen. These levels may be described in different ways depending on the purpose of the investigation. An example of the use of four condition levels may be as follows — cf. also Table C.1.9:1:

- Condition 0: Intact, no changes
- Condition 1: Minor damages, some maintenance is suggested
- Condition 2: Malfunction, maintenance needed as soon as possible
- Condition 3: Out of order, replace or repair immediately

Table C.1.9:1 Presentation of results described at four different condition levels

Condition level	0	1	2	3
Property 1				
Property 2				
Property 3				
Property 4				
Property 5				
Property 6				

C.1.10 Description of exposure environment

In order to design short-term accelerated tests and to make correlations between short-term tests and long-term ageing, it is necessary to have good knowledge about the exposure environment. For this reason it is essential to measure and record as much information about the exposure environment. For this reason it is essential to measure and record as much information about the exposure environment as possible. Reference is made to existing standards for instance ISO 8565-1992 "Metals and alloys — Atmospheric corrosion testing — General requirements for field tests".

The macroclimate is in this connection often insufficient to describe the exposure environment as local conditions might have a major effect on the actual climatic influence. To get precise information about degradation factors qualitatively as well as quantitatively it is therefore necessary to measure the meso and microclimate that describes the climate down to the absolute proximity of the materials surface. Direct measurements may be exemplified by measurement of surface moisture by means of mini sensors and ultra violet radiation by means of dosimeters applied either directly on or in the absolute proximity of the surface of the specimens.

C.1.11 The build up of a detailed routine/test method

In order to achieve the incorporation of all the above mentioned a detailed routine/testmethod should be elaborated for the execution of the test. This should include at least the following :

1 Scope
    — The purpose of the test/experiment
2 Field of application
    — Where may this routine be used, what are the limitations
3 Administrative information
    — Name and address of the testing laboratory
    — Name and address of the test site
    — Name and address of the client

4 References
    — Exposure standards used (and wished)
    — Test methods used
    — Evaluation methods used
5 Definitions
    — E.g.. used terminology
6 Sampling
    — Method of sampling
7 Test object
    — Description of the test object
8 Test conditions
    — Description of the exposure conditions (e.g.. method and site)
    — Time planning
9 Reporting
    — Necessary documentation
    — What types of information should be incorporated in a report (specify)
    — Test results
10 Conclusion
    — What may be concluded from the experiments?
    — Prediction based on mathematical modelling

C.1.12 Reporting of data

The reporting of long term ageing test should at least include the following headlines:

- The purpose of the investigation
- Present the aim of the entire investigation in general and each sub test in particular
- Performer of the investigation
- Present the institute/company and name and affiliation of the person responsible for the investigation
- Inspected object or objects
- Give conclusive descriptions of the inspected materials, components etc.
- Plan for the investigation
- Describe the used plan in headlines
- Methods for measuring and assessment
- Present a compilation of the used methods which gives a complete description of said methods
- Description of exposure environment
- Present conclusive results from performed measurements of exposure environment and account for methods used
- Compilation of results
- Present each individual result, calculations made and uncertainty of result
- Presentation and discussion of results
- Present final result and discuss validity and reliability

C.1.13 Future research needs

There is still a great need for research and development within long-term ageing tests, for instance further development of routines for evaluating long-term degradation by use of field exposure tests, experimental buildings and in-use testing. Especially one should encourage the development of internationally recognized methods which would make future use of test results more easy and valuable.

For field exposure tests, the research needs are especially on dose response functions for materials and development of further (and in particular international) standards.

For experimental buildings and in-use testing, the research needs are first and foremost to develop standards for the use or at least to give technical recommendations or guidelines for the use.

C.1.14 References

1. Masters, L.W. (ed.) (1986), *Prediction of Service Life of Building Materials and Components***.** CIB Publication 96, International Council for Building Research Studies and Documentation, (CIB), Rotterdam.
2. Masters, L.W. and Brandt, E. (1989) Systematic Methodology for service Life Prediction of Building Materials and Components, *Matériaux et Constructions/Materials and Structures*, 22, pp. 385-392.
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4. Brandt, E. (ed.) (1990), *Feedback from Practice of Durability Data — Appendix — Examples of Field Investigations of Buildings and Structures***,** CIB report 128, International Council for Building Research Studies and Documentation (CIB), Rotterdam.
5. Gjelsvik, T. (1985), *Large Scale Test Facilities for Durability Studies in Scandinavia.* Report No. 97, Norwegian Building Research Institute (NBI), Oslo.

Appendix C.1.A Available standards within exposure testing

"Standard Practice for Atmospheric Environmental Exposure Testing of Non-metallic Materials", ASTM G 7-89, USA.
"Practice for performing Accelerated Outdoor Weathering of Non-metallic Materials Using Concentrated Natural Sunlight", ASTM G 90-91.
"Outdoor weathering of plastics in the Australian environment", Part 1: Commercial products, Part 2: Guide for design purposes, AS 1745.1 & .2 — 1989, Aus.

C.2 Reliability based service life prediction

C.2.1 Introduction

The development of new building materials, building products or complete buildings (these are referred to as building elements) is directed to achieve a set of functions, such as safety, strength, water-tightness, protection and aesthetics. These functions have to be fulfilled during a certain period: the building element must be durable. The intended service life can range from a couple of years (for example wall paper) or many decades (for example structural framework) to centuries (for example large civil engineering structures like storm surge barriers). In practice the service life is hardly determined in an explicit way during the design process. The design is mainly based on measures that are intended to improve the durability. On the basis of knowledge with respect to possible deterioration processes, accelerated and field exposures, an implicit impression of the service life is gained. The practice shows finally the real service life.

To improve this situation with implicit expressions of the service life, new procedures have been developed for establishing the service life in an explicit way. An example of this is the procedure from the joint committee CIB W80/RILEM 71-PS “Prediction of the service life of building materials and building components”. Although the procedure is an important step forward, there are some practical shortcomings. In the first place there is no framework for establishing a target value for the service life. In the second place the functions of the building elements are not clearly defined. Furthermore, the fact that the service life shows a big scatter is an important practical obstacle.

Due to the fact that the service life has a big scatter, it is inevitable to treat it as a stochastic quantity. That means that the service life cannot be expressed as a single deterministic value. It has to be described in stochastic terms with the aid of a density function (see fig C.2:1).

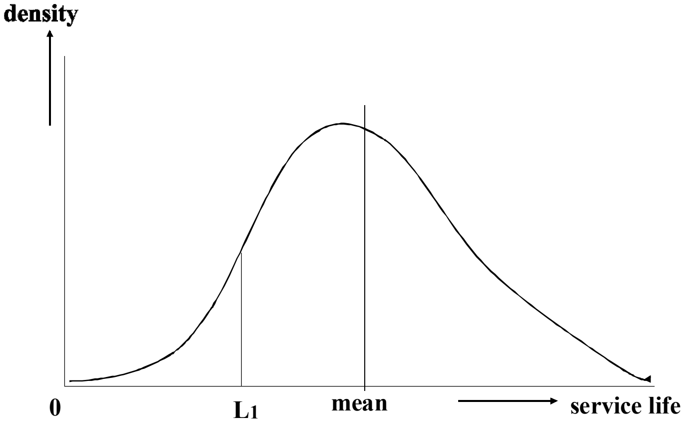

Fig C.2:1 Example of the probability density function of the service life

The area under the curve left from the value L_1 represents the probability that the value L_1 will not be reached, this is called the failure probability.

The service life ends if one of more essential functions can no longer be fulfilled by the building element. Therefore, it is essential to determine in an objective way the event of malfunctioning. This means that a method and a limit value must be available to establish the functioning. In other words the functioning must be expressed as a performance.

Fig 2:1 makes clear that a decision has to be taken with respect to the value of the accepted failure probability within the intended period of use. This value depends on the consequences of failure, such as economic loss or lack of safety. If these are small, a relatively high probability may be acceptable, but if they are considerable, the probability must be low. For wallpaper a high failure probability applies and for storm surge barriers the failure probability has to be very low. It is a common habit to use the expression ”reliability” instead of ”failure probability”. Reliability stands for the probability that no failure will occur. As the maximum value for the probability is equal to 1, the following relationship applies:

Reliability = 1 - failure probability (1)

The minimum reliability in the case that human life is threatened is often stated by legislation or in (structural) building codes. If failure involves only economic loss, an economic optimisation can be used to establish the accepted value. In fig C.2:2 a simplified example of such an optimisation is given. The figure shows that the basis price C_b of the considered product rises (in this example linear) with increasing reliability, while the risk R decreases (in this case non-linear). The risk is defined as the product of the failure probability and the failure costs (direct and indirect costs). The total cost is the sum of both: $C_{total} = C_b + R$. Where C_{total} has a minimum, the corresponding reliability is the (economic) most optimal value.

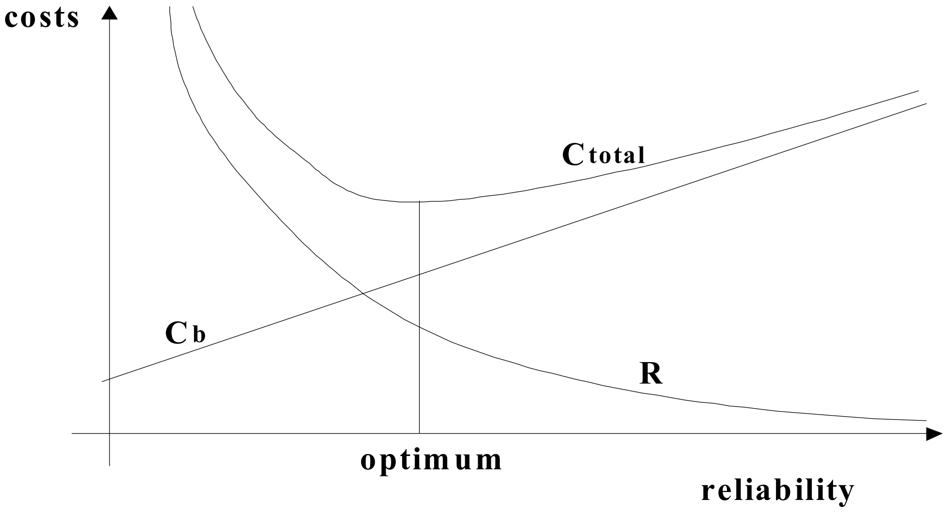

Fig C.2:2 Example showing the optimum reliability, leading to a minimum of total costs

The example in fig C.2:2 can be extended with other life cycle costs like maintenance, repair, energy etc.

As there exists a close relation between design, reliability and maintenance, it is relevant to specify a ”minimum” maintenance plan for the building element already at the time of design. This specification gives directives on the amount and quality of future inspection and repair actions, necessary to maintain the required reliability.

Although the probabilistic approach is formally the only correct approach, it cannot be expected that all designs for building elements will be made in this formal way. For that reason it will be necessary to develop simplified rules, that have to be calibrated to probabilistic designs. This situation is comparable to the practical structural design procedures, where no formal probabilistic calculations are made. Instead the calculations are based on characteristic values for loads and strengths, and on partial safety factors that ensure the target reliability and economy of the structure.

Today, for structural engineering the performance-based design has been developed to a high level. According to the ongoing activities [1-3] it can be expected that it will be developed further towards durability design for concrete and other structures. Therefore it is also logic to extend the concept to the durability of building elements in general. This will also make it possible to base the design on the mentioned CIB/RILEM procedure for the prediction of service life.

C.2.2 Performance based structural design approach

The structural design approach in the modern building codes is based on the principles of the reliability analysis. This approach has proven to be very successful as it provides a rational and objective design. The basis of the conventional design procedure for the safety and the serviceability of structures with a static loading can be expressed as the limit state function:

$$R - S = R\ (X_1, X_2, ..., X_n) - S\ (X_{n+1}, X_{n+2}, ..., X_m) > 0 \qquad (2)$$

where:

R is a function that describes the load bearing capacity of the structure

S is a function that describes the influence of the load on the structure

X_i is a basic variable for the functions R or S

Part of the basic variables X_i may be model factors representing the uncertainty in the models for R and S. The set of limit states to consider is in principle not restricted to the standard set that is present in most building codes.

A limit state can in general be defined as the border that separates desired states from the undesired or adverse states. A desired state is a situation that is acceptable to the owner of the structure or local authorities or insurance companies, where acceptable is defined as satisfying the primary functional demands.

In all modern structural building codes, two main types of limit states are distinguished:

- Ultimate limit state (ULS)
- Serviceability limit state (SLS)

The ultimate limit states refer to collapse, fracture, overturning, lifting or sliding and other events where the safety of the structure is of importance. In principle this refers to the loss of static equilibrium. The structural serviceability limit states refer to the functionality (fitness for purpose). There is, however, no obstacle to extend them with aspects like comfort for the user, aesthetic or cosmetics. In Table C.2:1 examples of serviceability items are presented. The Table also gives some examples of durability aspects. It is already mentioned that, in the present design concepts, these aspects are not approached via the service life.

Table C.2:1 Examples of serviceability aspects

Aspect	**Examples**
User comfort	static deflection vibrations local structural damage settlements of foundations
Functionality	unevenness of floors drainage of roofs jamming of doors and windows out of plumpness functioning of elevators
Cosmetics	out of plumpness large deformations cracks and fissures gaps between components efflorescence soiling and staining graffiti
Durability	crack width in concrete corrosion of reinforcement protection by coatings deterioration of protection systems

The basis of a limit state consists of the fact that after crossing into the unacceptable region, direct measurable consequences follow: the structure does no longer meet its primary demands. Indeed this is a performance-based approach.

This design procedure, given by relationship (2) is worked out in such a way that the failure probability is restricted:

$$P\{failure\} = P_f = P\{R - S < 0\} < P_{target} = \Phi(-\beta) \qquad (3)$$

where:

P_f	the probability of failure of the structure, this can also be noted as $P\{failure\}$
$(1 - P_f$	the reliability of the structure)
P_{target}	the accepted maximum value of the probability of failure
Φ	the standard normal distribution function
β	reliability index (this parameter is normally used in codes instead of the failure probability)

In fig C.2:3 a simple example of the probability density functions of R and S is given, demonstrating that combinations of R and S are possible where relationship (2) is not valid.

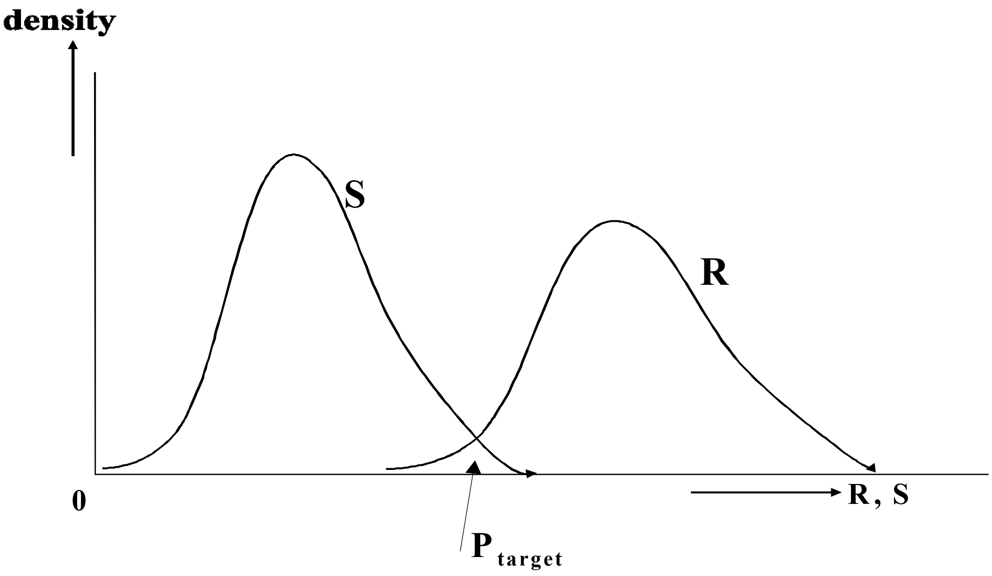

Fig C.2:3 Failure probability as a consequence of the probability that $S > R$

With the aid of probabilistic techniques this failure probability can be calculated. As such techniques can be very time consuming, the calculation has been simplified for normal design calculations. Despite the simplifications the same reliability will be reached more or less. For instance, in a semi-probabilistic procedure relationship (3) can be translated in:

$$R_c / \gamma_R - S_c \cdot \gamma_S = R_d - S_d > 0 \qquad (4)$$

where:

R_c	load bearing capacity of the structure based on characteristic values
γ_R	material factor
S_c	characteristic value of the influence of the loading
γ_S	load factor
R_d	design value of the load bearing capacity
S_d	design value of the load

The characteristic values are often defined as values with a probability of exceedence of 5%, but other probabilities are possible.

C.2.3 Durability design

In most cases durability concerns the serviceability of the structure. The simplest category is the one that is directly recognised as for instance a problem of cosmetics. But also if limits for necessary repair or maintenance actions are exceeded, this is usually considered as a serviceability problem.

Many building codes indeed relate durability to serviceability, as for instance in the case of concrete codes. In particular crack width limits are defined as serviceability limit states. Only in cases where deterioration might go on unobserved, the durability problem might be directly associated with an ultimate limit state. As an example, consider steel anchors for fixing facade elements. Corrosion may easily go undetected and collapse of the facade element may be the eventual result.

In the performance based structural design both the resistance R and the load S are considered to be time independent. In many loading situations this is not realistic. Relationship (2) should than be rewritten as a time dependent limit state function:

$$R(t) - S(t) > 0 \qquad (5)$$

Special cases for this limit state function occur if either R or S is not time dependent. The working out of these relationships does in principle not differ from working out (5).

Relationship (5) applies for all t in the time interval $(0,T)$. T is the intended service period (i.e. reference period). Even if the loading or the capacity of a structure is time dependent, the limit state functions for the design of structures are never formulated in this way (with an exception for fatigue). They are always simplified to time independent quantities for which relationship (1) can be used. Well known simplifications include:

- Assuming that the material strength during the service life period is either equal to the short term strength or to the long term strength
- Defining for static loads one characteristic maximum value, related to the service life period
- Defining for fluctuating loads (like wind, traffic or waves) one characteristic maximum fluctuation in the service life period

From a mathematical point of view it can be stated that relationship (5) is of the same type as the functions that will have to be solved for the durability design. These relationships are the transition between time independent design and durability design. If a part of the basic variables X_i refer to degradation these relationships constitute the durability design formulae.

Durability design can be presented in two apparently different, but essentially equivalent, formats. These are the: design for the “intended service period” (also called design for the “reduced resistance”); and, “lifetime” design (also called “time domain” design). For design for the “intended service period”, the limiting condition is that the limit state, at a given reliability, may not be reached within the intended service period. The format for the durability design is, in a large measure, comparable to the format for conventional structural design. For the “lifetime” design, the reliability of the structure is related to the probability that the design lifetime will be exceeded.

The intended service period concept can be expressed in a design formula, equivalent to relationship (3):

$$P_{f.T} = P\{R - S < 0\}_T < P_{target} = \Phi(-\beta) \qquad (6)$$

where:
$P_{f.T}$ probability of failure of the structure within T
T intended service period

Probably it will be possible at a later stage to simplify relationship (6) to a similar one as for the conventional design procedure (4).

In some structural codes indications have already been given for values of T and P_{target}. For example, in Eurocode 1 for the ultimate limit state (collapse): T = 50 years and $\beta = 3.8$

corresponding with $P_{target} = 7 \cdot 10^{-5} \approx 10^{-4}$.

For the lifetime design relationship (5) must be transformed to a lifetime probability distribution function. This can be done by describing it as an explicit function of time:

$$F(L) = F(t\{R,S\}) \qquad (7)$$

where L is the lifetime of the structure.

The reliability of the structure can be introduced by limiting the probability of exceeding a target value:

$$P_f = P\{L < T\} < P_{target} = \Phi(-\beta) \qquad (8)$$

A possible, more practical form for a design formula is:

$$t_d > t_{target} \qquad (9)$$

where:

t_d design value of the lifetime
t_{target} target lifetime

A special case for a durability design is the accumulated damage design. In this instance the deterioration process is described by a monotonic increasing or decreasing function $D(t)$ where t is time. Assuming that some critical level D_{cr} for $D(t)$ can be defined, then in this case, the limit state function for a durability problem can be formulated as:

$$g(\underline{X},t) = D_{cr} - D(t) \qquad (10)$$

where $\underline{X} = (X_1, X_2, X_3, \ldots., X_n)$ are stochastic variables modelling uncertain quantities as before.

In fig C.2:4 the lifetime design for an ultimate limit state design is illustrated by means of the probability density function of the service life. This example is based on a target service life of 50 years and a target reliability $\beta = 3.8$ (comparable with the requirements in for example the Eurocodes). The symbol σ stands for the standard deviation of the service life. Without any further probabilistic calculation it is clear that the mean service life has to be some centuries. This is for most of our structures far beyond experience.

Fig C.2:4 also indicates that the margins between the mean service life $\mu(L)$ and the intended service period can be very large, depending on the type of the distribution, the mean, the standard deviation (scatter) and the target failure probability. The margin in the figure is approximately $\mu(L) / L_{target} = 250/50 = 5$. In other examples other margins can be found.

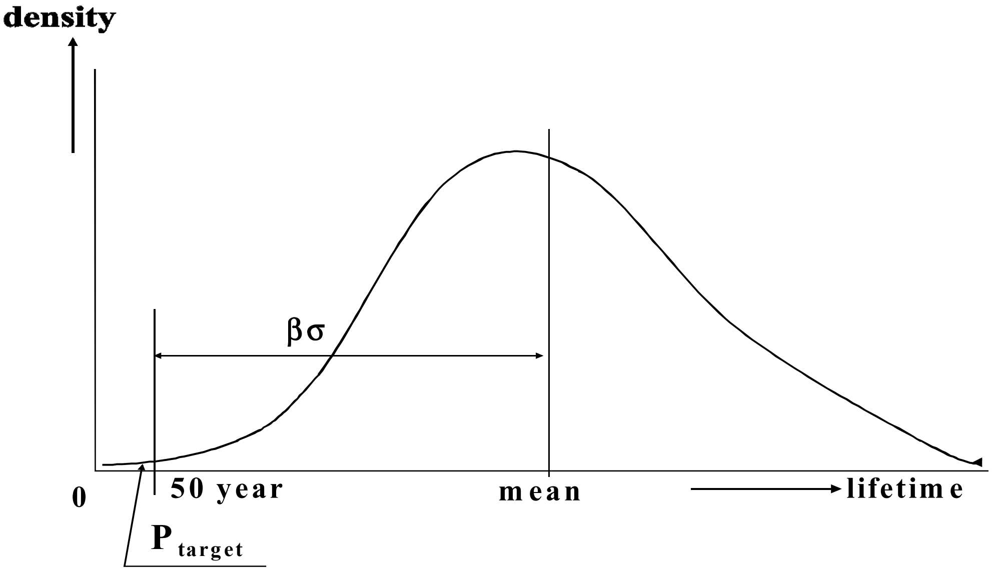

Fig C.2:4 Example of a service life distribution for the ultimate limit state of a structure

For many building elements the requirements are lower. In fig C.2:5 an example is given for a target service life of 25 years and a reliability index *b* = 1.64 (failure probability approximately 5%). These values may for example be true for luxury parquet. This figure shows that the mean service life is more restricted. The margin between the mean service life and the target service period is approximately $m(L) / L_{target}$ = 50/25 = 2.

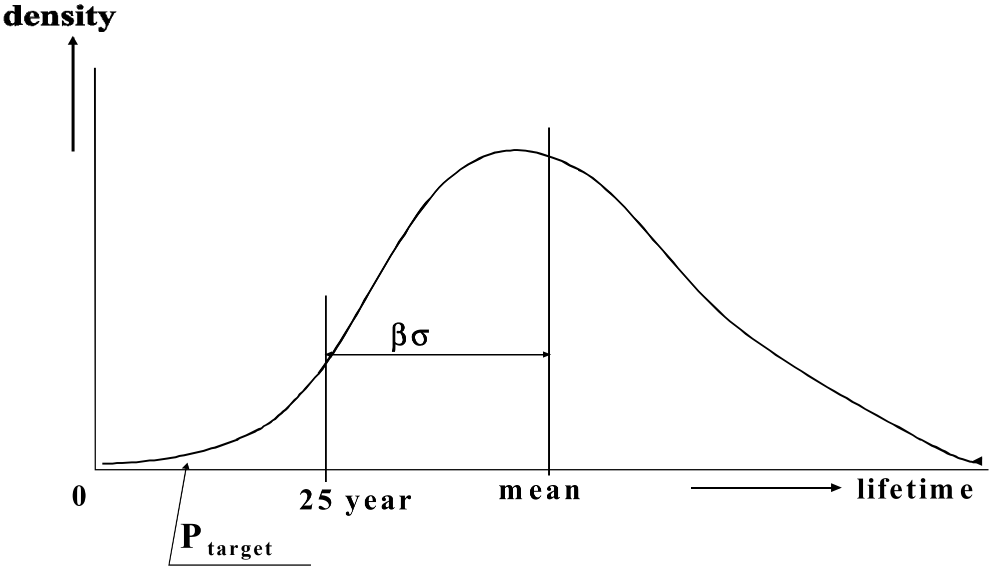

Fig C.2:5 Example of a service life distribution for a serviceability limit state

Fig C.2:6 shows the similarities and differences between the intended service period design (in the upper part of the figure) and the lifetime design (in the lower part of the figure). The upper part can be compared with fig C.2:3, but now being given for the whole period from 0 to *T*. The lower part can be compared with figs C.2:4-5. The illustration makes clear that for both approaches exactly the same information has been used. Consequently they will lead to exactly the same result.

It is important to realise that both the intended service period design and the lifetime design are not necessarily restricted to the conventional ultimate and serviceability limit states. The processes (or mechanisms) involved can be of mechanical, physical, chemical or biological nature. As been mentioned before, the limit states can refer to aspects such as structural safety, serviceability, functionality, comfort, aesthetics and so on.

The number of limit states (and their types of criteria) that have to be considered for one building is very large. That means there is a need for priorities and simplifications. In the context of this chapter no attention will be paid to that.

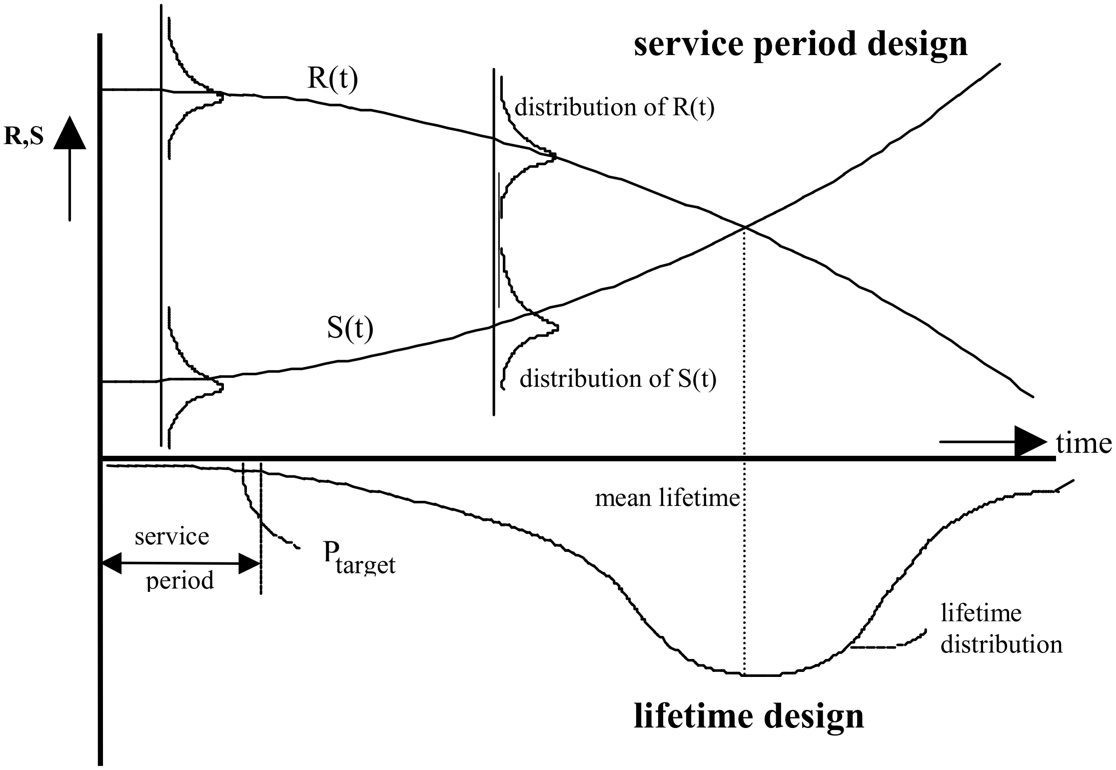

Fig C.2:6 Similarity between the service period and the lifetime approach

Table C.2:2 gives some indications about values for the target reliability in existing codes for ultimate and serviceability design of structures. For most limit states with respect to the durability of building elements the target reliabilities will be lower.

Table C.2:2 Indication of target reliabilities β related to the design life [4]

Relative cost of safety measures	**Serviceability limit states**	**Ultimate limit states**		
		Expected consequences given a failure		
		Minor	Moderate	Large
High	1,0	2,8	3,3	3,8
Moderate	1,5	3,3	3,8	4,3
Low	2,0	3,8	4,3	4,8

C.2.4 Probability theory [4]

C.2.4.1 Uncertainty

Often the limit state models are subject to several sources of uncertainty. Three types of uncertainty can be distinguished:

- Physical uncertainty – due to the natural variability in the properties of a building element some of the quantities used to model the behaviour of the element will be subject to uncertainty
- Model uncertainty – is related to the mathematical formulation of the model e.g. by introducing a number of simplifications or by neglecting the dependencies between some quantities; the model uncertainty can be quantified by experiments and measurements of the physical process
- Statistical uncertainty – model uncertainty and other uncertain variables are assessed (e.g. by expected value, standard deviation and distribution type) on the basis of tests and measurements; only a limited number of data will be available, this introduces another significant source of uncertainty

A probabilistic model consists of a random quantities, that have to defined by:

- The types of distribution
- The parameters as mean value and standard deviation
- Random variation in time
- Random variations in space.

The modelling of the time and space variability should preferably be written as the sum of independent parts. These parts can easily be incorporated in the limit state function. In the time domain this leads to:

$$X(t) = X_0 + S_{low}(t) + S_{hort}(t) \qquad (11)$$

where:
X_0 a time invariant part
S_{low} slowly varying parts, e.g. modelled as time variant independent blocks
S_{hort} short term varying parts

In the space domain this leads in a similar way to:

$$X(x,y) = X_o + X_{site} + X_{local}(x,y) \qquad (12)$$

where:
X_0 a space independent part
X_{site} a part depending on the country and / or building site
$X_{local}(x,y)$ a part reflecting the small scale local variability within an element

C.2.4.2 Probabilistic calculations

Once a service life model of the process to be described is present and a limit state has been established as well as the uncertainties of the variables have been determined, the failure probability has to be calculated. The two principle types of limit state formulations have been given by the relations (4) and (6). These are formulations of exactly the same limit state. For simplicity relationship (4) will be used to demonstrate the probabilistic calculation that is necessary to establish the failure probability. The limit state function can also be written as:

$$g(\underline{X}) = R(\underline{X}_1) - S(\underline{X}_2) \qquad (13)$$

where:

$g(\underline{X})$ multivariate limit state function
$\underline{X}$ vector with all basic variable $(X_1, X_2, X_3, \ldots, X_n)$
$R(\underline{X}_1)$ contribution form the capacity to the limit state function
$\underline{X}_1$ vector with basic variables related to the capacity
$S(\underline{X}_2)$ contribution from the actions (loads and environment) to the limit state function
$\underline{X}_2$ vector with basic variables related to the actions

The failure probability is now defined as:

$$P_f = P\{\, g(\underline{X}) < 0 \,\} = P\{\, R\text{-}S < 0 \,\} = \underset{g(\underline{X})<0}{\quad} RSdsdt \qquad (14)$$

A more general notation of this is:

$$P_f = \underset{g<0}{\quad} f_{\underline{x}}(\underline{\xi})d\underline{\xi} \qquad (15)$$

where $f_x(x)$ is the multivariate density function of $\underline{X}$.

Only a few exact, analytical solutions are known to calculate this failure probability. In practice only numerically approximations will be used. The Joint Committee on Structural Safety has defined four levels of probabilistic calculations for structural design. The levels can, with small modifications, also be applied for durability design of building elements.

- Level 0 stands for the safety margin approach: the safety margin originates from experience. The resulting safety is not in probabilistic terms and the probability of failure is unknown.
- Level I is also a safety margin approach, but the safety factors are coupled to the basic - stochastic - variables and determined in a probabilistic concept. Although the result is not in probabilistic terms, an estimate of the failure probability can be given dependent on the magnitude of the safety factors involved.
- Level II calculates failure probabilities, but approximates the failure boundary, the G-function, in a so-called design point. These approximations may over- or

underestimate the failure probability and the concept has no means to improve the results, or even to estimate the error involved.

- Level III finally, calculates the exact probability of failure, mostly numerically. The errors made are only due to numerical inaccuracies and can be influenced by the grid size. With infinite small grid size, exact results are obtained. At any grid size, an estimate of the error can be obtained.

As we are in this stage only interested in the calculation of probabilities, only level II and level III methods will be considered. First the Level III methods will be discussed. Theoretically (15) is solved numerically very easily. The integral is discretised as:

$$P(F) = \sum_{g(\xi)<0} \cdots \sum f_{\underline{x}}(\xi)\Delta\xi_1 \ldots \Delta\xi_n \qquad (16)$$

As the integral of (15) is generally called a Riemann integral, the numerical solution of this integral is often called the Riemann procedure. After discretising the probability density functions *f(R)* and *f(S)* (for every possible set of discrete values of the basic variables) the g-function is evaluated. If the result is less than zero, this particular combination of basic variables leads to failure and the associated probability density is calculated from f_x. To obtain the total probability involved with failure, the probability density is accumulated for all sets where $g(\underline{X}) < 0$.

For the level II approach several procedures are available. The main reason for this is the over- or underestimation of this approach. Depending on the type of limit state this can be reduced by applying the most appropriate procedure. The basic procedures are the FORM (First Order Reliability Method) and the SORM (Second Order Reliability Method).

The FORM procedure estimates the failure probability by:

- Transforming (if necessary) the basic variables to independent and normal distributed variables
- Linearising the limit state function g(X) = 0 in a special point, called the design point; in some procedures this in done in the mean point g(μ(X)); another possibility is the most likely failure point (highest probability density along the failure boundary g(X) = 0).

The SORM procedure is in basic similar to the FORM. The only difference is the approximation of the failure boundary with a quadratic function. This is an advantage in case the failure boundary has a large curvature.

It is more or less standard to present the main result of a probabilistic calculation as a reliability index β instead of the failure probability. The relationship between these two parameters has already been given:

$$P_f = \Phi(-\beta) \qquad (17)$$

In Appendix C.3.D a table is given for the standard normal distribution that can be used to find the failure probability in case β is known.

One of the main basic variables in durability problems is the time. This will sometimes cause a problem in finding the failure probability within the reference period. A possible solution for that kind of problems is to divide the reference period in small, quasi-static, intervals and calculate the failure probability for each interval. Summation of them gives the failure probabilities.

C.2.5 References

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C.3 Evaluation of service life distributions from field data on age distributions

C.3.1 Scope

In survey investigations of building stocks for example, data on age distributions of construction details are often collected. What interests the material scientist, though, is not the age distribution but rather the service life distribution. Presented herein will be three renewal process models describing the relationship between such an age distribution found and the corresponding service life distribution sought [1]. The last model is of a general form, encompassing the two former.

C.3.2 Introduction

Consider a certain part of an item (e.g. a building or any other construction), which will be repaired or replaced many times during the item's entire service life. Such parts, belonging to a stock of items, as defined by proper means, are characterised by a specific, but still unknown, service life distribution. Furthermore, suppose that at a certain time a sampling is taken of this stock. The parts of the items sampled constitute the set of objects to be analysed. Data on the age (the time since the previous maintenance, or, if this has not occurred, the time since the item was erected) of each object is collected. Obviously there is a correspondence between the age distribution of the objects found and the service life distribution being sought. In the following, the analytical relationship between these distributions will be given; its validity however will depend on the fulfilment of the following conditions:

- Steady-state condition, i.e. for a long period, compared to the mean service life of the objects, the number of items within the stock as well as the distributions of factors affecting the service life (loads, material qualities, maintenance methods and failure criteria) have been unaltered;
- Discarding of objects where maintenance has not been carried out, in spite of the performance of those objects surpassing any acceptable failure criterion; and
- If it is anticipated that the service life distributions differ between fresh and repaired parts, the objects have to be separated into two independent groups: one consisting of replaced parts or parts belonging to new items, and another consisting of repaired parts.

C.3.3 Models

C.3.3.1 Randomly-distributed homogeneous service life

Randomly-distributed homogeneous (RDH) service life means that the longitudinal service life distribution equals the transversal distribution, i.e. the distribution of consecutive service lives (maintenance intervals) for a specific object is the same as the service life distribution for all objects at an instant time. This is an idealised picture, one that is not typically found in the real world, since factors affecting the service life (loads, material qualities, maintenance methods and failure criteria) usually are object correlated (e.g. dependent on the local environment or on preferences by the owners/maintenance companies).

At any instant time, let $a(t)$ be the probability density function (PDF) of the stochastic variable T_A, representing the age of the objects, where the age is defined as the time since the previous maintenance. When $f(t)$ denotes the PDF of the stochastic variable T_L, representing the service life of the objects, then:

$$a(t) = \frac{1}{N}\int_t^\infty f(t')\mathrm{d}t' \qquad (1)$$

where $1/N$ is a normalising factor given by:

$$N = \int_0^\infty t' f(t')\mathrm{d}t' \qquad (2)$$

Further details can be found in [1, 2], for example.

C.3.3.2 Constant longitudinal randomly distributed transversal service life

Constant longitudinal - randomly-distributed transversal (CL - RDT) service life is the other extreme where the service life is a constant in time for a specific object, while there is a distribution in service life within the set of objects. This means that, for a specific object, the maintenance interval will always be the same, which may be even more unrealistic than the RDH model. Nevertheless, in order to set a limit, it is reasonable to state the expression for the age PDF in terms of the transversal service life PDF:

$$b(t) = \int_t^\infty \frac{f(t')}{t'}\mathrm{d}t' \qquad (3)$$

where $f(t)$ now denotes the transversal service life PDF of the objects and $b(t)$ represents the age PDF [1].

C.3.3.3 Mixed model

As neither the RDH nor the CL-RDT model are realistic, merely constitute the theoretical opposite bonds, a phenomenological model which interpolates between these extremes would seen appropriate:

$$c(t) = \frac{1}{N}\int_t^\infty \frac{f(t')}{t'^p}\mathrm{d}t' \qquad (4)$$

where $f(t)$ is the transversal service life PDF of the objects and $c(t)$ the age PDF. The exponent p, with $0 \le p \le 1$, can be regarded as a correlation parameter, where p=0 corresponds to no object correlation and p=1 to a total object correlation. $1/N$ is the normalising factor given this time by

$$N = \int_0^\infty \frac{f(t')}{t'^{p-1}} \mathrm{d}t' \qquad (5)$$

C.3.4 Numerical examples

In order to develop an idea how the mixed model affects the analysis, age PDFs for both the mixed (c(t)) and the RDH (a(t)) model have been simulated assuming transversal service life PDFs of the Weibull type [3]:

$$f(t) = \beta\alpha^{-\beta} t^{\beta-1} e^{-(t/\alpha)^{\beta}} \qquad (6)$$

The scale parameter α and the shape parameter β are related to the Weibull mean by:

$$E[T_{\mathrm{L}}] = \alpha\Gamma\left(1 + \beta^{-1}\right) \qquad (7)$$

and to the Weibull standard deviation by:

$$\sigma[T_{\mathrm{L}}] = \alpha\left\{\Gamma\left(1 + 2\beta^{-1}\right) - \Gamma^{2}\left(1 + \beta^{-1}\right)\right\}^{1/2} \qquad (8)$$

It turns out that for almost any choice of the correlation parameter p and Weibull parameters α and β when generating an age PDF $c(t)$, it is possible to generate an age PDF $a(t)$ with almost an identical shape (deviation at any point < 1 % in terms of $c(0)$) by the use of different Weibull parameters. However, for wide distributions when p is close to 1, the fit is somewhat worse. This finding can most likely be generalised to any realistic PDF.

Thus, without any additional information and data processing, p cannot be determined and the full advantage of the model cannot be realised. How to proceed, though, is not altogether clear. At the very last, a screening of p using a number of typical cases would be desirable and also possible, when extensive longitudinal and transversal data sets are available (most data sets sampled are just transversal). Subsequently, by comparison and qualitative reasoning these estimations of p could likely be generalised with fair confidence to other cases.

Nevertheless, in its present form, the model points out possible ranges of the mean life E and standard deviation σ in terms of the correlation parameter p, thus providing uncertainties in E and σ which until now have normally been overlooked. This can be accomplished by, initially, analysing an age PDF using the RDH model (p=0) and solving the integral equation for the transversal service life PDF, from which E and σ can be estimated. Then, it is possible to estimate the, in practice, maximum error made in E and σ by using the CL-RDT model (p=1) instead and taking the differences between the E values and between the σ values, respectively, of the two models.

An example of this is given in fig C.3:1, assuming transversal service life PDFs of the Weibull type. To the left, the maximum relative error of E is given as a function of σ/E as determined by the RHD model. In addition, as an example of a reasonable degree of correlation, the relative error made if p was actually 0.5 is given. To the right, the corresponding relative errors of σ are given. For a wide-ranging transversal service life PDF, the maximum errors in E and σ will, as expected, be considerable, although less pronounced for the latter. (The general parameters E and σ are voluntarily chosen as the basic units, instead of the Weibull specific α and β, since fig C.3:1 is, as a good approximation, likely to be valid for the vast majority of realistic service life PDFs.)

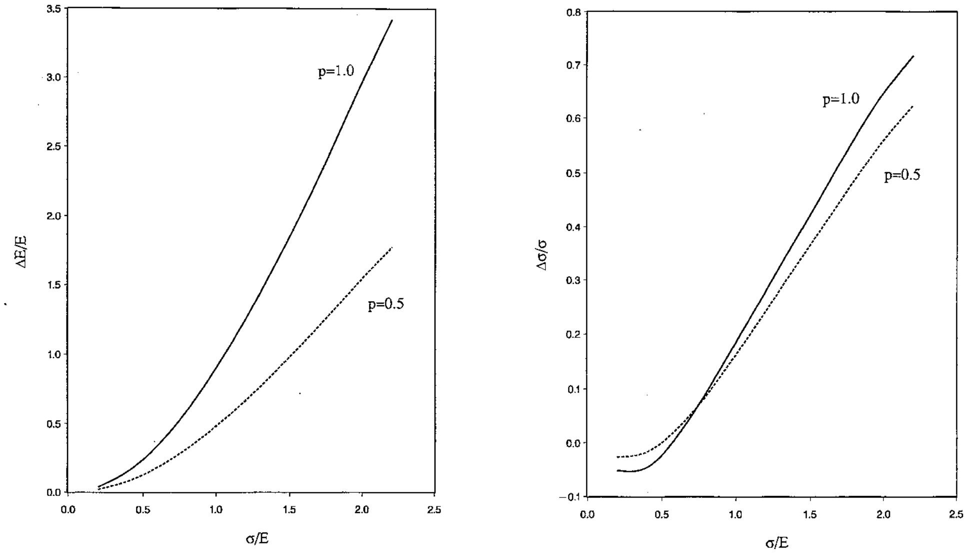

Fig C.3:1 The relative error of the mean E (left) and the standard deviation σ (right) as functions of the normalised standard deviation σ/E for p=1 (solid line) and p=0,5 (dashed line), respectively. (Weibull transversal service life PDFs assumed.)

To illustrate how the relative error in E and σ depends on p, one case is given in fig C.3:2. An age PDF was generated using the RDH model for a transversal service life PDF of the Weibull type with $\sigma/E=1$ (which corresponds to $\beta=1$ and α being arbitrary). Subsequently the age PDF obtained was analysed using the mixed model with various, fixed values of p and relaxed Weibull PDFs. As can be seen, the error in E is almost linearly dependent on p, whereas the error in σ levels out at $p\approx0{,}7$.

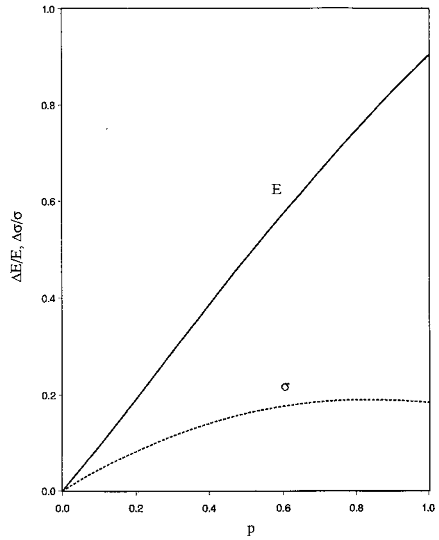

Fig C.3:2 The relative error of the mean E (solid line) and the standard deviation σ (dashed line), respectively, as functions of p for $\sigma/E=1$ (Weibull transversal service life PDFs assumed.)

An applied example is found in [4], where a survey investigation of the status and age of certain building components and materials are reported. The service lives of these components and materials were estimated [4] using the RDH model [5], assuming Weibull service life distributions. Subsequently, these empirical service life data have been processed further, and used as input in a model to calculate the cost of corrosion due to air pollution [6]. Table C.3:1 shows the original result (p=0) for two paint applications, and the results with the data re-analysed, using p=0,5 and p=1,0, respectively.

Table C.3:1 Estimated mean values and standard deviations (years) of Weibull service life distributions fitted to real material data, using various p values.

	p=0		*p*=0,5		*p*=1,0	
	E	*σ*	***E***	*σ*	***E***	*σ*
Paint on wooden windows	7,6	4,8	9,1	5,0	10,5	4,9
Site painting of zinc-coated sheet steel	7,0	1,7	7,2	1,7	7,4	1,6

As expected, the re-analysed mean values deviate considerably from the original one when the standard deviation is of the same order of magnitude as the mean (wide service life distribution), while the deviation is negligible when the standard deviation is relatively small (narrow service life distribution). As a comparison, the uncertainty of the estimated mean due to data scattering was about ±4 % and ±13 % for the two cases, respectively.

C.3.5 Conclusions

- From a theoretical point of view, the appropriate way to extract a service life PDF from an age PDF would be to use the mixed model described above.
- However, the mixed model yields ambiguous numerical solutions and the correlation parameter p, contained in the model, is not straightforwardly estimated by other means. Nevertheless, possible ranges for the mean and the standard deviation of the transversal service life PDF can readily be obtained by using the two extreme models (the RDH model with $p=0$ and the CL-RDT model with $p=1$).
- One way to improve the situation would be to screen p by analysing, if available, a number of typical cases with extensive longitudinal and transversal data sets. Subsequently, through comparison and qualitative reasoning these estimates of p could likely be generalised with a fair level of confidence to other cases.

C.3.6 References

1. Jernberg, P. (1996) Service life analysis from field data on age distributions, (Accepted for publication in *Materials and Structures*).
2. Cox, D.R. (1962) *Renewal theory*, Methuen, London; Wiley, New York.
3. Weibull, W. (1951) A statistical distribution function of wide applicability, *J. Appl. Mech.*, Vol. 18, pp. 293-7.
4. Tolstoy, N., Andersson, G., Sjöström, C. and Kucera, V. (1990) *External building materials - quantities and degradation*, Research report TN:19, National Swedish Institute for Building Research, Gävle Sweden.
5. Tolstoy, N. (private communication).
6. Kucera, V., Henriksen, J., Knotkova, D. and Sjöström, C. (1993) Model for calculations of corrosion cost caused by air pollution and its application in three cities, *Proceedings of the 10th European Corrosion Congress, Barcelona.*

Appendix C.3.A Terms and definitions

Accidental action	Action that is almost always absent (like collisions, explosions, fire or earth quakes)
Accumulated damage model	*Durability model* in which the resistance is decreasing due to accumulation of damage and a critical level for the accumulated damage can be defined
Action	Application of an influence on a structure or a structural component; actions can be *mechanical actions, environmental actions* or *accidental actions*
Agent	Substance or physical phenomena from the environment that causes the *environmental action*
Assessment	All activities to establish the condition of a building element
Calibration method	Method to ensure that an analysis on a lower level of accuracy will give in principle the same results as an accurate analysis
Capacity	Ability of a structure or structural component to withstand the influence of *actions*
Characteristic value	Percentile value of the *probability density function* of *random* variables
Density function	*Probability density function*
Design parameter	Parameters expressing the performance of a structure or structural component that have to be established during the design, for example dimension and material properties
Design value	Value to be used in a design calculation, where the magnitude is such that the aimed *reliability* will be reached
Deterioration	Reduction in the course of time of the building materials characteristics
Deterministic	An actual value of a quantity (variable or constant) which is certain
Distribution function	*Probability distribution function*
Durability	*Capacity* to withstand the influence of *actions* in the course of time
Durability modelling	Design based on equations that describe the influence of *deterioration processes* on the structural behaviour
Economical optimisation	*Optimisation* based on costs
Environmental action	Mechanical, biological, chemical or physical influence from the environment acting on a structure or structural component
Failure	Situation in which the structure has exceeded a *limit state*
First Order Reliability Method	*FORM procedure*

FORM procedure	Level II *Probabilistic calculation* based on a linearisation of the *limit state* function
Inspectability	Ability to inspect a structure or structural component, depending on factors such as accessibility and pre-warning
Levels of probabilistic calculations	*Probabilistic calculations* can be made on different levels with respect to the accuracy in calculating the probability of *failure*; level 0 is the conventional safety margin approach, level I is a safety margin approach that is connected to a *failure* probability; level II is based on approximations in calculating the *failure* probability an level III is the exact approach
Life	*Service life*
Life cycle	All phases during the *service life* of structure, starting form design and ending with demolition or reuse
Life span	*Service life*
Lifetime	*Service life*
Lifetime design	*Durability design* where the *limit state function* has been described as an explicit function of time
Life cycle costs	All costs during the whole *life cycle* of a structure
Limit state	Border that separates desired (by the owner) or acceptable (by the authorities) states form the undesired or adverse states of a building element; to describe the limit state one, or a set of *limit state functions* is needed
Limit state function	Mathematical function that describes a *limit state* or a part of it
Maintenance	Activities during the *service life / lifetime / life span* of a structure or structural component that are meant to maintain the *capacity* on the desired level
Mechanism	The way a building element reacts on the application of an *action*
Model	Mathematical formulation that gives a (simplified) description of the behaviour of a structure or structural component
Model uncertainty	Uncertainty related to the accuracy of the *model* used
Optimisation	Analysis to define the most favourable
Physical uncertainty	*Uncertainty* in the physical behaviour
Probability density function	Function that describes the probability that the value of a *random* quantity will fall in a certain interval: $f_X(x)\,dx = P(x<X<x+dx)$
Probability distribution function	Function that describes the probability of exceeding a certain value for a *random* quantity: $F_X(x) = P(X<x)$
Probabilistic calculation	Method to calculate the probability for a certain event
Random	*Statistic* or *stochastic*

Reduced resistance model	*Service period durability design*
Reference period	Design *lifetime* of a structure or structural component
Reliability	Probability that the building element will not *fail*
Reliability index	Measure for the *reliability*, commonly used in codes instead of the probability of *failure* *(see Appendix C.3.D)*
Reliability method	Method for establishing the *reliability* of a system such as a structure or a structural component
Reparability	Ability to repair a structure or a structural component, depending on factors such as accessibility and redundancy
Riemann procedure	Level III *probabilistic calculation* that calculates the failure probability by discretising the probability density function
Second Order Reliability Method	*SORM procedure*
Semi-probabilistic	Description on a *deterministic* level but based on *probabilistic* theories
Serviceability limit state	*Limit state* with respect the comfort of the user, functionality, aesthetics or cosmetics
Service life	The period of time after construction during which the *reliability* of the structure satisfies the specification
Service period durability design	*Durability design* in which the resistance is decreasing due to *deterioration*
SORM procedure	Level II *Probabilistic calculation* based on a quadratic approximation of the *limit state* function
Statistic	An actual value of an uncertain quantity which can only be described in terms of probability
Statistical uncertainty	*Uncertainty* due to a lack of statistical data
Stochastic	An value of an uncertain quantity that can only be predicted in terms of probability
Target reliability	Aimed level for the *reliability*
Time domain model	*Lifetime design*
Ultimate limit state	*Limit state* with respect to the loss of static equilibrium, like collapse, fracture, overturning, lifting, sliding
Uncertainty	An actual value of a quantity which can not be predicted

Appendix C.3.B Statistics

If due to uncertainty a numerical value cannot be predicted precisely, the variable X is called a random variable. The event that the variable X is smaller than some value x with a probability P is denoted $P\{X < x\}$. The probability $P\{X < x\}$ is called the probability distribution function.

$$F_x(x) = P(X<x) \qquad \text{(b1)}$$

As $F_x(x)$ is defined as probability it is always between 0 and 1. In fact, the function always starts at 0 for $x = -\infty$ and increases monotony to 1 for $x = +\infty$. If we differentiate $F_x(x)$ with respect to x we obtain the probability density function:

$$f_x(x) = dF_x(x)/dx \qquad \text{(b2)}$$

This function gives the probability that X is in the interval from x to $x+dx$:

$$P(x < X < x+dx) = f_x(x)\, dx \qquad \text{(b3)}$$

Appendix C.3.C gives an overview of the mathematical expressions, the shapes and the expressions for the mean μ and the standard deviation σ defined as:

$$\mu = \quad x f_x(x) dx \qquad \text{(b4)}$$

$$\sigma^2 = \quad (x-\mu_x)^2 f_x(x) dx \qquad \text{(b5)}$$

The mean is the quick characterisation for the order of magnitude of a quantity. The standard deviation is the quick characterisation of the scatter. As an alternative one often uses the coefficient of variation V:

$$V = \sigma/\mu \qquad \text{(b6)}$$

Of course, this measure is valid only if μ is a positive quantity.

Appendix C.3.C Probability Distributions [5]

Distribution type	Parameters	Moments
Rectangular $a \le x \le b$ $f_X(x) = \frac{1}{b-a}$	a b	$m = \frac{a+b}{2}$ $s = \frac{b-a}{\sqrt{12}}$
Normal $f_X(x) = \frac{1}{\sigma\sqrt{2\pi}} \exp\left(-\frac{1}{2}\left(\frac{x-\mu}{\sigma}\right)^2\right)$	μ $\sigma > 0$	$m = \mu$ $s = \sigma$
Lognormal $f_X(x) = \frac{1}{X\zeta\sqrt{2\pi}} \exp\left(-\frac{1}{2}\left(\frac{\ln x-\lambda}{\zeta}\right)^2\right)$	λ $\zeta > 0$	$m = \exp\left(\lambda + \frac{\zeta^2}{2}\right)$ $s = \exp\left(\lambda + \frac{\zeta^2}{2}\right)\sqrt{\exp(\zeta^2) - 1}$
Shifted Lognormal $x > \varepsilon$ $f_X(x) = \frac{1}{(x-\varepsilon)\zeta\sqrt{2\pi}} \exp\left(-\frac{1}{2}\left(\frac{\ln(x-\varepsilon)-\lambda}{\zeta}\right)^2\right)$	λ $\zeta > 0$ ε	$m = \varepsilon + \exp\left(\lambda + \frac{\zeta^2}{2}\right)$ $s = \exp\left(\lambda + \frac{\zeta^2}{2}\right)\sqrt{\exp(\zeta^2) - 1}$
Shifted Exponential $x \ge \varepsilon$ $f_X(x) = \lambda \exp(-\lambda(x-\varepsilon))$	ε $\lambda > 0$	$m = \varepsilon + \frac{1}{\lambda}$ $s = \frac{1}{\lambda}$
Gamma $x \ge 0$ $f_X(x) = \frac{b^p}{\Gamma(p)} \exp(-bx) x^{p-1}$	$p > 0$ $b > 0$	$m = \frac{p}{b}$ $s = \frac{\sqrt{p}}{b}$
Beta $a \le x \le b,\ r, t \ge 1$ $f_X(x) = \frac{(x-a)^{r-1}(b-x)^{t-1}}{(b-a)^{r+t-1} B(r,t)}$	a b $r > 1$ $t > 1$	$m = a + (b-a)\frac{r}{r+t}$ $s = \frac{b-a}{r+t}\sqrt{\frac{rt}{r+t+1}}$
Gumbel (L) $-\infty < x < +\infty$ $f_X(x) = \alpha \exp(-\alpha(x-u) - \exp(-\alpha(x-u)))$	u $\alpha > 0$	$m = u + \frac{0.577216}{\alpha}$ $s = \frac{\pi}{\alpha\sqrt{6}}$
Frechet (L) $\varepsilon \le x < +\infty, \quad u, k > 0$ $f_X(x) = \frac{k}{u-\varepsilon}\left(\frac{x-\varepsilon}{u-\varepsilon}\right)^{-k-1} \exp\left(-\left(\frac{x-\varepsilon}{u-\varepsilon}\right)^{-k}\right)$	$u > 0$ $k > 0$ ε	$m = \varepsilon + (u-\varepsilon)\Gamma\left(1 - \frac{1}{k}\right)$ $s = (u-\varepsilon)\sqrt{\Gamma\left(1-\frac{2}{k}\right) - \Gamma^2\left(1-\frac{1}{k}\right)}$
Weibull (S) $\varepsilon \le x < +\infty, \quad u, k > 0$ $f_X(x) = \frac{k}{u-\varepsilon}\left(\frac{x-\varepsilon}{u-\varepsilon}\right)^{-k-1} \exp\left(-\left(\frac{x-\varepsilon}{u-\varepsilon}\right)^{-k}\right)$	$u > 0$ $k > 0$ ε	$m = \varepsilon + (u-\varepsilon)\Gamma\left(1 + \frac{1}{k}\right)$ $s = (u-\varepsilon)\sqrt{\Gamma\left(1+\frac{2}{k}\right) - \Gamma^2\left(1+\frac{1}{k}\right)}$

Appendix C.3.D Reliability index of the standard normal distribution

Reliability index β	Probability
0.0	5.0E-01
0.2	4.2E-01
0.4	3.4E-01
0.6	2.7E-01
0.8	2.1E-01
1.0	1.6E-01
1.2	1.2E-01
1.4	8.1E-02
1.6	5.5E-02
1.8	3.6E-02
2.0	2.3E-02
2.2	1.4E-02
2.4	8.2E-03
2.6	4.7E-03
2.8	2.6E-03
3.0	1.3E-03
3.2	6.9E-04
3.4	3.4E-04
3.6	1.6E-04
3.8	7.2E-05
4.0	3.2E-05
4.2	1.3E-05
4.4	5.4E-06
4.6	2.1E-06
4.8	7.9E-07
5.0	2.9E-07

Joint CIB W080 / RILEM TC 140 – Prediction of Service Life of Building Materials and Components

Guide and Bibliography
to
Service Life and Durability Research for Buildings and Components

PART IV – Durability of Civil Engineering Structures - Annotated Bibliography

G.Y. Grondin, NRCC – National Research Council Canada

PART IV — FUNCTIONAL ASPECTS RELATED TO THE SERVICE LIFE OF BUILDING MATERIALS AND COMPONENTS[1]

G. Y. Grondin[2], NRCC – National Research Council Canada

A. SERVICE LIFE PREDICTION

1 Akoz, F. and Akman, M.S. (1990) Service life estimation for multi-ply flat roof membranes, (refer to section C).

2 Ammar, C. and Longuet, M. (1978) Belgian requirements about buildings service life, in *Proceedings of the 1st International Conference on Durability of Building Materials and Components, Washington DC 1978*, ASTM STP 691, American Society for Testing and Materials, pp. 77-90.

The results of a survey of expected and actual service life of buildings and their components are presented. The survey was made amongst the architects, contractors, owners, and other building professionals in Belgium. Although many building components were covered in the survey, only the interior door is covered in detail in this publication. Three figures presenting general results of the survey are presented. Those figures present expected service life from technicians (probably the designers) and the owners. The types of buildings included are dwellings, hospitals, office buildings, and schools. In general, it has been found that the service life required by technicians is higher than that required by the owners. The required service life for public buildings is lower than that required for housing buildings. Finally, a short table demonstrates that the observed service life for taps, interior doors, and elements of the curtain wall is lower than the required service life. The report presents interesting data. However, it would have been of greater interest to see more detailed results for other components of buildings, with less focus on interior doors. No references have been given in the report which make it difficult to obtain a more complete report of the survey.

3 Andrade, C., Alonso, C., Gonzalez, J.A. and Rodriguez, J. (1989) Remaining service life of corroding structures, (refer to section B).

4 Androic, B., Juric, S. and Dujmovic, D. (1989) Checking the reliability and durability for corrosion, in *Durability of Structures, IABSE Symposium, Lisbon*, 6-8 September, Vol. 57/2, pp. 859-64.

The authors discuss the decrease of the safety index which takes place with time under fatigue loading. As expected, the decrease in safety index is more accentuated when corrosion fatigue is considered. The reduction in fatigue strength due to the presence of corrosion is only assumed. Reductions from 0 to 50 % have been assumed. Curves of safety index vs. time are presented which can be used to estimate the required safety index for the new structure given the desired service life and the effect of corrosion in reducing the fatigue strength.

5 Binda, L. and Molina, C. (1990) Building materials durability: semi-Markov approach, (refer to section B).

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[1] Adpated from: Grondin, G.Y. Durability of Civil Engineering Structures : Annotated Bibliography, *Internal Report, Institute for Research in Construction, National Research Council Canada,* 646, pp. 78, July, 1993

[2] Currently, Associate Professor, Department of Civil and Environmental Engineering, University of Alberta, Edmonton, Canada

6 Borges, J.F. (1989) Some basic concepts in building and their relationship to durability, in *Durability of Structures, IABSE Symposium, Lisbon*, 6-8 September, Vol. 57/1, pp. 3-14.
The basic concepts of 1) essential requirements and performance criteria, 2) quality assurance and human error, 3) probabilistic reliability and safety differentiation, 3) liability and technical insurance are briefly introduced and their relationship to durability is discussed. It is concluded that, when applied to buildings, the essential requirements (safe, serviceable and durable) may be enlarged to become healthy, intelligent and friendly. It is also concluded that to achieve quality, and implicitly durability, standard quality assurance procedures should be followed. The theory of probabilistic reliability may be extended to cover deterioration models also. The differentiation between theoretical and effective probabilities of failure is that the effective probability reflects human error. Finally, the author concludes that the present tendency to resort more and more frequently to liability claims based on negligence should be denounced.

7 British Standard Institution (1992) *Guide to Durability of Buildings, and Building Elements, Products and Components*, (refer to section C).

8 Browne, R.D. (1982) Design prediction of the life for reinforced concrete in marine and other chloride environments, (refer to section C).

9 Clifton, J.R. (1991) Methods for predicting the service life of concrete, in *Proceedings of the 5th International Conference on Durability of Building Materials and Components, Brighton, 1990*, (eds. J.M: Baker, P.J. Nixon, A.J. Majumdar and H. Davies), E & FN Spon, U.K., 7-9 November, pp. 361-73.
The author discusses methods for predicting the service lives of new concrete. The methods discussed are: 1) estimates based on experience (does not form a reliable basis for service life prediction); 2) deductions from performance of similar materials (comparisons between the durability of old and new concrete is not straightforward), 3) estimates based on the results of accelerated testing (an important requirement for using accelerated testing is that the degradation mechanism in the accelerated test should be the same as that responsible for the in-service deterioration), 4) applications of reliability and stochastic concepts (service life models using stochastic methods are based on the premise that service life cannot be precisely predicted), and 5) mathematical and simulation modelling based on the chemistry and physics of degradation processes. This is a good paper which provides a clear picture of the present state of the art in the area of service life prediction.

10 Fagerlund, G. (1985) Essential data for service life prediction, in *Problems in Service Life Prediction of Building and Construction Materials*, (ed. L.W. Masters), Martinus Nijhoff Publishers, Dordrecht, pp. 113-38.
The author shows that durability based upon old experience-codes of good practice is a dangerous method that can lead to a lot of durability problems, especially when dealing with new materials or proven materials in new applications. It would be preferable if the qualitative concept ”durability ” was abandoned in favour of the quantitative concept ”service life ”. The use of the service life concept in design means that the traditional accelerated test methods must be abandoned in favour of non-accelerated tests. The problems with accelerated tests are: it is almost always impossible to translate the exposure time in the test to a real exposure time in the real environment; the acceleration sometimes changes the destruction mechanisms and, consequently, the test results will bear no resemblance to the real behaviour; the acceleration is sometimes so large that destruction occurs although it would not occur in practice; the synergistic effect often observed between two or more destructive actions cannot be revealed when only one destruction mechanism is studied. The service life prediction should be based on stochastic analysis of the factors influencing the destruction mechanisms. The effect of uncertain material or environmental data on the service life can always be analysed by a sensitivity analysis. The author presents a list of information necessary for a service life prediction.

11 Frohnsdorff, G. and Masters, L. (1991) Suggestions for a logically-consistent structure for service life prediction standards, in *Proceedings of the 5th International Conference on Durability of Building Materials and Components, Brighton, 1990,* (eds. J.M: Baker, P.J. Nixon, A.J. Majumdar and H. Davies), E & FN Spon, U.K., 7-9 November, pp. 113-26.

The methodology for service life prediction, ASTM E-632 and RILEM Recommendation No. 64, are too general to give detailed guidance on their application to individual items. The author proposes a hierarchy of codes, with the more general codes at the top, to give more specific guidance for the evaluation of the service life for different materials in various environments. Standards should be provided for the characterisation of the environment, for the characterisation of the materials, components, and systems, for the determination of the mechanisms and kinetics of degradation, for mathematical models of degradation, and for service life prediction. The author also presents a general methodology for service life prediction applicable to any item.

12 Frohnsdorff, G. and Masters, L.W. (1978) The meaning of durability and durability prediction, in *Proceedings of the 1st International Conference on Durability of Building Materials and Components, Washington DC 1978*, ASTM STP 691, ASTM, pp. 17-30.

The authors discuss the meaning of the term durability. They point that durability is not an absolute quality of an item but a term expressing a human perception of a quality which changes with the environment. It implies likelihood of lasting well in expected environmental exposures. The ASTM definition of durability is presented. The definition incorporates the concept of design requirements being met or exceeded for a specific period of time such as the design service life. The authors discuss some of the problems of service life prediction from standard durability tests. A more detailed discussion of the ASTM recommended practice for development of accelerated tests for the prediction of service life is presented. Two practical examples (coating for steel protection and concrete exposed to a sulphate environment) of the use of the recommended practice are presented. The examples outline the procedure to use to design and interpret accelerated test results used for service life prediction. It is emphasised that, in order to predict the service life, one must define the nature of the material, characterise the nature of the environment, and have a good knowledge of the deterioration mechanism. The authors recommended that reliability concepts be used in service life prediction.

13 Haagenrud, S.E. (1985) Mathematical modelling of atmospheric corrosion and environmental factors, (refer to section B).

14 Haagenrud, S.E., Kucera, V. and Gullman, J.(1982) Atmospheric corrosion testing by electrolytic cells in Norway and Sweden, in *Atmospheric Corrosion*, (ed. W.H. Ailor), John Wiley and Sons, New York, pp. 669-94.

The electrochemical technique is based on measurement and integration of the current in electrolytic cells with an external imposed DC voltage. The use in several applications is described, i.e., the influence of environmental parameters on the cell current and time-of-wetness, comparative studies of corrosion properties of different alloys, and analysis of the corrosion environment in an industrial plant. The possibilities and limitations of the technique are discussed.

15 Harrison, W.H. and Gaze, M.E. (1989) Laboratory-scale tests on building mortars for durability and related properties, (refer to section B).

16 Hearn, G. and Testa, R.B. (1993) Resonance monitoring of building assemblies for durability tests, *Journal of Testing and Evaluation JTEVA,* Vol. 21, No. 4, July, pp. 285-95.

Changes in fundamental frequencies as a result of deterioration form the basis of resonance techniques. The authors present a method which is a variance of existing techniques used for monitoring of deterioration due to freeze-thaw testing. The latter technique monitors a single resonant frequency to detect deterioration. The technique presented in this paper monitors several modal frequencies and is used to detect both the occurrence and location of deterioration. The method is used to monitor fatigue damage in a welded steel frame. Some details of the test performed on the simple steel frame are presented. However, the authors do not discuss the sensitivity of the method to the size of the fatigue crack. A change in natural frequency of less than 10 % was observed between the time a crack was first detected and the time the test had to be stopped. The example given seems to indicate that the method is rather insensitive to the size of the crack. The authors discuss the potential application of the method for durability testing of building assemblies. Testing of building assemblies is deemed essential because of the need to distinguish between failure related to material durability and failures related to stress concentration or localised effects.

17 Hergenroeder, M., Schall, G. and Rackwitz, R. (1989) Reliability and risk function for deteriorated structures, in *Durability of Structures, IABSE Symposium, Lisbon*, 6-8 September, Vol. 57, pp. 511-6.

Some statistical models of degradation process are reviewed. Some concepts for updating reliability characteristics by using inspection results are also very briefly reviewed. The use of those concepts for the reliability analysis of carbonation of concrete and fatigue crack propagation is briefly illustrated. The paper lacks details to make the liaison between the concepts and the applications. A list of nine references where more detailed explanation may be found is provided.

18 Hookham, C.J. (1992) Service life prediction of concrete structures – case histories and research needs, (refer to section D).

19 Ishizuka, Y. (1983) The degradation and prediction of service life of building components, *Durability of Building Materials*, Vol. 1, No. 4, pp. 345-52.

The Government Building Department of the Ministry of Construction in Japan conducts a field survey of the state of concrete and steel buildings every five years. The survey is intended to provide a ”degradation index ” of structure, building components and materials, individually. The service life prediction of buildings is based on a statistical analysis of the buildings of the same age across the country. The values of the degradation indices for the same structural components of the same age over the country was found to follow a normal distribution. The statistical method developed for the prediction of service life accounts for the difference due to the conditions of climate and site.

20 Klaver, E. and Mijnsbergen, J. (1993) *Modelling the Durability of Brickwork Masonry: the Concept of the Relative Performance*, (refer to section C).

21 Legget, R.F. and Hutcheon, N.B. (1958) The durability of buildings, in *Symposium on Some Approaches to Durability in Structures*, ASTM STP No. 236, American Society for Testing and Materials, pp. 35-44.

A general discussion of the durability of buildings, its definition and implication on the design, is presented. ”There is no such thing as the durability of a building, the durability of the major components of a building being inevitably different ”. The term ”service life” should be used instead of the vague and indefinite word ”durability”.

22 Lipfert, F.W. (1989) Atmospheric damage to calcareous stones: comparison and reconciliation of recent experimental findings, (refer to section B).

23 Lipfert, F.W. (1987) Effects of acidic deposition on the atmospheric deterioration of materials, (refer to section B).

24 Lipfert, F.W., Benarie, M. and Daum, M.L. (1986) Metallic corrosion damage functions for use in environmental assessments, (refer to section E).

25 Lucchini, A. (1990) Models for the evaluation of the service life of building components, in *Proceedings of the 5th International Conference on Durability of Building Materials and Components, Brighton, 1990*, (eds. J.M: Baker, P.J. Nixon, A.J. Majumdar and H. Davies), E & FN Spon, U.K., 7-9 November, pp. 615-24.

The author presents a proposed logistic for the evaluation of the service life of a structure. The procedure for the evaluation of the service life proposed by the author can be summarised as follows: 1) define the environmental characteristics and the various materials in the component of the structure considered; 2) define the performance specifications and the functions of the material relevant to the system; 3) for each material obtain the models describing the response of the material to the environments. After this stage experimental investigation may be necessary to define the analytical models necessary to predict the material response; 4) the analytical evaluation of each system (materials/environment) and identification of the critical function (material property) – critical material; 5) service life prediction.

26 Lucchini, A. (1990) An approach to design for durability of the building technological system, in *Proceedings of the 5th International Conference on Durability of Building Materials and Components, Brighton, 1990*, (eds. J.M: Baker, P.J. Nixon, A.J. Majumdar and H. Davies), E & FN Spon, U.K., 7-9 November, pp. 477-83.

The paper introduces an approach to design for durability of the building technological system based on the theory of disturbance factors. The disturbance factor is defined as the factor triggering anomalies in the functioning (reflected by the structure's behaviour at time t=0) or evolution of the over time behaviour of a building component leading to shortening of the service life.

27 Masters, L.W. and Brandt, E. (1989) Systematic methodology for service life prediction of building materials and components, *Materials and Structures / Matériaux et Constructions*, Vol. 22, No. 131, pp. 385-92.

The paper outlines a systematic approach for service life prediction, it identifies the information needed to assess the service life. The paper first defines important terms applicable to durability in buildings. Terms such as durability, performance over time, degradation factor, service life are included in the list of 25 terms. A flow chart describing the procedure to adopt for service prediction is presented and discussed. The main stages in the process are: problem definition where users needs are identified and the performance criteria are defined; the preparation where the degradation factors and mechanisms and the consequences of degradation are identified; pre-testing to demonstrate that the deterioration mechanisms can be reliably accelerated; the testing phase where accelerated tests are conducted and compared with long-term testing; the interpretation stage where the test results are used to obtain the acceleration factor from the test results and the service life is predicted based on the accelerated test results; preparation of the test report. This is the procedure adopted by RILEM and CIB for service life prediction. The paper does not provide any example of an actual assessment of the service life for a typical building component.

28 Morinaga, S. (1990) Prediction of service lives of reinforced concrete building based on the corrosion rate of reinforcing steel, (refer to section C).

29 Philipose, K.E., Feldman, R.F. and Beaudoin, J.J. (1991) Durability predictions from rate of diffusion testing of normal portland cement, fly ash, and slag concrete, (refer to section B).

30 Pihlajavaara, S.E. (1984) The prediction of service life with the aid of multiple testing, reference materials, experience data, and value analysis, in *Proceedings of the 3rd International Conference on Durability of Building Materials and Components*, *1984 Espoo, Finland*, 12-15 August, Vol. 1, pp. 37-64.

The author presents a strategy to evaluate experimentally the durability of new materials. When a material is tested under accelerated condition, it is important to test a reference material in order to be able to correlate the accelerated test conditions with the actual environmental conditions. Before engaging in a long-term test programme, preliminary tests should be designed to eliminate the materials which do not have potential to be durable under long term conditions. The prediction of service life from the analysis of the values obtained from accelerated tests on the new material and the reference material, and correlation of the real environment with the accelerated test environment is discussed. The service life must also be defined statistically or in terms of the permissible amount of inferior material quantity or specimens.

31 Raharinaivo, A., Brevet, P., Grimaldi, G. and Pannier, G. (1986) Relationships between concrete deterioration and reinforcing steel corrosion, (refer to section B).

32 Rostásy, F.S. and Bunte, D. (1989) Evaluation of on-site conditions and durability of concrete panels exposed to weather, (refer to section C and Assemblies).

33 Sentler, L. (1987) Service life predictions of concrete structures, *Durability of Building Materials*, Vol. 5, No. 1, pp. 81-98.
The author discusses the use of stochastic methods to predict the service life of structures. The change in material properties such as the permeability of concrete is described as a function of loads and time. The stochastic model for the effect of load on the permeability of concrete evolved from creep tests whereby the strain in concrete changes with time. The effect of load level and environment on the deterioration rate of concrete and steel is discussed. The combined effect of load and environmental agents is also presented. In order to relate the loads and environment to the deterioration of a structure the relation between loads and load effects and between environment and micro-climate must be considered.

34 Shirayama, K. (1985) Research activities and administrative measures on durability of buildings – the state of the art in Japan, *Materials and Structures / Matériaux et Constructions*, Vol. 8, No. 105, May-June, pp. 215-21.
The author presents a summary of the research work performed in Japan on the durability of structures over the past century. A significant increase in the number of papers on durability and service life has been observed in the Japanese literature since 1965. In 1979 a committee was established to systematise the concept of durability. The first task of this committee was to define the terminology related to the field of durability. A sub-committee then proceeded to prepare principles for the process of planning for durability. Work was carried out between 1973 and 1978 to develop a procedure for predicting the service life of dwellings and their components. An extensive research project initiated in 1980 is dealing with the development of techniques for improving service life of buildings. The concept of service life being the important factor, research has been conducted to develop systems for more flexible buildings (the change in building use must be anticipated at the design stage). The author mentions two important projects launched with the purpose of providing housing with such flexibility. Other projects initiated in the '80's are dealing with improvement of durability of materials and buildings, and development of non-destructive testing techniques. Nation wide efforts are also directed at administrative measures to improve durability. One such example is long term guarantee system.

35 Siemes, A.J.M. and Vrouwenvelder, A.C.W.M. (1985) Durability – a probabilistic approach, *Durability of Building Materials*, Vol. 3, No. 1, pp. 101-13.
The scatter in observed service life of building structures is very high. The authors present an investigation of the use of reliability analysis to aid optimisation of design for durability. Essentially the same techniques are used as those which were proven to be successful in design for ultimate and serviceability limit states without deterioration effects. The results of a survey of carbonation effects on outdoor balcony slab (concrete cover, maximum-mean depth of carbonation, etc...) are presented in the form of a mean value and a coefficient of variation. This shows how can various alternative constructions can be compared in terms of expected capitalised costs. From a sensitivity analysis the variables that are mainly responsible for the scatter in the service life can be identified. The technique of service life prediction and capitalised cost evaluation seems be a promising tool for the decision making in the design of new structures, repairs and renovations.

36 Siemes, A.J.M., Vrouwenvelder, A.C.W.M. and van den Beukel, A. (1985) Durability of buildings: a reliability analysis, *Heron*, Vol. 30, No. 3, pp. 1-48.
This paper is a more detailed version of the above paper by Siemes and Vrouwenvelder (1985). Worked out examples of service life calculation and life cycle cost analysis illustrate the practical use of reliability analysis in durability assessment.

37 Siemes, A.J.M., Vrouwenvelder, A.C.W.M. and van den Beukel, A. (1985) Stochastic modeling of building materials performance in durability, in *Problems in Service Life Prediction of Building and Construction Materials* (ed. L.W. Masters), Martinus Nijhoff Publishers, pp. 253-63.

This is the same material as presented in the above two papers. No new information is presented.

38 Singh, G. (1991) The Promise and challenge of evaluation of service life, in *Proceedings of ACI International Conference on Evaluation and Rehabilitation of Concrete Structures and Innovations in Design, Hong Kong*, (ed. V.M. Malhotra), ACI SP-128, Vol. II, pp. 1423-37.

A general introduction is followed by justification of the need for and benefits of evaluation of service life of structures. A probabilistic simulation (using Monte Carlo technique) approach implemented in a computer programme is presented to evaluate the service life distribution of a structure once the deterioration model is established and the statistical distribution of the variables involved is known. A sensitivity analysis is also performed to determine which of the model parameter has a greater influence on the service life. Models for life-cost analysis are also presented. The methodology presented calls for research and developments towards understanding of the fundamental time-dependent mechanisms of degradation. In the author's view other methods such as estimates based on past experience, estimates through comparison, extrapolation of accelerated testing and statistical analysis of past data without consideration of mechanisms, cannot yield reliable estimates on their own.

39 Sjöström, C. (1987) A view on building materials durability research: activities at the National Swedish Institute for Building Research, in *Proc. of the 1st Intl. Conf. on Durability of Construction Materials, RILEM, Versailles, France*, 7-11 Sept., pp. 915-8.

The major part of the research at the Materials and Structures Division is concerned with a long-term performance under in-use conditions of building materials and components. The central theme in research on the durability of building materials is to create knowledge and methods for reliable predictions of service life. An important element that is often lacking in service life studies is feedback of the performance of materials in service. The Institute has developed methods for statistical surveys of the technical properties of the building stock. Projects on the long-term performance of loose-fill thermal insulation are carried with special attention given to the settlement of loose-fill materials. Some studies have looked at the material properties changes with time compared to those values assumed at the design stage. The author mentions that the development of measurement techniques and measurements and descriptions for in-use environments ought to be important research areas within building materials durability research.

40 Sjöström, C. (1985) Overview of methodologies for prediction of service life, in *Problems in Service Life Prediction of Building and Construction Material*, (ed. L.W. Master), Martinus Nijhoff Publishers, pp. 3-20.

The author presents an overview of some proposed or used methodologies for approaching research on the service life of building materials and components. To provide a more general approach to durability testing an ASTM committee has prepared a standard for developing accelerated tests to aid prediction of service life (ASTM E632). This standard acted as a model for other methodologies such as the ones used by the Centre Scientifique et Technique du Batiment (CSTB) and the National Swedish Testing Institute. Other methodologies have been proposed by RILEM TC 60-CSC, the Australian Standard 1745 and, CIB working commission W60. A common aspect of the various approaches is the importance of identifying the degradation mechanisms and the need for comparing the results from predictive service life tests or artificial ageing with the findings from long term tests under service conditions. A generic methodology has been proposed in the draft report of the CIB W80 / RILEM TC 71-PSL Committee. General research needs are outlined. Among the most urgent are: the gathering of in-service performance data through field inspection of buildings; development of procedures for feedback of the performance of materials under in-use conditions; development of methods for inspecting the state of existing buildings to assess the remaining service life; work to increase the knowledge of micro environment and of the interdependence of micro, meso and macro climates; development of mathematical models for the observed degradation.

41 Sneck, T. (1981) RILEM and durability, *Matériaux et Construction / Materials and Structures*, Vol. 14, No. 83, pp. 379-90.

The author presents a summary of the discussion that took place during the September 22-23, 1980 RILEM meeting. In the report the concept of durability is elaborated in general terms and the contributions and the discussion at the meeting are reviewed. The report is divided into seven sections. After a brief introduction the author presents a general background in which performance aspects are discussed as well as the concept of durability and definitions of terms are given. The close relation between durability and performance over time is discussed. The third section explains why there is an interest in durability and discusses economic implications. A need to disseminate the information presently available is recognised. The fourth section discusses the existing techniques to evaluate the performance of materials, components, and structures. The need for feedback of the performance of materials in service is stressed. Although data collected from field failures are important, there is a need to develop methods for inspecting the state of existing buildings showing no sign of distress. The fifth section gives a description of the past and present RILEM activities. Section six identifies areas where work is needed. Those are the performance over time of exterior and interior building components, elements, bearing structures, and materials for heat and moisture insulation. The author points the need for better methods of field data collection, the characterisation of in-use environment and relations between short term test results and field performance. Finally, section seven gives a summary. All the RILEM committees are asked to be aware of the importance of durability problems and try to integrate them in their studies.

42 Somerville, G. (1992) Service life prediction – an overview, *Concrete International*, Vol. 14, No. 11, pp. 45-9.

The author presents an overview of service life prediction of structures. Looking at past experience, the author states that aggressive actions (chlorides, sulphates) are the loads to use in the development of a viable design method for service life based on degradation models. The micro-climate is identified as the critical factor in degradation of materials. A short discussion of what should be a satisfactory in-service performance is presented. Performance should include function in use, financial consideration and technical provisions. A discussion of current design methods for service life (concrete cover, crack control) relying on concrete technology approach, is presented. In addressing the future developments, the author states that the greatest concern should be with corrosion, and the most pressing needs are to bring together all current developments concerned with durability and to convert the available information into design format. ”If we recognise the transportation mechanisms created by water, wind, and temperature, then by good design and a combination of architectural and structural detailing, we can prevent or significantly reduce the risk of aggressive actions reaching critical areas.”

43 Soronis, G. (1992) The problem of durability, *Building Design. Construction & Building Materials*, Vol. 6, No. 4, pp. 205-11.

The paper is said to be a state of the art report in the area of design for durability. It is mostly based on the proceedings of the International Conferences on the Durability of Building Materials and Components held between 1978 and 1990. The paper emphasises the need of a technology and knowledge transfer between the research scientists and designers. The volume of material published on design for durability is too overwhelming for designers to take the time to study the available material. Definitions are given for terms related to the design for durability. Some of the defined terms are durability, deterioration, performance over time, design life, economic life, macro, meso and micro climate. Factors which need to be accounted for in order to achieve an optimum design are discussed (those are design and functions, design and environment, design and materials, and design and economy). The most urgent research needs are listed as: to develop an internationally accepted, systematic, functional and performance-oriented methodology to facilitate the design process; to develop computerised systems aimed at giving designers decision making information; to develop practical guidelines for authors of scientific publications so that they might interpret their scientific results in a way which more closely meets the needs of building designers. The author presents guidelines to help authors of scientific papers to convey information for durability to building designers.

44 Spence, J.W. and Haynies, F.H. (1988) *Theoretical Damage Function for the Effects of acid Deposition in Galvanised Steel Structures*, (refer to section B).

45 Theophilus, J.P. and Bailey, M. (1984) The significance of carbonation tests and chloride level determination in assessing the durability of reinforced concrete, in *Proceedings of the 3rd International Conference on Durability of Building Materials and Components, 1984 Espoo, Finland*, 12-15 August, Vol. 3, pp. 209-38.

The significance of carbonation and chloride concentration in concrete is established by a brief discussion of the mechanism of corrosion of metals embedded in concrete. The authors outline various methods of determining the depth of carbonation in concrete. Of all the techniques reviewed the use of acid/base indicator phenolphthalein is found to be the most practical and was found to be accurate when compared to other methods. Various methods of determining the chloride content in concrete are enumerated. The total chlorides within the concrete can be evaluated by titration of a nitric acid extract or other rapid chemical techniques more suitable for field use, x-ray fluorescence. Since chloride content is usually specified as a percentage of cement content, methods of determining the cement content are also discussed. From a study of the accuracy of the various techniques, it seems that the chemical titration technique is preferred.

46 Thielen, G. (1983) Implementation of durability related specifications in technical guidance documents, in *Durability of Concrete Structures*, CEB – RILEM International Workshop, (ed. S. Rostam), 18-20 May, Copenhagen, pp. 327-39.

The paper tries to set-up a conceptual frame and to discuss technical details concerning the implementation of information on durability in technical guidance documents such as codes and recommendations guiding design, construction and repair of concrete structures. The author introduces the concept of structural performance. The performance concept necessitates to classify the information in four categories: requirements (safety, serviceability, structural appearance); criteria upon which the requirements are to be satisfied (safety criteria could be stability, strength, fatigue resistance, ductility, etc..); assessment and practice where all information must be compiled which is necessary to achieve in practice the required level of durability. The author recognises that experience still forms an important basis for engineering decisions concerning durability, although the use of new materials or existing materials in new environments will limit the value of experience. It is believed that analytical models and calculation methods will become more important in the evaluation of durability.

47 Tuutti, K. (1980) Service life of structures with regard to corrosion of embedded steel, (refer to section B).

48 Van Court, D.P. (1978) The owner / tenants interest in unifying durability data, in *Proceedings of the 1st International Conference on Durability of Building Materials and Components, Washington DC 1978*, ASTM STP 691, pp. 71-6.

The author advocates the development and the use of unified tests for different building materials. Presently, different tests are used for different building materials, making it difficult to compare various materials, especially when new materials are involved. Unified testing techniques for different materials would enable the owner / tenants and the designers to select the most efficient material for any given job. The author does not make any recommendation on the procedure to adopt to develop such unified test. (Note: although desirable from an economic and environmental point of view, the development of unified tests is very complex because different materials have different deterioration processes affected by different factors.)

B Materials

1 ACI Committee 201 (1985) *Guide to Durable Concrete*, 201.2R-77 (Reaffirmed 1982), ACI Manual of Concrete Practice, American Concrete Institute, Detroit.

This guide is an update of a previous committee report "Durability of Concrete in Service" which appeared in a 1962 ACI Journal. Separate chapters are devoted to each of the main types of concrete deterioration, namely, freezing and thawing, aggressive chemical exposure, abrasion, corrosion of materials embedded in concrete, and chemical reaction of aggregates. Recommendations are given for conditions where corrosion may be a problem. Among those recommendations we find: use of low permeability concrete, use of adequate steel cover, insure good drainage, limit chlorides in the concrete mix, use of positive protection systems.

2 Akers, D.J. (1990) Evaluation of reinforced concrete masonry in a highly corrosive environment, (refer to section D).

3 Allan, J.A. (1992) Retrofit ties for brick veneer, *Masonry*, Vol. 31, No. 5, September-October pp. 17-8.

Typically, distressed masonry walls possess corroded wall ties, ineffective wall ties and/or inadequately spaced wall ties. In order to retrofit masonry walls with deficient ties, a number of repair ties are available. The author briefly presents three different ties which can be used for brick veneer with a masonry wythe backup system and one tie for use with a metal stud backup system. The mechanical rather than the material characteristics of the ties are discussed. The author concludes by saying that most masonry walls showing signs of distress are the result of poor design, poor workmanship and/or a lack of attention to details during original construction.

4 Amoroso, G.G. and Fassina, V. (1983) *Stone Decay and Conservation – Atmospheric Pollution, Cleaning, Consolidation and Protection*, Materials Science Monographs, No. 11, Elsevier, New York.

This is a very useful book giving a comprehensive presentation of the problem of stone deterioration. The first chapter looks at the deterioration of stone by the action of water (rising damp, crystallisation of salts, and frost action in the presence of water). The following five chapters look at the environment and environment agents responsible for the deterioration of stone. Air pollutants and in particular sulphur dioxide and aerosols are discussed in some detail. A description of the atmosphere and the main sources of pollutants., the physical and chemical properties of aerosols, the nature and effect of wet and dry deposition, the effect of climatic factors on the dispersion of pollutants, and the oxidation of sulphur dioxide are some of the topics addressed in these chapters. Helpful guidance is given with practical examples for the diagnosis of stone decay and the cleaning and repair of stone. Various methods of cleaning and their area of application are presented. Also, various methods and materials for the repair of stone buildings and statues are presented. Finally, advice on different ways to protect stone from the harmful effect of atmospheric pollutants is given.

5 Andrade, C., Alonso, C., Gonzalez, J.A. and Rodriguez, J. (1989) Remaining service life of corroding structures, in *Durability of Structures, IABSE Symposium, Lisbon*, 6-8 September, Vol. 57/1, pp. 359-64.

The authors present a theoretical study of the loss of strength of reinforced concrete members due to rebar corrosion. The bending, shear, and axial strengths are investigated. The theoretical model is based on a constant rate of rebar corrosion. It is assumed that the corrosion will proceed without damage to the concrete cover, without loss in bond strength, and no loss in steel mechanical properties. No pitting or localised corrosion effects are studied.

6 Arliguie, G. and Grandet, J. (1987) Influence de la corrosion atmosphérique des armatures d'acier galvanisé sur leur comportement dans le béton, in *Proceedings of the 1st International Conference on Durability of Construction Materials, RILEM, Versailles, France*, 7-11 September, pp. 998-1004.
The zinc on galvanised rebars retards the hydration process of Portland cement. The interaction of the zinc with the cement paste results in the formation of a layer of cement paste around the rebar different from the bulk mass. The authors found that prior atmospheric oxidation of the galvanised rebars can minimise the hydration retardation observed with non-corroded rebars. Details of the reaction between the zinc layer (for corroded and non-corroded samples) and the cement paste are given.

7 Ashton, H.E. (1970) *Irradiation Effects on Organic Materials*, Canadian Building Digest 121, National Research Council of Canada, Ottawa, January.
The author gives a description of the effect of sunlight irradiation on organic materials such as plastics and wood. Although most ultraviolet light emitted by the sun is absorbed by the ozone in the upper atmosphere, the long wavelength UV finds its way to the ground. Since this part of the UV spectrum still has enough energy to break molecular bonds in the long molecule chains of polymers and wood, UV absorption will cause changes to the structure and properties of organic materials. To break the long chains, however, the UV has to be absorbed by the material. Some polymers such as acrylics are transparent to UV and therefore will have good resistance to UV. Others, however, absorb UV or the impurities in the material absorb UV and reverse polymerisation taking place (scission of the long polymer chains) result in cross-linking, which in turn results in increase hardness and loss of ductility. In other instances, changes only affect the absorption characteristics of the polymer and yellowing or discoloration can take place. In some cases initial irradiation products are coloured and absorb subsequent UV light, thus preventing deeper penetration. Since only the outer layer is affected, cracking will be restricted to the surface, thus giving rise to crazing.

8 Atteraas, L. and Haagenrud, S. (1982) Atmospheric corrosion in Norway, in *Atmospheric Corrosion*, (ed. W.H. Ailor), John Wiley and Sons, New York, pp. 873-91.
Since 1970 a number of programmes for field exposure of metals have been initiated in Norway at a number of locations. Bare and surface coated specimens have been studied. In addition to the standard flat plate specimens, specimens of various shapes and wire on bolt specimens have been used. In conjunction with the field tests, laboratory test programmes have been conducted for the investigation of single factors, short-time test programme development, and instrument development. The test results obtained from atmospheric exposure test sites are described. It was also found that the corrosion rate of weathering steel was more or less constant after four years of exposure. It was found that short term exposure test results can be very misleading if long-term corrosion must be considered. Correlation of corrosion rate with environmental factors such as duration of precipitation, concentration of SO_2, concentration of strong acid in precipitation and chloride concentration was found to exist. Various regression equations are presented for steel and zinc. Other test programmes aimed at studying metal siding, aluminium alloys, and coated steels are also described.

9 Avent, R.R. (1985) Decay, weathering and epoxy repair of timber, *Journal of Structural Engineering*, ASCE, Vol. 111, No. 2, February, paper No. 19510, pp. 328-42.
The paper presents results of research on the decay and weathering effects on epoxy repaired timber structures. An experimental study conducted on two types of weathered joints showed that the epoxy repair responded well. For the south-eastern U.S., the deterioration of the glue line was no more severe than the deterioration of the wood itself. It was therefore concluded that the usual precautions in maintenance of exposed wood be used to protect joints which have been epoxy repaired. It was found that decayed joints can be repaired by epoxy injection. However, due to the difficulties of implementing such repairs, it is usually better to replace seriously weathered and decayed timber.

10 Baker, A.J. (1980) Corrosion of metal in wood products, in *Durability of Building Materials and Components*, ASTM STP 691, American Society for Testing and Materials, pp. 981-93.

The report presents information on the theory of metal corrosion as it relates to the corrosion products and the deterioration of the damp wood in contact with the corroding metal fasteners. New data are also presented on the corrosion of eleven fastener materials in water-borne salt preservative-treated wood. One theory of corrosion of fasteners in wood is that corrosion takes place because of the presence of a differential aeration cell whereby the exposed part of the fastener acts as the cathode due to the higher concentration of oxygen at the surface and the embedded part of the fastener acts as the anode. A condition similar to a crevice corrosion condition sets up and the acidity of the electrolyte around the fastener will increase, thus causing deterioration of the wood around the fastener. If the fasteners are protected cathodically, it has been shown that the alkaline conditions existing around the cathode causes a deterioration of the wood around the fasteners, thus reducing the strength of the joint. A three year test programme on nails of various metals in chromated copper arsenate and ammoniacal copper arsenate treated wood showed that copper, silicon bronze, and stainless steel types 304 and 316 nails are suitable for long service life. Aluminium and zinc, tin-cadmium, and cadmium coated steel nails were found unsuitable for long service life.

11 Baker, M.C. (1969) *Decay of Wood*, Canadian Building Digest, Division of Building Research, National Research Council of Canada, CBD 111.

The decay of wood is the result of fungi attack under special conditions of moisture and temperature. The necessary conditions for fungi growth are: a source of infection which transmits the fungi to the wood (contact of sound wood with rotting wood), a suitable substrate to sustain fungus growth (this is the case for untreated wood), moisture (moisture contents between 35 to 50 % are necessary for fungi to flourish but wood cannot be considered immune until the moisture content is below 20 %), oxygen, and suitable temperature (although fungi growth is stopped at low temperature, it will resume when the temperature rises again). Acidity is also known to play an important role in the growth of fungi. Two types of rotting, namely, white rot and brown rot are briefly discussed. Five important types of fungi are briefly described. Although control of any one of the essential conditions would be sufficient to prevent fungi growth, control of the moisture content is the easiest in most situations.

12 Barton, K. and Czerny, M. (1980) The relation between the properties of the medium and the kinematics of atmospheric corrosion of steel, zinc, copper and aluminium. Assessment of results of first five-year stage in corrosion testing program of the member-nations of the COMECON, *Protection of Metals*, Vol. 16, pp. 301-8.

The authors present the results of a five year environment exposure test programme on standard specimens of steel, zinc, copper, and aluminium. The test sites were located in Hungary, Bulgaria, Czechoslovakia, and USSR, which encompass unpolluted temperature regions, subtropical regions, and very polluted industrial regions. The test specimens were exposed to the open air and sheltered in louvered cabins. The parameters measured during the course of the investigation were the relative humidity, the temperature, and the cumulative sorption of SO_2 on an alkaline surface. The time-of-wetness was taken as the time at which the relative humidity was at least 80 % while the temperature was above freezing. A regression analysis of the data was performed to determine whether the observed corrosion rates could be explained by the measured parameters. It was found that a good relationship ($r > 0,8$) exists between the corrosion rate and the, time-of-wetness and level of SO_2. The relation was found to apply mainly to steel, zinc, and copper. The corrosion rate of aluminium was not well explained by the measured time-of-wetness and the level of SO_2 . Damage functions are proposed for the metals tested in ambient air and under sheltered conditions.

13 Barton, K., Knotkova, D., Strekalov, P., Kemhadze, V., Kozhukharov, V., Sobor, A., Zaydel, M. and Bestek, T. (1980) Atmospheric corrosion of metallic systems II. Analysis of the corrosion aggressiveness of the media at the atmospheric testing stations of member-nations of COMECON, according to the results of five-year tests on steel, zinc, copper, and aluminium, in *UDC 620.193.2*, pp. 323-9.

The authors discuss the results of five-year atmospheric corrosion tests on steel, zinc, copper, and aluminium, in rural, industrial, and coastal atmospheres. They consider the feasibility of quantitative classification of the corrosive aggressiveness of the atmosphere on the basis of the total duration of wetting of the metallic surface, the contamination of the air by corrosive aggressive components, and a combined factor equal to the product of the annual duration of wetting and the concentration of sulphur dioxide and chlorides in the air. The measurements taken at each site are listed differently than in the previous paper. Namely, air temperature, relative humidity, amount of liquid precipitation, depth of snow cover, duration of fog, duration of wetting of metal surface (with the aid of an Fe-Cu galvanic sensor), wind speed and direction, sulphur dioxide content of air, amount of incident chloride particles over a two month period, and amount of dust pollution. Corrosion tests were conducted both in the open air and in a louvered cabin. The corrosiveness of the atmosphere in the louvered cabin was found to be lower than in the open air. It was also found that as the aggressiveness of the external atmosphere decreases, the level of aggressiveness of the medium in a semi-enclosed atmosphere also decreases but much more sharply than in the open air, although there are exceptions to this rule. In particular, aluminium corroded more rapidly in the louvered cabin than in the open air.

14 Bazant, Z.P. (1979) Physical model for steel corrosion in concrete sea structures – theory, *Journal of the Structural Division*, Proceedings of the ASCE, Vol. 105, No. ST6, June, pp. 1137-53.

The author presents a mathematical formulation of the corrosion process in concrete. After outlining the chemical reactions involved, he formulates the transport of oxygen and chloride ions through the concrete cover, the mass sinks and sources of oxygen, ferrous hydroxide, and hydrated red rust due to chemical reactions, the depassivation of steel due to critical chloride ion concentration, the cathodic and anodic electric potential with the concentration polarisation of electrodes, and the flow of electric current through the electrolyte in the pores of the concrete. Finally, a complete formulation in the form of an initial-boundary-value problem is obtained. Some of the limitations of the formulation are: 1) the depassivation results from the presence of chloride and not from a reduction in pH (the chloride concentration must reach a threshold value); 2) the corrosion processes on the microscopic scale leading to the formation of corrosion pits are not considered; 3) the formation of red rust only is considered (this is the most voluminous corrosion product and is produced in the presence of oxygen, while black rust has half the volume and is formed in the absence of oxygen); 4) the concrete is submerged in sea water.

15 Bazant, Z.P. (1979) Physical model for steel corrosion in concrete sea structures – application, *Journal of the Structural Division*, Proceedings of the ASCE, Vol. 105, No. ST6, June, pp. 1155-66.

The theoretical physical model developed in the previous paper is applied to a simplified calculation of corrosion rates and times of corrosion cracking of concrete cover. Approximate estimates are made of the resistance of the corrosion cell, oxygen and chloride ion transport (assumed to be quasistationary and one-dimensional), time of steel depassivation by chloride ions, and cover cracking due to rust expansion. The model is not evaluated by comparison with test data. Nevertheless, the model shows that the diffusivity of chloride ions and oxygen mainly at the cathodic areas is usually the controlling factor.

16 Beasley, K.J. (1988) Use and misuse of exterior travertine cladding, *ASCE Journal of Performance of Constructed Facilities*, Vol. 2, No. 4, November, pp. 242-53.

Travertine is a type of limestone consisting primarily of calcium carbonate deposited from solutions. It is characterised by its white to brown natural colour and random holes and veins. The author discusses various causes of failure of travertine used as exterior cladding. Travertine being a sedimentary rock, it contains bedding planes. The bedding planes create sources of weakness in tension normal to the bedding plane. The major causes of failure are: tension perpendicular to the bedding planes, freezing of water in the pores, the use of epoxy to fill voids (epoxy has a coefficient of thermal expansion much higher than travertine), severe thermal exposure, and corroding of steel anchors. The author identifies corrosion of mild steel anchors and reinforcement as the most common cause of serious stone distress. The use of stainless steel anchors and shelf angles is strongly recommended, along with proper joint and drainage design, to achieve a durable exterior cladding.

17 Beeby, A.W. (1983) Cracking, cover, and corrosion of reinforcement, *Concrete International*, Vol. 5, No. 2, pp. 35-40.

The author examines several design codes and compares their requirements as they pertain to durability. The codes have dealt with durability by specifying limits to cover, minimum cement contents and/or maximum water-cement ratios, and crack widths. The author has observed that all these limits vary significantly between codes. The author presents an assessment of those limits based on current knowledge and the results of exposure tests. Evidence from tests designed to investigate the effect of crack width indicates that the rate of corrosion of the reinforcement is unaffected by the presence of cracks and the size of the cracks. Larger cracks are likely to cause the breakdown of the passivity of the reinforcement rather earlier and hence to cause corrosion to start earlier. Tests reported in the literature have demonstrated that both the concrete cover and the mix proportions are important factors. The author, however, does not offer any explanation for the wide variation which exists between various design codes. A brief discussion of the environment severity is presented whereby the temperature and the relative humidity are believed to be two important factors. Finally, a design model for the corrosion process is discussed where the process is take as a two phase process: initiation and active corrosion phase. No specific guidelines are given to assess the duration of each phase.

18 Berke, N.S. (1991) Corrosion inhibitors in concrete, *Concrete International*, Vol. 13, No. 7, July, pp. 24-7.

The author presents a brief review of the studies done in the area of concrete inhibitors prior to 1980. The problem with some inhibitors is that they adversely affect the properties of the concrete. Calcium nitrite is used commercially on a wide scale and is not detrimental to concrete properties. Studies performed in the 1970's showed that the mechanism of corrosion protection was that of anodic inhibition. Further research showed that nitrite modifies the oxide film on the reinforcing bar to be more protective than the film that naturally occurs in concrete. Studies have also shown that calcium nitrite might decrease the resistivity of concrete. However, long-term corrosion data show that, in spite of the decrease in resistivity, corrosion rates are significantly reduced.

19 Berke, N.S., Pfeifer, D.W. and Weil, T.G. (1988) Protection against chloride-induced corrosion; *Concrete International*, Vol. 10, No. 12, pp. 45-55.

The authors believe that quality concrete alone is not sufficient to allow a structure to meet its design life in the presence of chlorides. The use of microsilica and calcium nitrite is advocated. Microsilica reduces the permeability of the concrete, considerably slowing the ingress of water-borne chlorides. Calcium nitrite, a corrosion inhibitor, promotes the stabilisation of the steel passive layer, thereby controlling the corrosion rate. The authors show the beneficial effect of microsilica in slowing down the ingress of chlorides. A life cycle cost analysis has shown that the use of microsilica and calcium nitrite in severe environments is cost effective.

20 Bhattacharjee, S., Roy, N., Dey, A.K. and Banerjee, M.K. (1993) Statistical appraisal of the atmospheric corrosion of mild steel, *Corrosion Science*, Vol. 34, No. 4, pp. 573-81.
Regression analysis of exposure data at 17 test sites throughout India lead to the development of 17 different damage functions for steel. Not enough information is presented to be able to construct the damage functions since the units for the factors measured are not given. The authors state that damage functions obtained from exposure tests are site specific and cannot be used for sites with climatological conditions different than those for which the function was derived. The factors specifically investigated were the temperature, the relative humidity, rain fall, the number of rainy days, sulphur concentration in air, and chloride concentration in air. Among those factors, the level of sulphur dioxide, and the concentration of chlorides were found to be the most significant parameters. The effect of relative humidity was found to be not significant at all but one site. At three different sites none of the measured parameters were found to be significant.

21 Bier, T.A., Kropp, J. and Hilsdorf, H.K. (1987) Carbonation and realkalinization of concrete and hydrated cement paste, in *Proceedings of the 1st International Conference on Durability of Construction Materials, RILEM, Versailles, France*, 7-11 September, pp. 927-34.
The authors have found that the pore structure of the cement paste is strongly influenced by the duration of curing. A coarse pore structure causes a higher depth of carbonation than a dense structure which is reached after a prolonged curing period. Carbonation alters the pore structure by transforming the calcium hydroxide and the CSH-gel into calcium carbonate. The application of a mortar layer on carbonated surface zones of mortars and concretes was found to restore a high alkalinity in the carbonated matrix.

22 Biestek, T. (1982) Testing electrodeposited coatings in tropical China, in *Atmospheric Corrosion*, (ed. W.H. Ailor), John Wiley and Sons, New York, pp. 775-85.
The author presents a brief review of the previous work performed on atmospheric corrosion of electrodeposited coatings in tropical countries. The results of corrosion tests at various exposure sites in China are summarised. The results of those tests were the basis for the Polish National Standard on electrodeposited and conversion coatings for articles exposed to tropical conditions. Test sites at Canton University, Canton Electrochemical Institute, Shanghai, and Julin, Hainan Island are described. The performance of zinc, cadmium, zinc chromated, cadmium chromated, nickel, brass-nickel, and copper-nickel-chromium is reported.

23 Binda, L. and Molina, C. (1990) Building materials durability: semi-Markov approach, *Journal of Materials in Civil Engineering*, Vol. 2, No. 4, November, pp. 223-39.
The authors look at the deterioration of mortar, brick, and brick masonry as a result of crystallisation of salts. Laboratory tests were conducted whereby samples were immersed in a saturated salt solution (sodium chloride, magnesium sulphate and sodium sulphate) and subsequently dried under given temperature and relative humidity. The change in compressive strength and elastic modulus was used as a measure of deterioration with the number of wetting and drying cycles. The tests showed that, in the case of constant temperature and humidity in the drying cycle, the material damage and the thickness of delamination is a function of the cycle duration. The use of compressive strength and elastic modulus represent obvious problems when the deterioration is evaluated as a function of the number of cycles. For this reason, later tests made use of reduction in cross-section area of the upper surface of prism specimens as a measure of deterioration. Tests were also conducted on specimens superficially impregnated with commercial products used for protection of external surfaces. The test results showed various response varying from accelerated deterioration, retarded deterioration and retardation of initiation of deterioration followed by sudden deterioration. A stochastic approach to predict the deterioration is presented. The probability of failure (defined as a specified loss of area) is defined as the probability that the applied number of cycles is greater than the number of cycles required to cause the desired level of deterioration (resistance). Since that resistance changes with time, a semi-Markov process was used to describe the stochastic process of deterioration. This process is described in some detail. Although the derivations may look complicated at first glance, they are quite simple (the authors use two pages of derivations to say that the probability of failure is equal to the probability that the service life is exceeded). The procedure presented in this paper can be used for the service life prediction of materials or components which follow a semi-Markov approach.

24 Binda, L. and Baronio, G. (1984) Measurement of the resistance to deterioration of old and new bricks by means of accelerated ageing tests, *Durability of Building Materials*, Vol. 2, No. 2, pp. 139-54.

During restoration of a building some of the original bricks may have to be replaced by new ones. The compatibility of these new elements with the remaining ones is of prime interest for the structural integrity of the building facade. The authors present a technique used to characterise the original brick and the new one using accelerated tests. Freeze-thaw and salt crystallisation tests were used as accelerated ageing tests. The degree of alteration was characterised by the loss of compressive strength. The measurement of ultrasonic pulse velocity in bricks was found to be an adequate technique to characterise them in terms of their durability to a detrimental environment.

25 Bjegovic, D., Ukraincik, V. and Beus, Z (1990) Evaluation and repair of concrete structure in urban environment: case study, (refer to section D).

26 Boucherit, N., Hugot-Le Goff, A. and Joiret, S. (1992) Influence of Ni, Mo, and Cr on pitting corrosion steels studied by Raman spectroscopy, *Corrosion*, Vol. 48, No. 7, pp. 569-79.

Through the use of Raman Spectroscopy the authors have tried to explain the role of Ni, Mo, and Cr on pitting corrosion of steels. It was found that Ni does not intervene against pitting corrosion (no difference was found between Fe-18Cr and AISI 302). The role of Cr in reducing the pitting rate was found to be its ability to stabilise a particular form of green rust which does not integrate chloride and hinders the Cl- ingress at the surface. Chromium plays its role mostly at the film/electrolyte interface once pitting has been initiated. The part played by Mo is to protect the metal source and to limit its roughening by the growth of a molybdate layer at the metal/film interface. Mo and Cr act in synergy.

27 Branca, C., Fratesi, R., Moriconi, G. and Simoncini, S. (1992) Influence of fly ash on concrete carbonation and rebar corrosion, in *Proceedings of the 4th International Conference on Fly Ash, Silica Fume, Slag, and Natural Pozzolans in Concrete, Istanbul, Turkey*, (ed. V.M. Malhotra), ACI SP-132, May, Vol. 1, pp. 245-55.

The effect of fly ash addition, either with or without a cement reduction, on the carbonation of concrete was investigated. The results of an experimental investigation indicated that fly ash addition reduces the carbonation rate when used without cement reduction, whereas it accelerates the process when used to replace cement. This is explained by the fact that fly ash possesses a lower content of calcium hydroxide than portland cement. Potential measurements of embedded rebar in the test samples indicated that every process which can reduce the concrete alkalinity is a necessary condition but not a sufficient one to promote corrosion of reinforcing steel.

28 Brillas, E., Costa, J.M. and Vilarrasa, M. (1990) Effect of exposure time on the atmospheric corrosion of steel, in *11th International Corrosion Congress on Innovation and Technology Transfer for Corrosion Control, Florence, Italy*, 2-6 April, Vol. 2, pp. 2.79-86.

The authors report the results of exposure tests on steel specimens at various test sites in Spain. Generally, the corrosion rate was found to be greater on the surface of the specimens towards the ground which is sheltered from the cleaning action of the rain. Corrosion was found to progress more rapidly during the first year of exposure and then decreased with time. It was found that the weight loss of the specimens could be expressed by the bi-logarithmic law $W = kt^n$, where n was found to be fairly constant between sites but k was found to vary significantly from site to site. The factor k was therefore correlated to the site conditions in term of SO_2 and Cl^- concentration. Using regression analysis, the weight loss expression applicable to all the sites was found to be $W = (177 + 1{,}39\,[SO_2] + 2{,}95\,[Cl^-]\,)\,t^{0{,}64}$.

29 Brown, P.W. and Masters, L.W. (1982) Factors affecting the corrosion of metals in the atmosphere, in *Atmospheric Corrosion*, (ed. W.H. Ailor), John Wiley and Sons, New York, pp. 31-49.

The authors present a discussion of the corrosivity of various atmospheric agents. The agents discussed are: solar radiation and wind (affect the time-of-wetness); temperature which also can affect the time-of-wetness but which effect on corrosion is not yet clear; moisture which is affected by the relative humidity, the composition of the metal, the presence of contaminants, and the presence of corrosion products; air pollutants such as sulphur oxides (with SO_2 and SO_3 the most important), hydrogen sulphide (not corrosive to ferrous metals at concentrations normally found in the atmosphere), oxides of nitrogen (nitric oxide, NO, and nitrogen dioxide, NO_2, being the most important w.r.t. corrosion), and aerosols. The paper is a good source of reference on factors affecting atmospheric corrosion.

30 Building Research Association of New Zealand (1985) *Glazing Plastics – 1. Types and Durability*, Building Information Bulletin 241.

There are four main types of plastic used for glazing: acrylic, glass fibre reinforced polyester (GRP), polycarbonate (plexi-glass), and PVC. Plastics are lighter, tougher, and easier to colour than glass, but they are less resistant to fire, they don't remain transparent, they have more thermal movement, usually cost more initially and are easily scratched. The four main types of glazing plastics are described and their modes of deterioration are presented. The main problems related to the durability of glazing plastics are: loss of surface quality caused by scratching, microcracking or crazing, pitting, surface erosion, chalking; discoloration of the resin; change of the colour of the pigments. Exposure to UV and oxygen causes a deterioration of the mechanical properties of plastics. Surface coatings such as polyvinyl fluoride or acrylic can extend the service life of glazing plastics. Guidelines for cleaning plastics are given. Water dripping from the troughs in corrugated plastic roofing can cause rust spot corrosion on unpainted galvanised steel directly below and, therefore, details which allow drainage of plastic roofing on to unpainted galvanised steel should be avoided.

31 Building Research Association of New Zealand (1985) *Glazing Plastics – 2. Design and Fixing*, Building Information Bulletin 242.

This information bulletin gives some basic guidelines for the proper installation of glazing plastics to avoid premature durability problems. Glass fibre reinforced polyester and PVC are the two plastics discussed in the bulletin. Guidance on the bending radius is given for bending of plastics to avoid stress crazing. Cutting slots of square cornered holes or nicks and notches should be avoided since cracks readily propagate from such stress raisers. Special consideration must be given to accommodate thermal expansion since the thermal movement of glazing plastics is up to nine times that of glass. The use of sealant and tape on plastics should be reduced to a minimum since sealants and glazing plastics are both based on organic polymers and there is potential for interaction between them which could be deleterious to the performance of the plastic. Some sealants may cause surface marring or crazing of the plastic and their use should be avoided when the glazing plastic is subjected to long-term stress.

32 Building Research Association of New Zealand (1984) *Corrosion and Staining of Glass Windows*, Building Information Bulletin 236.

Water is responsible for most of the deterioration observed in window glass. When water comes into contact with glass, sodium atoms near the surface of the glass can interchange with hydrogen atoms in the water. This leaves the surface layers rich in calcium atoms, as calcium is more firmly fixed in the glass structure. This calcium rich surface layer can react with carbon dioxide from the atmosphere to form insoluble residues on the glass. As a result of this reaction the water and the glass surface become alkaline and if the water remains in contact with the glass, the degree of alkalinity will cause further breakdown of the glass. If wetting of the glass surface is non-uniform, corrosion of the glass surface becomes noticeable. The bulletin lists a few techniques to prevent corrosion and staining and guidance for maintenance.

33 Building Research Association of New Zealand (1979) *Use of Anodised Aluminium*, Building Information Bulletin 213.

Anodising of aluminium is performed by passing an electric current through the aluminium while it is immersed in a suitable bath. This produces a considerable thickening of the protective oxide film. The film consists of a dense thin film above which is a tough relatively thick porous film. Use of this porous film is made to colour the surface. The pores of the surface layer can be closed by chemical sealing. The anodised aluminium can be attacked by strongly alkaline or strongly acidic chemicals and, localised attack can occur where the film has been damaged. Guidelines for design and installation practices for good service behaviour are given.

34 Building Research Establishment (1989) *Wet Rots: Recognition and Control*, BRE Digest 345, June.

Although both wet rot and dry rot can occur together, the type of rot depends on the conditions in the building. The same basic principles govern the successful eradication of both types of rot but additional measures are necessary with dry rot because of its ability to grow through masonry. The digest describes the main types of wet rot likely to be encountered in buildings, describes how to distinguish them from dry rot, and presents a strategy for their control.

35 Building Research Establishment (1986) *Zinc Coated Steel*, BRE Digest 305, January.

This digest presents some of the corrosion characteristics of zinc and steel. Zinc will form a stable protective film in environments with a pH between 6 to 12,5, with a low chloride and low oxygen content. Zinc provides protection to steel in two distinct ways. If the zinc coating is continuous over the surface it provides a barrier between the steel and the corrosive environment. The corrosion rate of zinc, once its protective oxide layer has formed, is relatively low compared to that of steel. If the zinc coating is damaged and the steel substrate is exposed, zinc will provide galvanic protection by acting as a sacrificial anode. Zinc coatings can be applied to steel either by hot dip galvanising, electrodeposition, metal spraying, or sherardising. Curves are presented which gives typical life of zinc as a function of coating thickness in various environments. Finally, corrosion of zinc in contact with various building materials is discussed.

36 Building Research Establishment (1985) *Corrosion of Metals by Wood*, BRE Digest 301, September.

Wood and metals are used together in numerous applications in construction. Several factors can affect the rate of corrosion of metals in contact with wood. In this digest the factors are identified as moisture, choice of timber species, impregnation with salts, type of metal, the use of wood preservatives, flame retardants and exposure conditions in buildings. Moisture is identified as the most important factor causing corrosion of metals in wood. The threshold value of the moisture content of wood below which no appreciable corrosion of embedded metal occurs is about 20 %. Most timbers are acidic and, as such provide a corrosive atmosphere for embedded metals. Many softwood species are less aggressive than many hardwoods. Impregnation by salts (sea water or some wood preservatives and fire retardants) create better electrolyte which helps speed up the corrosion process. Moisture control is recommended as the most efficient way to prevent corrosion of metals embedded in wood and, incidentally, rotting of the wood. Contact between dissimilar metals in wood is also to be avoided since a corrosion cell would be set up whereby the least corrosion resistant metal would suffer from accelerated corrosion.

37 Building Research Establishment (1985) *Dry Rot: its Recognition and Control*, BRE Digest 299, July.

Dry rot refers to wood decay caused by the fungus Serpula lacrymans. Since dry rot remedial treatment requires elaborate and expensive control measures, its positive identification is important. As opposed to wet rot, dry rot can grow through masonry and may therefore require more elaborate procedures to eradicate the problem. The visual appearance of wood affected by dry rot cannot be used for positive identification. However, the appearance of the fungus is described. Growth of Serpula lacrymans takes place in unsaturated wood with a moisture content above 30 %. In well designed and ventilated buildings the moisture content of timber remains below 20 %. The primary measure to prevent dry rot is therefore to dry the structure to a moisture content below 20 %. As a secondary remedial measure, identification and removal of infected timber is important. Application of fungicidal fluid can prevent the spread of dry rot. When dry rot is detected, it is preferable to replace the affected timber and treat the unaffected timber near the region affected. Preservative treated timber should be used for replacement.

38 Building Research Station (1966) *Durability and Application of Plastics*, Building Research Station Digest 69, April.

Durability of plastics used in various applications in the building industry is reviewed. Sunlight, warmth and moisture are the major influences in exposure although oxygen and atmospheric pollutants play a role. Ultraviolet radiation initiates many of the chemical reactions by which plastics are oxidised and degraded. Temperature plays an important role in determining the reaction rate leading to breakdown. Moisture may physically weaken the bond between a resin and its filler or reinforcement, it may cause dimensional changes and may also lead to changes in colour. When plastics are buried in soil, attack by rats or termites may lead to failure. Typically, degradation of plastics is much faster in rural areas where the plastics are not covered by a protective layer of dirt. The most common forms of breakdown are fading, darkening, yellowing, and chalking or erosion. Loss of plasticisers or breakdown of polymer chains may cause embrittlement of plastics. Various types of plastics commonly used in the building industry are presented and their durability and applications are discussed briefly.

39 Callaghan, B.G. (1982) Atmospheric corrosion testing in southern Africa, in *Atmospheric Corrosion*, (ed. W.H. Ailor), John Wiley and Sons, New York, pp. 893-912.

Long-term atmospheric corrosion exposure programmes have been established throughout Southern Africa by the Council for Scientific and Industrial Research (CSIR). The paper presents the results of various national research projects aimed at evaluating the performance of metals and metal coatings. A twenty-year exposure programme showed that the corrosion rate for mild steel, Cor-Ten, copper, aluminium, and stainless steel decreased with time. The corrosion rate for zinc increased with time in all areas of the country. Corrosion rates reported were based on mass loss and, consequently, does not reflect pitting. Corrosion rates from 5,8 μ/year (in dry polluted areas) up to 257 μ/year (marine environment) have been reported. Except for one exposure site close to the sea, Cor-Ten showed lower corrosion rate than plain carbon steel. The presence of industrial pollution was found to increase significantly the rate of corrosion even in the dry inland areas. The excellent protection against corrosion afforded to steel by thermally sprayed aluminium coatings lead the authors to recommend further consideration for this system in areas where corrosion is severe.

40 Calvo, L. and Meyers, M. (1991) Overlay materials for bridge decks, *Concrete International*, Vol. 13, No. 7, pp. 46-7.

The authors advocate the use of polymer overlays to provide a protective barrier to bridge decks. The conventional asphaltic overlays do not provide the appropriate wear or skid resistance and add considerable weight to the deck. The polymeric materials are usually thinner and therefore lighter and do not require extended bridge closure during installation of the system.

41 Canovas, M.F., Selva, N.H. and Kawiche, G.M. (1992) New economical solutions for improvement of durability of portland cement mortars reinforced with sisal fibres, *Materials and Structures / Matériaux et Constructions*, Vol. 25, pp. 417-22.

Vegetable fibres are sometimes used as a reinforcement of Portland cement mortar. Their decomposition with time due to their inherent weakness in this environment represents a significant problem. The research programme was carried out with the aim of solving the problem of vegetable fibre mineralisation. The work showed that the use of natural products derived from timber can reduce the mineralisation of vegetable fibres in cement mortars. The products used showed a good efficacy in alkaline reduction, mortar pore sealing, and reduction of water absorption in the mortars and fibres.

42 Carter, P.D. (1991) Sealing to improve durability of bridge infrastructure concrete, *Concrete International*, Vol. 13, No. 7, pp. 33-6.

The author relates his own experience with the use of concrete sealers on bridges in Alberta. Concrete sealers are believed to be effective at reducing the rate of deterioration of concrete curbs exposed to severe corrosive environments. ”The need for curb repairs on the Alberta bridge system has been significantly reduced since the early 1980's when routine sealing with acrylic sealers replaced the previous sealing done with linseed oil in mineral spirits ”. The lack of standardised techniques to assess the various sealers available on the market is believed to cause much of the problems encountered when selecting a sealer for specific applications.

43 Carter, J.P., Linstrom, P.J., Flinn, D.R. and Cramer, S.D. (1987) The effects of sheltering and orientation on the atmospheric corrosion of structural metals, *Materials Performance*, Vol. 26, No. 7, July, pp. 25-32.

The authors describe the results of a field exposure programme of four metals: carbon steel, Cor-Ten A weathering steel, copper, and zinc. Samples were exposed to the open atmosphere with either one side (skyward or groundward) masked, or both sides exposed and some specimens were sheltered under a transparent plastic cover. It was found that the average corrosion loss of specimens masked (average of skyward and groundward exposure) coincided to the corrosion loss of specimens exposed on both faces, except for zinc which showed a lower corrosion loss when exposed on both faces than the average of skyward and groundward exposure. This is believed to be the result of galvanic action between the two exposed faces. The corrosion loss on the skyward face was found to be greater for zinc and copper. The opposite was observed for both steels. The sulphur content in the corrosion layer was found to be greater for the boldly exposed steel than on the sheltered corrosion film. The reverse was found to be correct for zinc and copper. This was explained by the action of rain washing the corrosion products on zinc and copper. The steel corrosion film was not as much dissolved by rain water. For the same reason, the groundward corrosion film for steel was found to contain more sulphur. The limiting sulphur concentration in the corrosion films formed on sheltered and boldly exposed zinc panels suggested that in some gross way, the zinc corrosion film can be saturated with sulphur. This was not observed for the other metals.

44 Chen, S., Huang, H., Liu, C. and Pan, Y.(1992) Technique for detecting sensitization in austenitic stainless steel, *Corrosion*, Vol. 48, No. 7, pp. 594-8.

The use of potentiostatic pulse technique has been shown be effective in detecting sensitisation of austenitic stainless steel. Compared with the conventional electrochemical potentiokinetic reactivation test, the latter technique is found to be faster and is almost non-destructive.

45 Cohen, J.M. and Monteiro, P.J.M. (1991) Durability and integrity of marble cladding: a state-of-the-art review, *ASCE Journal of Performance of Constructed Facilities*, Vol. 5, No. 2, May, pp. 113-24.

Marble is a non-traditional cladding material for high-rise buildings. The improvement in cutting processes in the last few years have allowed panel thickness to decrease markedly. As a result, problems have developed with marble cladding on buildings which are no more than 20 years old. Expensive replacement work of marble cladding on the Lincoln First Tower in Rochester and on the Amoco Building in Chicago is the result of the most common problem with thin marble panels, namely permanent bowing of the material. The authors outline the research that has been done on the use of stone cladding on buildings over the past century. It seems that the cause of the problem with marble panels is not well understood but it is most likely related to the crystal structure of the marble. The most important agents for the deterioration of marble are pollutants such as carbon dioxide, nitric acid, and especially sulphur dioxide. The authors indicate that there is a significant lack of proper guidelines for selecting and testing marble's durability and stability. The present ASTM standard tests do not provide the basis for a comprehensive test programme.

46 Cramer, S., Carter, J.P., Linstrom, P.J. and Flinn D.R. (1988) Environmental effects in the atmospheric corrosion of zinc, in *Degradation of Metals in the Atmosphere*, (eds. S.W. Dean and T.S. Lee), ASTM STP 965, ASTM, Philadelphia, pp. 229-47.

As part of the NAPAP (National Acid Precipitation Assessment Program) work, field exposure corrosion tests are conducted. The authors present some of the results obtained from zinc corrosion tests. The paper discusses seasonal variations in air quality and rain chemistry at four exposure sites in the U.S. The effect of environmental conditions on the nature of the corrosion film chemistry is investigated. The test data, obtained over a three-year period, showed evidence that the corrosion film consists of an outer layer that is relatively unprotective, and an inner layer sensitive to sulphur dioxide (SO_2) concentrations that controls the corrosion process in long-term exposures. Measurements of SO_2 levels in the air indicated that the maximum level usually occurs during winter and the minimum occurs during summer. This is in good agreement with patterns related to regional energy demands. Monitoring of the hydrogen ion loading of the precipitation showed that the maximum value usually occurs during the summer while the minimum amount occurs during winter. No explanation for this six-month offset is given. As a result of an analysis of the test data, a damage function was proposed which relates the precipitation pH and quantity to the zinc runoff loss.

47 Davies, H. (1990) Studies of the performance of fusion bonded epoxy coated reinforcement during the construction process, in *Proceedings of the International Conference on Protection of Concrete, University of Dundee*, (eds. R.K. Dhir and J.W. Green), Scotland, U.K., 11-13 September, pp. 269-80.

The authors presents an historical review of the use of epoxy coated reinforcement in the U.S., Canada, and the U.K. He reports on the results of site trials of epoxy coated reinforcement designed to assess the damage sustained by the coating during 1) transport and site storage, 2) preparation of reinforcement cage, 3) placement and compaction of concrete. The test specimens were small walls. The steel was examined before placement of the concrete and it was found that 40 % of the bends had holidays. After the concrete was placed and vibrated, the concrete was removed and the reinforcement was washed. An average of 7,8 defects (where the steel was exposed) per meter of rebar were detected. Placement and vibration of the concrete was found to inflict significant damage to the coating. The use of a vibrator in the forms was found to be the main cause of damage during placement of the concrete. In general, although the damage to the coating before placement of the concrete was within acceptable limits set by British codes, those damages were found to exceed the limits after placement of the concrete. The author advises caution in the use of epoxy coated rebars since the implications of his findings on the durability of reinforced concrete are not yet known.

48 Dhir, R.K., Hewlett, P.C. and Chan Y.N. (1990) Assessment of concrete durability by intrinsic permeability, in *Proceedings of the 5th International Conference on Durability of Building Materials and Components, Brighton, 1990*, (eds. J.M: Baker, P.J. Nixon, A.J. Majumdar and H. Davies), E & FN Spon, U.K., 7-9 November, pp. 503-13.

The intrinsic permeability of concrete was measured using air test and water test. Although the air tests did not give the same results as the water permeability tests, a correlation was found to exist between the two tests. It was shown that the compressive strength of concrete alone should not be used as a means of assessing its permeability potential since no direct relationship exists between strength and permeability. The paper does not cover any aspect of concrete durability other than the fact that the authors mention in their abstract that the link between measured permeability and durability is currently being studied.

49 Edgell, G.J. (1987) The resistance to corrosion of steel in reinforced and prestressed masonry, in *Proceedings of the 1st International Conference on Durability of Construction Materials, RILEM, Versailles, France*, 7-11 September, pp. 1021-4.

The paper describes the provisions for the protection of steel in masonry given in BS 5628: Part 2. The code suggests the type of steel (plain carbon steel, galvanised steel, austenitic stainless steel) to be used if durability is to be obtained in various environments. The sites are classified into categories E1, E2 and E3 using the exposure to wind driven rain. The basis of the classification is the amount of wind driven rain falling on a vertical surface during the worst likely spell of bad weather in a three year period. The fourth exposure condition is for severe exposure whereby the masonry is exposed to salt or moorland water, corrosive fumes, abrasion or de-icing salts. Under those severe exposure conditions, E4, the code specifies the use of austenitic stainless steel or carbon steel coated with at least 1 mm of stainless steel. Special requirements are also placed when the porosity of the materials used (brick, block or mortar) exceeds a certain limit. The code also considers the situations where protection is given by concrete cover rather than by the steel resistance to the environment.

50 Fagerlund, G. (1977) The critical degree of saturation method of assessing the freeze/thaw resistance of concrete, *Materials and Structures / Matériaux et Constructions*, Vol. 10, No. 58, July-August, pp. 217-29.

This is a report prepared on behalf of RILEM Committee 4 CDC and was submitted for comments. A test procedure is presented whereby the freeze/thaw resistance is determined from the degree of saturation at which damage of concrete occurs upon freezing. The freeze/thaw resistance is defined as the critical degree of saturation minus the actual degree of saturation. Therefore, the proposed test consists of two phase: one phase to determine the critical degree of saturation and the second phase to determine the actual degree of saturation. For assessing a type of concrete, the actual degree of saturation is replaced by the capillary degree of saturation which gives a measure of the amount of water that can be absorbed when the concrete is in contact with water. This leads to a simpler test which is easily reproducible. The paper explains in detail the procedure to adopt to determine the critical degree of saturation either using multiple or single cycle freeze/thaw. No comments are made about the relation between the single and multiple cycle test. The critical degree of saturation is determined from measurements of the dynamic E-modulus and damage due to freeze/thaw is detected as the point at which the modulus starts to drop more rapidly.

51 Fassina, V. (1988) Environmental pollution in relation to stone decay, *Durability of Building Materials*, Vol. 5, Nos. 3&4, pp. 317-58.

The paper presents the various sources of atmospheric pollutants such as sulphur compounds, nitrogen oxides, ozone, hydrogen chloride, carbon dioxide. The corrosion reaction associated to each compound is also presented along with discussion of their implication on stone decay.

52 Feliu, S. and Morcillo, M. (1982) Atmospheric corrosion testing in Spain, in *Atmospheric Corrosion*, (ed. W.H. Ailor), John Wiley and Sons, New York, pp. 913-21.

Field and laboratory testing are extensively used to study the problem of atmospheric corrosion in Spain. The authors present a description of the field testing programme which includes a network of test sites covering various types of environment such as urban, industrial, marine, and rural. One of the objectives of the field tests is to correlate corrosion data with meteorological data. Relative humidity, temperature, precipitation and the number of days with rain are recorded. In addition, the SO_2 and chloride levels are being monitored at various sites. The time-of-wetness is generally taken as the time during which the relative humidity is greater than 85 %. Assuming that the time-of-wetness was almost constant at various test sites with varying level of sulphur dioxide, it was shown that the corrosion losses for steel exposed to an atmosphere containing 0,5 mg SO_2/dm^2/day was about 30 % lower than in an atmosphere having 1,75 mg SO_2/dm^2/day. The effect of SO_2 on zinc was found to be more marked than on steel (the rate of corrosion of zinc under the same change of SO_2 was halved). A study of long-term atmospheric corrosion showed that the bilogarithmic equation fitted well the observed rates of corrosion for steel, zinc, copper, and aluminium after three years of exposure in open and sheltered environments. It was observed that, whereas the rate of corrosion decreases with time in the open atmosphere, it remains fairly constant in a sheltered environment. This behaviour can be expected if the rust formed on the metal surface is not protective, but rather a source of moisture capillary condensation and of accumulation of pollutants.

53 Flinn, D.R., Cramer, S.D., Carter, J.P. and Spence, J.W. (1985) Field exposure study for determining the effects of acid deposition on the corrosion and deterioration of materials – description of program and preliminary results, *Durability of Building Materials*, Vol. 3, No. 2, pp. 147-75.

As part of the research activities of Task Group G – Effects on Materials, within the National Acid Precipitation Assessment Program (NAPAP), the Bureau of Mines (BOM), U.S. Department of the Interior has initiated a field-exposure programme to study the effects of acid deposition on the corrosion and deterioration of metals commonly used in outdoor structures. Structural metals are exposed at five different sites. At each site the air quality (SO_2, NO, NO_x, NO_2, O_3), meteorology (wind speed and direction, temperature, relative humidity, precipitation, solar radiation, and surface wetness time and events) and rain chemistry (pH, cations and anions) are continuously measured. The test programme and the preliminary results after two years of exposure of the test samples are presented. The metals investigated are: 1010 Carbon steel; Cor-Ten A steel; 110 Copper; 3003-H14 Aluminium; 191 Zinc; Galvanised steel; and Galvalume. Concentration of sulphur in the corrosion films of all the metals is higher on the groundward side than on the skyward side for one-month exposure. Dry deposition of neutralising particles on the skyward side, coupled with washing by rain, probably accounts for this difference.

54 Ford, P. and Corderoy, D.J.H. (1975) The corrosion characteristics of galvanised reinforcing bar, in *Proceedings of the 6th International Congress on Metallic Corrosion, Sydney, Australia*, December, pp. 1508-16.

Experimental results showed that the galvanised bar improved the corrosion resistance of reinforcement embedded in concrete at depth of cover from 10 mm and 20 mm when subjected to impressed currents. The corrosion resistance of galvanised bars in a natural marine environment was greater than that of the black bar. The black bar was heavily coated with rust whereas the galvanised bar was protected by the zinc coating. The lowest overall corrosion rate for the galvanised coating is in the pH range of 7 to 12, and as such the coating is able to withstand a lowering of the pH due to carbonation of the concrete to a greater degree that the steel reinforcement. Above a pH of approximately 12,5, the zinc reacts rapidly with the alkaline environment to form soluble zincates. This occurs when the zinc coating is placed in wet cement. When the galvanised coating has been passivated the steel-concrete bond is superior for the hot dip galvanised bar than for the black bar.

55 Fukushima, T., Sato, N., Misamatsu, Y., Matsushima, T. and Aoyama Y. (1982) Atmospheric corrosion testing in Japan, in *Atmospheric Corrosion*, (ed. W.H. Ailor), John Wiley and Sons, New York, pp. 841-72.

The cost of corrosion protection in Japan has been evaluated at 1,9 % of the gross national product (US$12 thousand million) in 1974. Over 60 % of that cost can be attributed to atmospheric corrosion alone. Various exposure sites have been established throughout Japan under the supervision of the Japan Weathering Test Center. Corrosion testing of carbon and weathering steels (conducted by the Research Group on the Corrosion Protection of Steel Frame Structures) showed that the time-corrosion curves can be described well by the bilogarithmic law. Multiple regression analysis of the test data indicated that the temperature, the relative humidity, the chloride concentration, the sulphur dioxide concentration and the amount of rainfall had a significant influence on the rate of corrosion. A regression equation has been presented. Exposure test results are presented for aluminium and its alloys, stainless steels, and coated steels. Studies on the nature and properties of atmospheric corrosion products of various metals are presented. Finally, a description of accelerated corrosion tests developed by the Japan Society of Steel Construction is briefly discussed.

56 Gatto, F. and Perrone A. (1982) Atmospheric corrosion testing of aluminium in Italy, in *Atmospheric Corrosion*, (ed. W.H. Ailor), John Wiley and Sons, New York, pp. 827-39.

The authors present the results of an investigation of the atmospheric corrosion of aluminium conducted by the Istituto Sperimentale dei Metalli Leggeri (ISML) since the early 1950s. Marine, industrial and urban test stations were established. The influence of anodising the surface was investigated in the three atmospheres. It was found that in order to offer good protection, the thickness of anodised aluminium had to be greater as the impurity level in the alloy increases and the structure of metallic compounds becomes coarser. It was found that a 24-µm film obtained through a good sulphuric acid process did not show significant corrosion after nine years of exposure in an urban-industrial area.

57 Graedel, T.E. (1984) Concentrations and metal interactions of atmospheric trace gases involved in corrosion, in *Proceedings of the 9th International Congress on Metal Corrosion, Toronto*, 3-7 June, Vol. 1, pp. 396-401.

The author presents a review of the literature which shows that a total of 14 trace gases in the atmosphere interact with metals. The results of tests show that hydrogen sulphide (H_2S), sulphur dioxide (SO_2), ammonia (NH_3), nitrogen dioxide (NO_2) and ozone (O_3) are present in sufficient concentrations in the atmosphere to corrode metals. The other gases were found to be in amount small enough to be ignored. Typically encountered concentrations of the 14 important gases are presented. The significant gases affecting corrosion of steel and zinc are reported to be H_2S, SO_2. Although other gases such as formic acid (HCOOH) and nitric acid (HNO_3) are also reported to affect corrosion of steel and zinc, their concentration in the atmosphere is believed to be too low to be of importance. Although not clear, it seems that the synergetic effect of some gases has been ignored in this study.

58 Graedel, T.E. and McGill, R. (1986) Degradation of materials in the atmosphere, *Environmental Science Technology*, Vol. 20, No. 11, pp. 1093-100.

The paper presents a general summary of the problem of corrosion in the atmosphere. The authors discuss topics such as the assessment of the degradation potential of the atmosphere (the atmosphere contains various corrosive elements which are occurring either naturally (oxygen and carbon dioxide) or the result of pollution (sulphur)), degradation by rain and snow, degradation by airborne particles, degradation by dew and fog, and indoor degradation. The authors also discuss the synergistic effects of the various corrodents.

59 Grimm, C.T. (1985) Durability of brick masonry: a review of the literature, in *Masonry: Research, Application, and Problems*, (eds. Grogan and Conway), ASTM STP 871, pp. 202-34.

The paper summarises most of the literature published on durability of brick masonry since the beginning of the century. The destructive agents which are believed to cause deterioration of brick masonry are discussed briefly for the bricks and the mortar. Water and salts which are carried by water are believed to be the two most important agents of destruction of brick masonry. The mechanisms of destruction are also presented. Internal stresses created as the saturated brick undergoes a change in temperature is believed to cause damage is the porosity of the brick cannot dissipate the pressure quickly. Similarly, crystallisation pressure as water evaporates can cause some distress. The porosity and pore size distribution of brick units is believed to be one of the most important property of brick which controls its durability. Other topics addressed in this review are efflorescence, environment, brick specifications, design considerations, construction and maintenance. No fully reliable test for evaluation of frost resistance is believed to exist. The properties of the mortar is also outlined to be very important on the overall durability of brick masonry. When discussing the environment, conditions leading to more cycles of freezing and thawing and conditions where water may bring more salt in the brick are identified as the only conditions causing durability problems. Temperature gradient is not identified as a contributing factor to deterioration. Finally, the author concludes that the ASTM freeze-thaw test for bricks is the best method presently known to assess the durability of brick. The method, however, presents some imperfections. It is believed that keeping the masonry dry is the most important action that can be taken the improve durability of bricks.

60 Grondin, G.Y. (1993) *Damage Functions for Service Life Prediction of Zinc and Steel Components*, National Research Council Canada, Institute for Research in Construction Internal Report IRC-IR-647, August.

This report presents a review of about 20 different damage functions presented in the open literature for zinc and steel. The damage functions are compared to each other in terms of the predicted effect of time-of-wetness, sulphur dioxide, chlorides, precipitation acidity, temperature, and degree of sheltering. Little agreement was found to exist between the various functions. It is concluded that the damage functions based on field exposure studies cannot be used reliably for environmental conditions substantially different from those present in the field for which the damage functions were derived. Damage functions based on theory are believed however to have good potential but testing of those theoretical damage functions still remains to be done. The time-of-wetness and sulphur dioxide were found to be two very important parameters in corrosion of zinc and steel.

61 Grube, H. (1987) Measuring gas permeability of concrete for assessing factors of durability, in *Proceedings of the 1st International Conference on Durability of Construction Materials, RILEM, Versailles, France*, 7-11 September, pp 1206-13.

The oxygen permeability has been found to be a suitable indicator to measure the open porosity and its changes in relation to concrete composition and curing. Concrete which is commonly regarded to be suitable for outdoor exposure ($w/c \leq 0{,}60$ and medium curing) showed coefficients of permeability of less than 10^{-16} m^2. It was found that concrete with $w/c = 0{,}8$ and cured extremely well can reach the same permeability as a badly cured concrete with $w/c = 0{,}6$. The tests described by the author showed that in the range of low permeability there seems to be a linear correlation between permeability and carbonation depth. It was found that the permeability measurements were strongly influenced by the thickness of the specimens used for the measurement. This may be explained by the increase in probability for local defects such as pores being able to connect the testing surfaces. Specimens with a 50 mm thickness were mostly used in the investigation. In general, the compressive strength of standard cubes seemed to be unsuitable to predict permeability or carbonation in progress.

62 Guttman, H. (1968) Effects of atmospheric factors on the corrosion of rolled zinc, in *Metal Corrosion in the Atmosphere*, ASTM STP 435, American Society for Testing and Materials, pp. 223-39.

A long term exposure programme has been carried out at Birchbank, B.C., where rolled zinc corrosion data and certain atmospheric factor data were obtained. Zinc is sensitive to variations in climatic and atmospheric pollution conditions, and, as a result, panels exposed on different dates for a specific period of time can corrode at different rates. An empirical equation has been developed for Birchbank which relates corrosion of zinc to the time of wetness of exposed panels and the average atmospheric sulphur dioxide content during the time panels are wet. The equation accounts for most of the observed variations in corrosion losses and is valid for exposure periods of up to 256 weeks duration. The paper also presents information concerning the relative corrosion rates of the skyward and groundward surfaces of zinc panels, the relationship between time of panel wetness and relative humidity, and the relationship between atmospheric sulphur dioxide as measured by a Thomas autometer and by the lead peroxide method.

63 Guttman, H. and Sereda, P.J. (1968) Measurement of atmospheric factors affecting the corrosion of metals, in *Metal Corrosion in the Atmosphere*, ASTM STP 435, American Society for Testing and Materials, pp. 326-59.

Atmospheric factors such as time-of-wetness, panel temperature, atmospheric sulphur dioxide and atmospheric chloride content were measured at four inland and three coastal North American test sites while corrosion data for steel, copper, and zinc were being developed. Statistical analysis showed conclusively that the measured atmospheric factors completely controlled the rates of corrosion at all sites for at least the first month. For longer term exposures the atmospheric factors control the corrosion of zinc under most conditions investigated and of steel and copper at the marine sites. With steel and copper at the inland sites it was assumed that control of the corrosion process is gradually transferred from the atmospheric factors to factors related to the changing surface condition resulting from accumulation of corrosion products and foreign agents. This appeared to be true in all cases except steel at Ottawa and copper at South Bend. The results of this research programme indicated an area of future work concerned with the character of corrosion products which form on the surface of a corroding specimen and which modify the primary control of the atmospheric factors on the rate of corrosion of metals. (References are given for techniques of SO2 and chloride measurement techniques).

64 Haagenrud, S.E. (1985) Mathematical modelling of atmospheric corrosion and environmental factors, *in Problems in Service Life Prediction of Building and Construction Materials*, (ed. L. W. Masters), Martinus Nijhoff Publishers, pp. 229-52.
The corrosivity of the atmosphere is described by so-called dose-response relationships obtained by field tests at various sites in Scandinavia. The author describes the lab and field studies carried out by the Norwegian Institute for Air Research (NILU) to establish dose-response relationships for steel and zinc. It was found that long term corrosion rates cannot be estimated directly from short term measurements of corrosion rate. The rate of corrosion can increase or decrease with time depending on the characteristics of the atmosphere. In general, the rate of corrosion was observed to decrease with time, but the rate of decrease varies significantly from region to region. The sulphur dioxide and chloride concentration were found to have a significant impact on the rate of corrosion. The short term (one year) dose-response functions were used to classify the atmospheres according to corrosivity. An exponential rate model is proposed for the prediction of long term corrosion rate.

65 Hakkarainen, T. and Yläsaari, S. (1982) Atmospheric corrosion testing in Finland, in *Atmospheric Corrosion*, (ed. W.H. Ailor), John Wiley and Sons, New York, pp. 787-95.
The aggressivity of the atmosphere in Finland is believed to be generally low. As a consequence, not much attention has been given to the problem of atmospheric corrosion in the past. However, since the direct application of test results from other countries may be misleading when applied in Finland, more research has been directed toward that problem since 1968 when the first test site was brought into use. The main part of the testing is directed by the Technical Research Center of Finland. Steel producers and paint manufacturers also have their own testing facilities. A brief description of the five test sites (all located in southern Finland) owned by the Technical Research Center is presented. The test results on mild steel have shown that the rate of corrosion can be described by the bilogarithmic law. The results of test on weathering steels, galvanised steel, aluminium and copper alloys, and stainless steels are discussed. The test results so far have shown that the corrosion rates are indeed very low.

66 Harrison, W.H. and Gaze, M.E. (1989) Laboratory-scale tests on building mortars for durability and related properties, *Masonry International, Journal of the British Masonry Society*, Vol. 3, No. 1, pp. 35-41.
Small specimens of mortar made by normal brick-laying techniques were subjected to wet/dry, freeze/thaw cycles to simulate severe exposure. A total of 84 mixes, made with two types of cement, seven different sands, two types of mortar, and three levels of cement content, were investigated. Specimens of mortar beds laid between pairs of dry porous bricks had higher strength, lower carbonation rates and better durability than specimens laid between saturated or low absorption bricks which exerted no suction. Durability was assessed by the amount of degradation after exposure to 25 cycles of frost, sulphate or the combination of sulphate and frost. Resistance to frost was increased by the presence of entrained air. Only the air-entrained mortars made with sulphate-resisting Portland cement and a cement to sand ratio of at least 1:6 were resistant to the combination of frost and sulphate. It is estimated that for mortars with cement contents from 120-300 kg/m^3, the minimum times for carbonation of 100 mm joints in internal walls would vary from 1,5 to 4,5 years depending also on the mortar type.

67 Haver, C.A. (1989) Corrosion of steel embedded in masonry walls, *Material Performance*, Vol. 28, No. 12, December pp. 44-6.
The paper is a student poster based on a senior project for a bachelor's degree. The paper presents the results of an investigation of the corrosion of reinforcing steel embedded in a seawall. A half-cell potential contour map is presented and ASTM guidelines are used to interpret the contour map. The study showed that half-cell potential surveys conducted in accordance to ASTM C 876 cannot predict the presence or absence of corrosion in masonry walls. Carbonated masonry mortar was found to be protective of embedded steel, and the embedded steel did not corrode, even in the presence of environmental chlorides.

68 Haver, C.A., Keeling, D.L., Somayali, S., Jones, D. and Heidersbach, R.H. (1990) Corrosion of reinforcing steel and wall ties in masonry systems, (refer to section D).

69 Haynie, F.H. (1982) Evaluation of the effects of microclimate differences on corrosion, in *Atmospheric Corrosion of Metals*, (eds. S.W. Dean, Jr., and E.C. Rhea), ASTM STP 767, American Society for Testing and Materials, pp. 286-308.
The author presents the results of a statistical analysis of weather data and data from Manfeld's Atmospheric Corrosion Monitors (ACM) (these monitors are galvanic cells of copper/zinc and copper/steel). The ACM's were used to indicate the magnitude of corrosivity as well as time-of-wetness. Since covariance exists between some of the variables measured the large set of data collected was partitioned into small subsets. It was found that corrosion rates are strongly dependent on relative humidity (the time-of-wetness was related to the relative humidity through regression analysis of the field data), the flux to the surface of total oxides of nitrogen, and the flux to the surface of total sulphur. The relative humidity was found to be the most significant factor and was also found to be the factor the most in error since R.H. was not always measured at the site where the ACM's were located. For both the steel and the zinc ACM's, temperature and relative humidity affected corrosion rate in increasing order of significance.

70 Haynie, F.H. (1978) Theoretical air pollution and climate effects on materials confirmed by zinc corrosion data, in *Proceedings of the 1st International Conference on Durability of Building Materials and Components, Washington DC 1978*, ASTM STP 691, American Society for Testing and Materials, pp. 157-75.
The author attempts to evaluate the applicability of theoretical damage functions for zinc and galvanised steel. The author states that surfaces contaminated with sea salt are expected to be wet when the relative humidity exceeds 75 %. Deposition rate of SO_2 is calculated from the horizontal wind velocity. Using calculated deposition velocity and knowing the stoichiometry of the corrosion reaction, a damage function can be obtained. Unfortunately, the author does not present the damage function that he did obtain. Comparison of predicted value with theoretical predictions is discussed but the results are not clear. Test results of corrosion rate of zinc obtained by other researchers are presented but, unfortunately, TOW reported by the author is obtained from the average relative humidity over a long period of time and an expression derived by the author and calibrated by measurements made at sites other than the test sites where corrosion rates were measured. TOW for another set of data reported by the author is not reported. Although the paper presents some interesting concepts it is hard to follow the logic. The comparison of apparently theoretical predictions with experimental results is very obscure.

71 Haynie, F.H. and Upham, J.B. (1974) Correlation between corrosion behavior of steel and atmospheric pollution data, in *Corrosion in Natural Environments*, ASTM STP 558, American Society for Testing and Materials, pp. 33-51.
Atmospheric corrosion tests on enamelling steel at 57 sites showed that the average relative humidity and level of sulphate in suspended particulate or average level of sulphur dioxide had a significant effect on the depth of corrosion on the tested samples. According to statistical analysis, differences in average temperature, average total suspended particulate, and average nitrate in suspended particulate caused insignificant changes in the steel corrosion rate. Sulphur dioxide was a significant variable only when sulphate in suspended particulate was not included in the regression analysis.

72 Hilsdorf, H.K. (1989) Durability of concrete – a measurable quantity, in *Durability of Structures, IABSE Symposium, Lisbon*, 6-8 September, Vol. 57/1, pp. 111-24.
The author investigated if the air permeability of concrete only is suitable to characterise concrete durability in a general way. The characteristic air permeability coefficient KAC, defined by the author, correlates well with the progress of carbonation under laboratory conditions. Effects of water/cement ratio and curing are reflected by KAC. However, if cements containing larger amounts of components other than portland cement clinker is used, the relation between depth of carbonation and KAC is no longer unique. The author concludes that concrete air permeability is not the unique parameter to describe concrete durability, and it is unlikely that such a parameter exists. The author proposed an expression to calculate the depth of carbonation as a function of time and concrete permeability.

73 Ho, D.W.S, Beresford, F.D. and Lewis, R.K. (1984) Durability of above-ground structures as affected by concrete constituents, in *Proceedings of the 3rd International Conference on Durability of Building Materials and Components*, *1984 Espoo, Finland*, 12-15 August, Vol. 1, pp. 163-75.

The authors present test results from other researchers to demonstrate that concrete of comparable strength and slump, but of different constituents, do not necessarily carbonate at the same rate and the ease of water movement and the quality of the concrete (as indicated by water penetration) can vary over a wide range. The curing time was found to affect significantly the permeability of concrete.

74 Hoff, G.C. (1991) Durability of offshore and marine concrete structures, in *2nd International Conference on Durability of Concrete, Montreal, Canada*, (ed. V.M. Malhotra), ACI SP-126, Vol. 1, pp. 33-64.

The paper reviews the deterioration mechanisms of concrete (wetting and drying, freezing and thawing, abrasion by ice, chemical attack or mineral depletion by water, salt accumulation, attack by marine organisms). It also reviews the recent trends in strength development for concretes made with modern materials. The author concludes that the key factor to improve the durability of concrete in sea water is to make the paste fraction of the concrete less permeable by: 1) lowering the *w/c* ratio as low as practical; 2) adding finely divided siliceous material to all concrete such as natural pozzolans; 3) requiring sufficient amounts of cement and cementitious materials in the concrete.

75 Höglund, I., Espling, R. and Hilding, G. (1989) Functional studies of flat roofs covered with membranes of butyl rubber, in *Quality for Building Users Troughout (sic.) the World, CIB 89, XIth International Congress, Paris*, 19-23 June, Theme II: Lifespan of Buildings, Vol. 1, pp. 413-22.

The results of a survey of 21 roofs, consisting of loose-laid butyl rubber covered with gravel, are presented. All the roofs surveyed are situated in the Stockholm area and are more than ten years old. Some 38 % of the roofs were problem-free, 38 % had minor leaks which the janitor managed to stop, and 24 % had more serious leaks which the janitor could not stop. Testing of the butyl rubber showed that ageing of the rubber was not the cause of problems. Rather, details such as the number of roof drains per unit area, the hardness of the underlying surface, care to remove sharp objects from the surface covered with the rubber, and maintenance were found to be the factors controlling the durability of such roofing systems.

76 Horton, J.B., Borzillo, A.R., Harvey, G.J. and Reynolds, J. (1975) Corrosion characteristics of zinc, aluminium and aluminium-zinc alloy coatings on steel, in *Proceedings of the 6th International Congress on Metallic Corrosion, Sydney, Australia*, December, pp. 794-801.

The corrosion behaviour of three hot dip metallic coatings was investigated. The systems investigated were, hot dipped galvanised steel, aluminised steel, and galvalume sheet. The latter is an aluminium-zinc alloy developed by the Bethlehem Steel Corporation and comprising 55 % aluminium, 1,5 % silicon, with the balance zinc. The mechanism of corrosion of hot dip galvalume steel was investigated by examination of the coating structures before and after exposure at exposure sites in the U.S. and Australia. The results show that, initially, the zinc-rich phase corrodes preferentially, at a slower rate than galvanised coatings, and as this phase becomes depleted, or is replaced with corrosion product, the corrosion rate decreases further and becomes more characteristic of the aluminium-rich phase. Galvanic protection is provided at sheared edges, either by the zinc-rich phase corroding preferentially, or by both phases, depending on the chloride content of the environment. Comparative corrosion rate measurements showed that galvalume coatings are generally about 2 to 4 times as corrosion resistant as galvanised coatings of equal thickness.

77 Hudson, J.C. and Stanners, J.F. (1953) The effect of climate and atmospheric pollution on corrosion, *Journal of Applied Chemistry*, Vol. 3, No. 2, pp. 86-96.

This is an early paper presenting the results of one of the first field exposure test programme. This paper has been cited by many others as the source for proposed damage functions for corrosion of steel and zinc. The paper, however, does not propose any damage function, but only presents some of the results of a field exposure programme. The data presented and discussed cover some 20 test sites over the world and cover different types of environment varying from marine to rural environments. Details of the test programme are presented and the test results are discussed. The rate of corrosion was found to vary widely between the different sites. Sulphur pollution of the atmosphere was found to be a determining factor in Great Britain where an almost perfect correlation was found between the corrosion rate and the levels of sulphur at 14 sites. The rate of corrosion of zinc exposed to sulphur pollution was found to remain unchanged with time whereas the rate of corrosion of steel was found to decrease with time. This was explained by the protective nature of iron oxides. The effect of height above the ground on the corrosion rate of exposed samples was found to be negligible. Corrosion rates were found to decrease abruptly with increasing distance from the coast line. The corrosion rate is reported to drop by an order of magnitude within half a mile from the coast.

78 Idorn, G.M. (1991) Concrete durability & resource economy, *Concrete International*, Vol. 13, No. 7, pp. 18-23.

The author discusses the changes that took place over the years in the construction industry and technology and how those changes modified the durability of concrete. The technological developments in the concrete industry have brought significant improvements in the conventional concrete. The use of conventional portland cement with aggregates is now a thing of the past. The changes made over the years have affected the way in which the concrete is produced, placed, and cured. Use of chemical additives in concrete has rendered the use of vibration and early curing less critical. Unfortunately, the control of the quality is not up to date with the technological changes which took place in the product. Better tests have to be devised to assess concretes for effects such as alkali-aggregate reactivity, etc... Pointing at the research performed in the field of forensic engineering, the author states that the technology to replace the conventional standard sample testing seems within reach.

79 Johansson, L.-G. (1984) The corrosion of steel in atmospheres containing small amounts of SO_2 and NO_2, in *Proceedings of 9th International Congress on Metal Corrosion, Toronto*, 3-7 June, Vol. 1, pp. 407-11.

The author describes the results of a laboratory experiment where polished mild steel specimens were exposed in climatic chambers at constant humidity and pollutant concentration. The pollutants were 1,3 ppm SO_2 and 3,0 ppm NO_2. Since both pollutants result from the combustion of fossil fuel, they often occur simultaneously in the atmosphere. The test were conducted at 50 % RH and at 90 % RH. At low relative humidity SO_2 by itself was not found to be very corrosive since very little is adsorbed on the surface. However, at high relative humidity the corrosion effect of SO_2 was found to be much more important. In contrast the rate of corrosion from NO_2 was found to be very little affected by the relative humidity. The addition of NO_2 at a relative humidity of 50 % was found to substantially increase the corrosivity of SO_2. This effect, however, was not observed at 90 % RH. The lack of dependence of corrosivity on the RH in the presence of NO_2 was explained by the observed formation of a liquid phase on the surface at all the relative humidity levels investigated.

80 Johansson, L.-G., Lindqvist, O. and Mangio, R.E. (1988) Corrosion of calcareous stones in humid air containing SO_2 and NO_2, *Durability of Building Materials*, Vol. 5, pp. 439-49.

Three type of stone samples (limestone, marble, and travertine) were exposed to synthetic gas mixtures containing NO_2 and SO_2. Those pollutants are found in increasing amounts in the atmosphere as a result of combustion of fossil fuels. The results of the exposure tests showed that SO_2 generally has a greater influence on the corrosion of polished stones than NO_2. However, the addition of NO_2 to SO_2 containing atmosphere was found to increase the corrosion substantially. The tests were conducted at a high relative humidity of 90 %. The synergistic effect was not observed at a relative humidity of 50 %. The composition of the corrosion products was investigated and the possible reaction scheme for the interaction of SO_2 and NO_2 with the stone surface was proposed.

81 Justo, M.J. and Ferreira, M.G.S. (1993) The corrosion of zinc in simulated SO_2-containing indoor atmospheres, *Corrosion Science*, Vol. 34, No. 4, pp. 533-45.

Electrochemical tests were carried out on zinc/zinc cells covered with a thin layer of zinc sulphate to simulate atmospheres containing 20, 50 and 75 $\mu g/m^3$ of SO_2 and a relative humidity of 85 %. The testing procedure using $ZnSO_4$ assumes that the first step in the corrosion process by SO_2 is the oxidation of SO_2 to sulphate. The tests try to simulate indoor environment since the effect of rain is not considered. Current density and impedance were measured after the specimens were exposed to the test solution and placed in a dry or 85 % RH atmosphere for drying. The results showed that the corrosion rates are higher for the lower concentration of equivalent SO_2. The drying time for the higher concentrations was higher than for the low concentration, which is contrary to the case for iron. The amount of sulphate in the corrosion products increases with the equivalent concentration of SO_2. However, in outdoors exposure, the washing off of the corrosion products by rain stimulates the dissolution of the sulphate formed, with the corrosion products layer being continually re-established by hydroxide/oxide formation.

82 King, G.A. (1988) A corrosivity survey on a grid of sites ranging from rural to moderately severe marine; part 1. Steel, galvanised steel, and zincalume coated steel, *Corrosion Australasia*, Vol. 13, No. 1, pp. 5-11.

The paper presents the results of a corrosion survey set up at 68 sites on a 2×2 km^2 grid in the south-west of Melbourne, Australia. An earlier test programme by the author was carried out over a 900 km^2 and used over 300 sites but included only a low alloy steel. In the programme presented here the corrosion rate of mild steel, low alloy copper steel, galvanised steel and zincalume (also called galvalume by others) steel are measured using small plate coupon specimens and CLIMAT testers. The testing site is fairly flat and westerly winds carry chloride at some distance inland. The effect of the distance from the sea was specifically investigated and it was found that the corrosion rate of steel and galvanised steel decreased dramatically within the first kilometre from the sea but very little change was observed beyond that distance up to 13 to 14 km. Regression equations, in the form of hyperbolic functions, expressing the corrosion rate in terms of the distance from the sea are given for the various metals tested in this programme. Galvanised steel was found to perform badly at beach front sites whereas Zincalume performed very well at those sites. The results of the study indicate that a severe marine environment is one which extends no more than 2 km from the coast. This is in contrast to the 7 km distance given the Australian Standards.

83 King, G.A., Dougherty, G.J., Dalzell, K.W. and Dawson, P.A. (1988) Assessing atmospheric corrosivity in Antarctica, *Corrosion Australasia*, Vol. 13, No. 5, pp. 13-5.

Although it has been generally believed that corrosion rates are very low in the Polar regions because of the low temperatures and low relative humidity, corrosion problems developed in huts built for shelter on Ross Island. In order to study the phenomenon, limited number of standard corrosion specimens of copper steel were exposed to the coastal site where corrosion problems had been encountered and an inland site. The corrosion rates observed at the coastal site where the average annual temperature is about -21°C was over ten times the rates observed at the inland site. Corrosion rates over the first year of exposure was about 10 µm/year and dropped to about 4,4 µm/year over a three year period. This was unexpected since the long-term rate is usually 60 % of the short term rate. The corrosion rate recorded at the inland site was very low (0,9 µm/year) due to the extreme dryness at the site. The relative humidity and temperature seem to be the only environmental parameters measured at the test sites. The corrosion rates observed at those sites are compared with corrosion rates observed by others at cold sites and dry sites. It was concluded that corrosion protection is required at coastal sites in Antarctica.

84 King, G.A. and Gibbs, P. (1986) Corrosivity mapping around a point source of pollution, *Corrosion Australasia*, Vol. 11, No. 6, pp. 5-9.

A previous survey of the corrosivity of the atmosphere around Melbourne, Australia, showed that an area of increased corrosivity existed around an industrial plant. The work presented in this paper is attempt to determine the extent of the influence of this source of pollution on the corrosion rate. The programme described here made use of a smaller grid covering an area 3 km around the plant. Contour lines of corrosion loss for copper steel were plotted both for this programme and the earlier programme for comparison. Very little resemblance was found between the results of both programmes. The later programme showed a significant reduction in corrosion. Survey of fuel consumption and operations at the plant revealed that the reduction in fuel consumption and possible increased downtime during the second period may be the cause. The programmes were only three years apart. A survey showed a more frequent precipitation during the second programme, thus indicating that rain fall is not correlated to corrosivity.

85 Knotkova, D., Barton, K. and Cerny, M. (1982) Atmospheric corrosion testing in Czechoslovakia, in *Atmospheric Corrosion*, (ed. W.H. Ailor), John Wiley and Sons, New York, pp. 991-1014.

A brief discussion on the use of various types of corrosion tests from long-term field tests to laboratory accelerated corrosion tests is presented. General principles for establishing and utilising test stations are also discussed. A brief presentation of station networks and sites description are presented. The central Czechoslovak station network is supported by the systematic research on atmospheric corrosion which has been carried out by the State Research Institute of Material Protection since the early 1950's. Meteorological and aerochemical measurements performed at the test stations enable the calculation of the time-of-wetness, taken as the time during which the relative humidity is greater than 80 % while the temperature is above freezing. Regression analysis of field exposure test results has shown that the time-of-wetness and SO_2 levels better characterise the aggressivity of open atmospheres for steel, zinc, and copper, and, in a less obvious manner, of an environment protected against the effects of precipitation.

86 Kucera, V. (1985) Influence of acid deposition on atmospheric corrosion of metals: a review, in *Materials Degradation Caused by Acid Rain, 20th State-of-the-Art Symposium of the American Chemical Society*, (ed. R. Baboian), ACS Symposium Series 318, pp. 104-18.

The author presents a brief review of the influence of acidifying air pollutants on the atmospheric corrosion of metals based mainly on more recent results from Europe and especially from Scandinavia. Research has shown the dominating effect of SO_2 on the corrosion rate of carbon steel and zinc. Dose-response functions, in terms of the SO_2 level only, have been derived based on corrosion rate measurements. The inclusion of time-of-wetness in the linear models does not always improve the correlation. The effect of NO_x alone on the corrosion of steel was found to be negligible. However, NO_x was found to increase significantly the rate of corrosion in the presence of SO_2 at a RH of 50 % but the synergistic effect was not observed at 90 % RH.

87 Kucera, V., Haagenrud, S., Atteraas, L. and Gullman, J.(1988) Corrosion of steel and zinc in Scandinavia with respect to the classification of the corrosivity of atmospheres, in *Degradation of Metals in the Atmosphere*, (eds. S.W. Dean and T.S. Lee), ASTM STP 965, American Society for Testing and Materials, Philadelphia, pp. 264-81.

Exposure test results on steel and zinc at 32 sites in Scandinavia are presented. The tests were performed on standard specimens. Measurements were taken of the air SO_2 concentration, precipitation SO_4 and chloride concentration, precipitation pH, temperature, and relative humidity. The time-of-wetness was defined as the time during which the relative humidity was greater than 80 % and the temperature above freezing. Statistical analysis was performed on the test results collected over a period of eight years for steel and over four years for zinc. A bilogarithmic relation between rate of corrosion and time was used for steel while a linear relation was used for zinc. The constants in the two models were found from a regression analysis to be functions of SO_2 and Cl^- concentration. The difference in corrosion rates observed between sites was found to be a result of the pollution level rather than climatic conditions. It was found that the corrosion rate was greater during the first and second year of testing at sites where precipitation was high. A system of classification of corrosivity was proposed based on levels of SO_2 and chlorides in the atmosphere.

88 Kucera, V. and Mattson, E. (1982) Atmospheric corrosion of bimetallic structures, in *Atmospheric Corrosion*, (ed. W.H. Ailor), John Wiley and Sons, New York, pp. 561-74.
The following parameters of importance are discussed: the electrode potential difference, the anodic and cathodic polarisation and the existence of passivating coatings, the distance between anodes and cathodes, the thickness of the moisture film on the surface, and the influence of atmospheric pollution. Some field test methods are described including the circular disk method, the wire-on-bolt method, combination of plates of dissimilar metals, and electrochemical test methods. Important published investigations are surveyed, and some charts, useful in materials selection, are presented.

89 Lashermes, M., Guilhaudis, A., Reboul, M. and Trentelivres, G. (1982) Thirty-year atmospheric corrosion of aluminium alloys in France, in *Atmospheric Corrosion*, (ed. W.H. Ailor), John Wiley and Sons, New York, pp. 353-64.
Various aluminium were tested at various sites such as severe marine, polluted urban, and clean rural. Standard plate specimens were exposed using standard exposure protocols. In addition the racks themselves made the object of some studies. The racks are made of aluminium angle sections attached together using gusset plates. The gusset plates were bolted using either anodised aluminium or bitumen coated steel bolts. The aluminium samples were of the 1000, 2000, 3000, 5000, and 6000 series. In general, only very light uniform corrosion took place. Corrosion took the form of pitting. Aluminium in the work hardened condition showed much better resistance to corrosion than annealed aluminium. Pure aluminium and manganese aluminium showed more corrosion resistance than the magnesium aluminium. The AlCu alloys showed the poorest performance because of the high risk of inter crystalline corrosion. The groundward surface of the plate samples corroded more than the skyward surface. The exposed racks showed good resistance to corrosion. The use of steel bolts was found to cause accelerated corrosion of the aluminium gusset plates due to galvanic action. In general, the test results showed that aluminium showed very little corrosion problem and can therefore be used without protection in marine environments.

90 Latta, J.K. (1962) *Water and Building Materials*, Canadian Building Digest, Division of Building Research, National Research Council, Canada, CBD 30.
The effect of water on various building materials is reviewed. Dimensional changes due to water absorption and water evaporation can result in large dimensional changes which can lead to failure of the material or rupture of the bond with its surroundings. Typical thermal and moisture expansions of various materials are given which show that moisture variation can be more detrimental on building materials than variation in temperature. The presence of water is also essential for corrosion of metals to take place. Water is also one of the agents essential for the decay of wood. Moisture contents less than 20 % is required to prevent rotting of wood. Blistering occurs when water is trapped between layers of built-up roofing and the heat from the sun causes the water to vaporise. Efflorescence results when water moves through a material and dissolves salts from some position inside the material and deposits them on the surface. Leaching of calcium from the calcium silicate of concrete or mortar can occur as a result of mildly acidic water moving through the concrete or mortar. Deterioration some building materials results from expansion of absorbed water upon freezing. The distribution of water and the nature and distribution of the voids have an important bearing on the resistance of a material to damage by freezing.

91 Legault, R.A. (1982) Atmospheric corrosion of galvanised steel, in *Atmospheric Corrosion*, (ed. W.H. Ailor), John Wiley and Sons, New York, pp. 607-13.
The author looks at the corrosion of zinc at three different exposure sites in the U.S.: semi-industrial, severe industrial, and marine. Corrosion rates were analysed separately on the skyward and the groundward sides of the exposure coupons. Damage functions were proposed for corrosion of zinc for the three different sites and for groundward and skyward exposure. The damage functions include only time of exposure as independent variable. The skyward corrosion loss is linear with time at the semi-industrial and severe industrial sites but non-linear with time at the marine exposure sites. The corrosion rate on the groundward side is non-linear at all the exposure sites. The corrosion loss on the skyward face of the test specimens was found to be significantly greater than that predicted on the groundward face. The effect of the steel substrate was found to be insignificant until it becomes exposed.

92 Lipfert, F.W. (1989) Atmospheric damage to calcareous stones: comparison and reconciliation of recent experimental findings, *Atmospheric Environment*, Vol. 23, No.2, pp. 415-29.

The author has compiled published data on the rates of deterioration of calcareous stones in the atmosphere and compared the various sets of data and the damage functions obtained from the various sets of data. The types of stone covered in this review are marble, some sandstones, and limestone. The author presents a brief description of the mechanisms of damage to carbonate stone which consists of deterioration from clean rain with the contribution from air pollutants such as SO_2 and HNO_3, the contribution from rain acidity. A general damage function based on theory is proposed. Damage functions are also derived from various exposure test programmes and various functions compared. The pitfalls of using regression analysis to derive damage functions are outlined. From the comparison of damage functions the author was not able to recommend any one statistically based damage function for general use. The type of stone, level of SO_2 and location of the exposure site were both found to be significant parameters. The height above the ground was found to be a significant parameter in some of the test programmes presented. The paper is very useful because it summarises some of the work that has been carried out to evaluate the effect of the environment on building stone and compares them. This paper, however, does not try to group all the various data together to form a larger data base from which a general damage function could be obtained. This was done by the author for steel and zinc.

93 Lipfert, F.W. (1987) Effects of acidic deposition on the atmospheric deterioration of materials, *Materials Performance*, Vol. 26, No. 7, July, pp. 12-9.

The author discusses the sources of acidic deposition, the general mechanisms by which acidic deposition acts to reduce material service life, the derivation of damage functions to predict such losses, and the potential economic consequences. It is estimated that 2/3 of the precipitation's acidity is attributed to sulphate and 1/3 with nitrate. Damage functions for zinc, copper, steel, aluminium, stone, and mortar are presented. The agents considered in the functions are: SO_2, time-of-wetness estimated from RH statistics, annual precipitation, precipitation pH, chloride ion deposition rate, and dust deposition rate. The importance of acidic deposition upon corrosion of various building materials was demonstrated.

94 Lipfert, F.W., Horst, R., Sherwood, S. and Lareau, T. (1986) The 1985 Acid precipitation materials damage assessment, in *Proceedings of the Symposia on Corrosion Effects of Acid Deposition and Corrosion of Electronic Materials*, (eds. F. Mansfeld et. al.), April, Vol. 86-6, pp. 10-45.

The cost of materials damage due to air pollution and precipitation acidity were estimated for 17 North-eastern states as part of NAPAP's 1985 current damage assessment. The materials evaluated included architectural metals, certain paints, mortar, and calcareous stones. The assessment was based on 5×5 km^2 grids for air quality and materials distribution. The annual costs were estimated between US$560 million to about US$6400 million. In order to arrive at a cost estimate of damage, damage functionswere obtained. Damage functions used to assess the deterioration rate of zinc, copper, aluminium, steel, paint, stone, and mortar. The damage functions for metals are based on site to site comparison and a generalised corrosion model. The function used for mortar is based on calcium carbonate dissolution only and losses due to freeze-thaw were ignored. The proposed damage functions do not account for all the damage mechanisms which can interact with pollutants. Details of the methodology used to estimate the distribution of materials across the survey area are presented. Local sulphur dioxide concentrations were estimated from analytical models and data on the types of emission sources. The time-of-wetness was also predicted from the measured relative humidity and it was found that it was correlated to the latitude, the density of housing, and proximity to the coast. The product of the sulphur dioxide concentration and the time-of-wetness was used in some of the damage functions as a representation of the deposition rate. The investigation showed that the deposition pattern of SO_2 and precipitation acidity are so widely distributed that local air pollution sources no longer dominate. The inventory of materials showed that the materials used for residential building are relatively predictable but considerable diversity was found for industrial and commercial buildings. Damage to paint was found to be the most costly followed by damage to mortar, galvanised steel, and building stone. Areas of needed research have been identified as: damage functions for real buildings, vertical surfaces and art objects; more research is needed to obtain damage functions for paints of various formulations; acid precipitation effects on plastics, polymers, reinforced concrete, and sealants are unknown.

95 Livingston, R.A. (1990) Improved prediction of masonry durability through advanced materials science methods, in *Proceedings of the 1st Materials Engineering Congress on Serviceability and Durability of Construction Materials, Denver, CO.*, 13-15 August, Vol. 1, pp. 482-90.

The paper emphasises the need to know and learn more about the durability of materials. The author suggests that the application of materials science should be useful to understand better the factors which affect the durability of masonry. No guidance is given on how to use materials science for prediction of durability or even to improve the predictions.

96 Loo, Y.H. and Ting, S.K. (1990) Effects of pulverised fuel ash on corrosion of steel in concrete, in *Proceedings of the 5th International Conference on Durability of Building Materials and Components, Brighton, 1990*, (eds. J.M: Baker, P.J. Nixon, A.J. Majumdar and H. Davies), E & FN Spon, U.K., 7-9 November, pp. 145-50.

The authors present the results of an experimental investigation of the effect of fly ash addition on the corrosion of steel in concrete. Small cylindrical mortar specimens with a 13 mm rebar placed at the centre of the specimens were used for the experiments. The results show the beneficial effect of fly ash addition on the corrosion resistance of the rebars after 91 days. The importance of the water to cement + fly ash ratio was demonstrated. The increased corrosion resistance was associated to the reduced pore sizes resulting when fly ash is added to the concrete mix.

97 Mangat, P.S. and Molloy, B.T. (1992) Factors influencing chloride-induced corrosion of reinforcement in concrete, *Materials and Structures / Matériaux et Constructions*, Vol. 25, No. 151, August/September, pp. 404-11.

The paper presents the results of a corrosion investigation on rebar electrodes embedded in concrete prisms which were exposed to cycles of sea-water spray for up to about 600 days. The experiments showed that the water/cement ratio is the dominant factor which controls rebar corrosion and a value of 0,45 is satisfactory for designing corrosion-resistant reinforced concrete. The cement content was found to have an insignificant influence on the corrosion resistance of rebar in concrete exposed to a marine environment. The chloride and hydroxide concentration of the concrete matrix and pore fluid were found to be of secondary importance in corrosion resistant concrete. A pore fluid Cl^-/OH^- ratio of 11 was found to be acceptable.

98 Masson, G.F. (1989) Sulfacrete – the preferred choice cement for durable foundation concrete and masonry, *Masonry International, Journal of the British Masonry Society*, Vol. 3, No. 1, May, pp. 31-3.

The author gives a description of Blue Circle Sulfacrete, a sulphate-resisting Portland cement manufactured and used in the U.K. The low tricalcium aluminate content (less than 2,5 % compared with 7-12 % in current ordinary Portland cements) confers sulphate-resisting properties. The rate of carbonation in concrete made with Blue Circle Sulfacrete and designed for durability in terms of BS8110: Part 1: 1985 and well cured, is similar to that in equivalent 28 day strength ordinary Portland cement concrete.

99 McGee, E.S. (1991) Influence of microclimate on the deterioration of historic marble buildings, *APT Bulletin, The Journal of Preservation Technol.*, V. 23, No. 4, pp. 37-42.

The author remarks that deterioration of marble in buildings is not uniform. This is not only dependent on the variations in chemical, mineralogical and physical properties of the stone but also on micro-climatic effects. Deterioration of marble is said to be the result of dissolution of calcite in acid rain. Although rain is believed to be a major cause of marble deterioration, a survey of existing buildings show that the problem also exists in sheltered areas. In sheltered areas, pollutants that are not regularly washed off the stone may react with moisture and calcite to form a new mineral phase on the surface (mostly gypsum). The subsequent deterioration of the marble can depend on whether the gypsum layer can be washed off by rain. A case study of the Philadelphia Merchants' Exchange building is presented. Deterioration of marble on this building was found to be strongly dependent on the exposure to rain, sun, and wind. Although no results are presented in this paper, the author mentions that instrumentation has been placed on the building to monitor the differences in the temperature, relative humidity, and dry deposition between different points on the building.

100 Meakin, J.D., Ames, D.L. and Dolske, D.A.(1991) Degradation of monumental bronzes, *APT Bulletin, The Journal of Preservation Technology*, Vol. 23, No. 4, pp. 58-63.

A study of the degradation rate of bronze used for statues was conducted. The Hiker statue, commemorating the Spanish-American War veterans, was replicated more than 52 times and have been placed at different locations across the U.S. After having established that the metallurgical composition of the replicas was essentially the same, the statues were used as test samples. Since uniform corrosion loss was expected to be too small to make any significant comparison between the statues, depth of pit measurements, made on vinyl polysiloxane replicas, were used to compare the severity of deterioration of the different statues. A comparison between different statues showed that the degree of attack is significantly different between different sites. A surface runoff study from bronze markers of composition similar to that of the statues was conducted at Gettysburg National Military Park. The results of the runoff water analysis showed that dissolution of the bronze occurs without preferential removal of zinc from copper. The data showed a strong correlation between the amount of sulphate and dissolved copper. The correlation between copper and nitrates in the runoff water was not as good but a correlation seems to exist.

101 Mikhailovsky, Y.N. (1982) Theoretical and engineering principles of atmospheric corrosion of metals, in *Atmospheric Corrosion*, (ed. W.H. Ailor), John Wiley and Sons, New York, pp. 85-105.

The nature of atmospheric corrosion is discussed as a combination of corrosion and electrochemical processes on metals beneath absorbed and thin phase layers of electrolytes. The effects of oxygen absorption, water molecule absorption, sulphurous gas absorption, and aerosols of salts are considered. The author makes use of field exposure test data from COMECON, which includes test sites in USSR and eastern Europe, to derive values of constants terms to use in proposed damage functions. The factors found to be significant in the corrosion of various metals and alloys are: time-of-wetness, temperature (for steel only), sulphur dioxide concentration, chlorides, and sheltering. The effect of temperature on corrosion of steel was found to be important both on the corrosion rate in an unpolluted atmosphere and on the corrosion rate adjustment for presence of sulphur dioxide. The effect of temperature was found to be insignificant on the effect of chlorides. The damage functions proposed for sheltered and boldly exposed conditions are interesting but the terms accounting for chloride and sulphur dioxide pollution are not well defined and it is not clear how those parameters are obtained. Consequently, the paper is more useful for a qualitative assessment rather than a quantitative assessment of corrosion agents.

102 Mikhailovsky, Y.N. and Strekalov, P.V. (1982) Atmospheric corrosion tests in the USSR, in *Atmospheric Corrosion*,(ed. W.H. Ailor), John Wiley and Sons, New York, pp. 923-42.

Atmospheric corrosion tests have been conducted in the former Soviet Union since the 1950's. The authors present a description of the test sites, the instrumentation used, and the type of data collected at the exposure sites. The corrosion tests are performed under exposed and sheltered conditions. Although most of the paper describes the types of atmospheric conditions at various test sites, some corrosion test observations are also presented. The rate of corrosion of steel in the open atmosphere was found to decrease with time whereas the rate of corrosion in a semiclosed atmosphere was found to be almost constant. Zinc was characterised by linear corrosion kinetics. In the sheltered environment the zinc corrosion was found to grow linearly with time at a much smaller rate (5-7 times) than in the open air. For aluminium, it was found that the corrosion rate was greater in the sheltered environment than in the open atmosphere. This was accounted for by the fact that in a sheltered environment atmospheric corrosive agents accumulate on the specimen surfaces. In the open air these agents are periodically washed away by rain.

103 Millard, S.G. and Gowers, K.R. (1991) The influence of surface layers upon the measurement of concrete resistivity, in *2nd International Conference on Durability of Concrete, Montreal, Canada*, (ed. V.M. Malhotra), ACI SP-126, Vol. 1, pp. 1197-220.

The measurement of electrical resistivity of concrete is a non-destructive technique used to assess the severity of corrosion of reinforcing steel. A study of the effect of the surface layer resistivity, as affected by the wetting and carbonation of the surface is presented. A theoretical analysis using classical treatment of the Laplace equation and a Finite Element Analysis were performed to investigate the effect of carbonation or wetting of the surface layer (a single surface layer) or the combined effect of wetting and carbonation (double layer). Analytical results were compared with physical measurements to validate the analysis. It was found that a high resistivity surface layer, formed by carbonation of the surface, has relatively little effect on the measurement of the underlying concrete resistivity as long as the carbonation thickness does not exceed 0.2 times the resistivity electrode spacing. Taking resistivity measurements upon a carbonated surface that has recently been wetted does however cause major difficulties. As shown by lab tests and finite element analysis, the wet surface layer can cause the resistivity measurement to increase by a factor of two to ten times. Taking resistivity readings on uncarbonated concrete that has a low resistivity surface moisture layer can also give rise to significant errors in the assessment of the underlying resistivity.

104 Millard, S.G., Gowers, K.R. and Gill, J.S. (1991) Reinforcement corrosion assessment using linear polarisation techniques, in *Proceedings of ACI International Conference on Evaluation and Rehabilitation of Concrete Structures and Innovations in Design, Hong Kong*, (ed. V.M. Malhotra), ACI SP-128, Vol. 1, pp. 373-94.

While potential mapping is commonly used to indicate regions of corrosion activity, it cannot be used to assess the rate of corrosion. The authors have demonstrated that perturbative measurements of direct corrosion rates could be used in the field to asses the rate of corrosion. Linear Polarisation Resistance (LPR) instrumentation, customised for reinforced concrete problems, can clearly indicate where significant corrosion is in progress and give as much information about the rate of corrosion as AC impedance.

105 Moresby, J.F., Reeves, F.M. and Spedding, D.J. (1982) Atmospheric corrosion testing in Australasia, in *Atmospheric Corrosion*, (ed. W.H. Ailor), J. Wiley and Sons, N.Y., pp. 745-54.

The authors describe the atmospheric corrosion research being pursued in Australia, New Zealand, and their adjacent islands. A brief description of the climatic conditions and atmospheric impurities found in various parts of Australasia is given. Field corrosion tests have been established since 1960, and the methods used are based on the U.K. or U.S. practice in almost all respects. BISRA standard copper steel samples are used by the Commonwealth Scientific and Industrial Research Organisation (CSIRO) to standardise test sites. The Joint Tropical Trials and Research Establishment (JTTRE) formed by the British and Australian governments has tested service equipment in Queensland. In 1979 investigators started an intensive corrosivity survey of 900 km^2 of the Melbourne metropolis. Coupons are suspended 3,7 m above ground and are on a 2×2 km^2 grid. The aim is to plot corrosivity map to aid the local electricity authorities, engineers, and builders to select materials of sufficient corrosion resistance.

106 Nagataki, S. and Ohga, H. (1992) Combined effect of carbonation and chloride on corrosion of reinforcement in fly ash concrete, in *Proceedings of the 4th International Conference on Fly Ash, Silica Fume, Slag, and Natural Pozzolans in Concrete, Istanbul, Turkey,* (ed. V.M. Malhotra), ACI SP-132, May, Vol. 1, pp. 227-44.

Mortars with and without fly ash (fly ash replaced some cement) were cured in distilled water or NaCl solutions for varying lengths of time. The samples were then exposed to accelerated carbonation. The influence of chloride ion on the depth of carbonation was thus evaluated. Samples were exposed to accelerated carbonation and then immersed in NaCl solution to study the effect of carbonation on chloride ingress. The penetration depth of chloride ion in fly ash mortar immersed in NaCl solution was found to be larger at an early age, but later became almost the same as that of the mortar without fly ash. The depth of carbonation of mortar cured in NaCl solution was found to be smaller than that in distilled water and the same trend was observed independent with initial curing time and the addition of fly ash. Fly ash mortar showed higher depth of carbonation than the control mortar. The corrosion potential of the steel reinforcement was found to be affected by both the chloride penetration and carbonation.

107 Noland, J.L. and Atkinson, R.H. (1983) Evaluation of brick masonry by non-destructive methods. strengthening of building structures – diagnosis and therapy, in *IABSE Symposium*, Venezia, Vol. 46, pp. 85-92.

The authors present an investigation of various non-destructive testing methods and their potential application to masonry. The various non-destructive technique test results were correlated to mechanical properties such as compressive strength, modulus of rupture and shear strength. The techniques were evaluated in terms of how well they could predict the mechanical properties. Although the study was inconclusive, certain methods appeared to have potential as practical means of evaluating masonry strength. The Schmidt Rebound Hammer and the ultrasonic pulse velocity were more closely related to the compressive strength and modulus of rupture, respectively. None of the techniques seems to be able to predict joint shear strength.

108 Oelsner, G. (1982) Atmospheric corrosion testing in the Federal Republic of Germany, in *Atmospheric Corrosion*, (ed. W.H. Ailor), John Wiley and Sons, New York, pp. 797-805.

The results of field tests for 12 different aluminium alloys under atmospheric conditions for a period of ten years are presented. In general the material loss due to corrosion and pitting depth were found to be more severe on the specimens inclined at 45° than the vertical test specimens. The underside of the inclined test specimens were found to corrode more than the vertical specimens. Anodised and polished samples were also tested. The samples placed in an industrial atmosphere were found to corrode more than those placed in a coastal area. The correlation between the composition of the atmosphere and the observed corrosion was not studied in this work.

109 Ohta, T. (1991) Corrosion of reinforcing steel in concrete exposed to sea air, in *2nd International Conference on Durability of Concrete, Montreal, Canada*, (ed. V.M. Malhotra), ACI SP-126, Vol. 1, pp. 459-77.

Long term exposure test results of reinforced concrete beams are reported. Test samples were exposed to sea air for 2 to 20 years. The parameters were thickness of concrete cover, type of cement, cement content, and crack width. The type of cement had a great influence on the depth of chloride penetration. The diffusion coefficient of chloride ion was found to be 3-4 times lower in blended cements than in ordinary portland cement. The thickness of the concrete cover was found to have a very important effect on the corrosion protection of reinforcing steel. The influence of crack width was found to be negligible for thin concrete covers whereas it had some influence on the corrosion for thick concrete covers (40 mm). The use of epoxy coating on the surface of the concrete was found to be effective in reducing corrosion of the reinforcing steel.

110 Ohtsu, M. and Yuno, K. (1991) Development of in-situ non-destructive evaluation (NDE) techniques for rebar corrosion, concrete deterioration and internal cracks, in *Proceedings of ACI International Conference on Evaluation and Rehabilitation of Concrete Structures and Innovations in Design, Hong Kong*, (ed. V.M. Malhotra), ACI SP-128, Vol. II, pp. 893-907.

The authors present in situ non-destructive techniques for estimating rebar corrosion, crack depth, deterioration due to microfracturing, and kinematics of crack nucleation in concrete structures. An improved half-cell potential technique is presented for monitoring of rebar corrosion. The depth of surface crack is evaluated by ultrasonic spectroscopy. The amount of microcracks associated with the deterioration is estimated from acoustic emission activity during a uniaxial compression test of core sample.

111 Oldfield, J.W. and Todd, B. (1990) Ambient-temperature stress corrosion cracking of austenitic stainless steel in swimming pools, *Materials Performance*, Vol. 29, No. 12, pp. 57-8.

The authors report instances of stainless steel SCC when exposed to swimming pools atmosphere. Although it is generally accepted that SCC of SS does not occur at temperatures below 60°C, several cases of failure in ambient environment around swimming pools have been reported. The Nickel Development Institute, studying the problem, has demonstrated that SCC of austenitic SS can take place at room temperature in chloride solutions of very low pH. Such condition however, does not seem to prevail in the environments where failures by SCC took place. Until the mechanisms of attack are clarified, use of type 304 and 316 (titanium stabilised) stainless steel for stressed components exposed to swimming pool atmospheres cannot be recommended.

112 Papadakis, V.G., Fardis, M.N. and Vayenas, C.G. (1990) Fundamental concrete carbonation model and application to durability of reinforced concrete, in *Proceedings of the 5th International Conference on Durability of Building Materials and Components, Brighton, 1990*, (eds. J.M: Baker, P.J. Nixon, A.J. Majumdar and H. Davies), E & FN Spon, U.K., 7-9 November, pp. 27-38.

The physicochemical processes of concrete carbonation are modelled mathematically. The purpose of the work was to cover some of the shortcomings of empirical carbonation models developed by other researchers. The mathematical model was used to predict the carbonation front with respect to cement paste composition and environmental conditions. A good correlation between laboratory results and prediction from the mathematical model was observed.

113 Philipose, K.E., Feldman, R.F. and Beaudoin, J.J. (1991) Durability predictions from rate of diffusion testing of normal portland cement, fly ash, and slag concrete, in *2nd International Conference on Durability of Concrete, Montreal, Canada*, (ed. V.M. Malhotra), ACI SP-126, Vol. 1, pp. 335-54.

The authors present the results of research in progress to establish the diffusion rates of chlorides and sulphate ions. The research programme has for its first objective the qualification of a concrete with a minimum service life of 500 years used for nuclear waste containment. Fly ash and slag systems (i.e. addition of fly ash and slag) indicated superior resistance to ionic ingress, and a decrease in water-to-cement ratio enhances the resistance even further. The slag system seemed to have the highest resistance to ionic ingress. The rate of ingress of chloride ions appeared to be diffusion controlled. The decrease in chloride content with depth of ingress was found to be more rapid with slag systems than others. The rate of ingress of sulphate ions was found to be considerably lower than chloride ions for all systems. Linear regression analysis of the limited data was used to predict the depth of penetration of the chloride ions in 500 years.

114 Raharinaivo, A., Brevet, P., Grimaldi, G. and Pannier, G. (1986) Relationships between concrete deterioration and reinforcing steel corrosion, *Durability of Building Materials*, Vol. 4, No. 2, pp. 97-112.

An investigation of the influence of crack geometry on concrete reinforcement corrosion is presented. Tests on reinforced mortar specimens were performed to show the effect of the cement-paste deterioration on the reinforcing steel corrosion. Tests on reinforced concrete specimens were conducted to determine the effect of aggregates on the diffusion of aggressive ions through the concrete. The specimens were cracked after preparation by subjecting the specimens to bending. Corrosion conditions were monitored during the tests using electrode potential measurements. The crack opening was found to have the greatest influence on steel corrosion for a given cover thickness. Corrosion of the steel in the samples was concentrated in the regions close to cracks.

115 Rasheeduzzafar, Dakhil, F.H., Bader, M.A. and Khan, M.M. (1992) Performance of corrosion resisting steels in chloride-bearing concrete, *ACI Materials Journal*, Vol. 89, No. 5, September-October, pp. 439-48.

Bare mild, galvanised, epoxy-coated, and stainless clad reinforcing steels have been evaluated in a seven-year exposure site programme for corrosion resistance performance in chloride-bearing concretes. The parameters investigated were the type of reinforcing steel and the chloride content. The test specimens consisted of concrete prisms with embedded rebars. The bare steel and the galvanised steel performed rather poorly in the test. The level of chloride in the mix was found to have a significant influence on the corrosion of the bare and galvanised steel. The corrosion rate was found to be greater in the concretes with the highest chloride contents. Epoxy coated rebars have corroded only in the specimens with the highest chloride content. The stainless clad reinforcing steel showed no sign of corrosion over the seven year period. Several references on corrosion of galvanised steel in concrete are presented.

116 Reddy, M.M. and Youngdahl, A. (1987) Acid rain and weathering damage to carbonate building stone, *Materials Performance*, Vol. 26, No. 7, pp. 33-6.

This paper presents the results of standard site exposure tests on marble and limestone at four American sites. Several methods were used to assess stone damage: chemical analysis of run-off water; measurement of surface contour and roughness changes; and weight loss. The paper presents an assessment of the three methods of assessing damage to building stones. All three methods yielded similar results in the assessment of stone material loss. The paper does not give any detail of measured precipitation chemistry. The runoff analysis, however, showed that a relation exists between rain pH and material loss. More details of the test programme have been used in a paper by Lipfert (1989) where damage functions from this test programme have been assessed.

117 Roper, H. and Baweja, D. (1991) Carbonation-chloride interactions and their influence on corrosion rates of steel in concrete, in *2nd International Conference on Durability of Concrete, Montreal, Canada*, (ed. V.M. Malhotra), ACI SP-126, Vol. 1, pp. 295-315.

The authors present the results of an investigation of corrosion of reinforcing steel subjected to the combined effect of a chloride environment and carbonation of concrete. Studies on interrelationships between carbonation and chloride ion ingress showed that carbonation rates increase with increasing water/cement ratios. Replacement of cement by fly ash did not detrimentally affect the carbonation resistance of concrete and the addition of chloride ions to the concrete mixes tended to reduce the carbonation of fly ash concretes. Potentiodynamic anodic polarisation testing of corroding reinforcement revealed that both carbonation and chloride ion concentration increase corrosion rate. It was found that corrosion caused by carbonation increased with increasing chloride concentration provided that the carbonation rate was itself not retarded by the presence of chlorides. This leads to the conclusion that corrosion rate is significantly influenced by complex interactions due to the presence of carbon dioxide and chloride ions.

118 Rostam, S. (1989) Influence of material properties on the durability of structure, in *Durability of Structures, IABSE Symposium, Lisbon*, 6-8 September, Vol. 57/1, pp. 125-36.

The influence of material properties on the durability of structures is an integrated part of the interaction between environmental aggressivity and the resistance of the structure. Durability is treated in a Service Life concept. The service life achieved depends on initial decisions, codes and standards and on management and maintenance systems employed. The design of new structures is guided by codes and standards. The CEB-FIP Model Code 1990 is the first Service Life Code and helps bridge the communication gap existing between the material scientists and the structural engineers.

119 Sandberg, L.B. (1991) Comparisons of silicone and urethane sealant durabilities, *Journal of Materials in Civil Engineering*, ASCE, Vol. 3, No. 4, Nov., pp. 278-91.
The results of a two-year ageing study on four sealants are presented. The four sealants are: a low-modulus silicone, a medium-modulus silicone, a one part polyurethane, and a two part polyurethane. The test specimens consisted of two small pieces of red granite bonded by a 12 mm bead of sealant. The specimens were exposed to various environments: ordinary room conditions, outdoors in a temperate climate, outdoor in a subtropical climate, and in a QUV accelerated weathering unit. The endurance of the sealants was evaluated in terms of load displacement capacity and surface cracking and dirt pickup. It was found that visible changes may be as good a predictor of service performance as mechanical property data. The low-modulus silicone showed the best performance and showed no evidence of degradation other than dirt pickup. Similar performance was also observed from the medium-modulus silicone. The one part urethane showed the worst performance because it was too stiff and was unable to withstand cyclic straining. The two part urethane suffered rapid deterioration, in the form of cracking, when exposed to UV and outdoor conditions. It is suggested that QUV exposure could be used as an accelerated test for silicone and 5 000 hours of QUV exposure is believed to be equivalent to ten years or more of field exposure.

120 Saricimen, H., Al-Tayyib, A.J., Maslehuddin, M. and Shamim, M. (1991) Concrete deterioration in high chloride and sulphate environment and repair strategies, in *Proc. of ACI Intl. Conf. on Evaluation and Rehab. of Concrete Structures & Innovations in Design, Hong Kong*, (ed. V.M. Malhotra), ACI SP-128, Vol. I, pp. 19-33.
The authors claim that standards of practice of reinforced concrete in temperate regions are inadequate for the climatic conditions existing in Saudi Arabia. Observations from field inspection of deteriorated underground electrical power manholes and sea water cooling canals are reported. The concrete cover provided is reported as a nominal value only. Chloride penetration measurements performed in a lab show that an extremely high chloride content exists 60 mm below the surface. The pH of the deteriorated concrete was from 12,6 to 13,2, indicating insignificant carbonation in the concrete. The authors did not report the strength of the concrete, the salt content of the aggregates used in the concrete, and the permeability of the concrete. (It is common in middle eastern countries to use aggregates containing significant amounts of chlorides. This could explain the high concentration of chlorides measured in the field samples. The authors failed to report the relevant information which could have pointed out either the inadequacy of foreign standards of practice for middle eastern countries or the generally poor quality control exercised).

121 Sarkar, S.L., Chandra, S., and Rodhe, M. (1992) Microstructural investigation of natural deterioration of building materials in Gothenburg, Sweden, (refer to section D).

122 Sereda, P.J. (1986) *Assessment of the State of Knowledge on the Long-Range Transport of Air Pollutants and Acid Deposition – Part 6: Effects on Man-Made Structures*, Federal Provincial Research and Monitoring Coordinating Committee (report prepared for Environment Canada, Downsview, Ontario).
Deterioration of building materials results from the action of several factors. The author discusses what part of the observed deterioration can be attributed to acid precipitation. After reviewing the research work being conducted to assess the effect of various pollutants on the deterioration of metals, coatings, and cementitious materials, the author concludes that the mechanisms of failure are not well known and the contribution from acid deposition still remains to be quantified. The author believes that the existing knowledge of deterioration of building materials from standard exposure tests can be used provided that the environment to which building components are exposed is characterised. The TOW, SO_2, NO_x and other particulate matter deposition rates are believed essential factors. For organic materials, UV irradiation and the number of wetting-drying cycles are also believed to be important. A review of various methods available to measure levels of pollutants is presented. An outline of research needs is given and .the assessment of the micro environment in buildings is identified as the most significant research need. Time-of-wetness, pollutants deposition flux and chloride deposition on building elements need to be evaluated and correlated to standard field exposure tests. Surveys if the extent of deterioration and corrosion of selected classes of materials on buildings should be undertaken coupled with statistics of life-cycle performance and maintenance costs.

123 Sereda, P.J. (1969) *Performance of Building Materials*, Canadian Building Digest, Division of Building Research, National Research Council of Canada, CBD 115.

Durability is viewed as a basic property of the materials which can be measured as the length of time it will serve satisfactorily. Durability, or service life, must always be related to the particular conditions involved. Materials react with their environment because of their chemical and physical and mechanical nature. It is its chemical nature that determines the reactivity of a material to other materials and to some elements of the environment. To understand the physical and mechanical response of a material we must think of the material in terms of its microstructure. In that respect we can differentiate between non-porous materials (metals, ceramics, polymers) and porous materials which offer more surface of contact with the atmosphere. The environmental factors which influence the performance of materials are briefly presented. Those are: temperature (the temperature inside the material rather than the air temperature must be considered), moisture content and its cycling, ultraviolet light which can affect the molecular bonds in organic materials, and gases, liquids, bacteria and other life forms. Various chemical and physical processes which affect the service life of materials are introduced such as ageing, efflorescence, frost action, wetting and drying and chemical attack.

124 Sereda, P.J. and Hechler, J.-J. (1989) *Assessment of the State of Knowledge on the Long Range Transport or Air Pollutants and Acid Deposition Update 1989 – Part 7 : Effects on Man-Made Structures*, Publication No. 31075, Industrial Materials Institute, National Research Council Canada.

This report is an update of an earlier report prepared by P.J. Sereda for the Federal/Provincial Research and Monitoring Coordinating Committee. The report is based on a review of the literature pertaining on the effect of pollution on structures and materials published between 1986 and 1989. A review of the important conferences and symposia held in that time period is presented. A summary of the published evidence of the effect of acid precipitation on corrosion of metals, deterioration of protective coatings and plastics, and deterioration of cementitious materials is presented. Other sections of the report summarise the mechanisms of reactions between the acid depositions and materials, discuss the predictability of the effects of acid deposition on materials and the damage assessment. Finally, a discussion of research needs is presented. The characterisation of the micro-environment in buildings and its relation to pollution levels is the main recommended research need. The authors also recommend that a survey of existing buildings should be conducted to determine the degree of risk that various materials experience. This information is believed to be essential to establish the priority of future research work. This is a very useful and informative report which addresses specifically the effect of precipitation acidity on building materials. Three different categories of materials are covered, namely, metals, coatings, and cementitious materials such as concrete and stone.

125 Sherwood, S.I. and Dolske, D.A. (1991) Acidic deposition and marble monuments at Gettysburg National Military Park, *The Journal of Preservation Technology, APT Bulletin*, Vol. 23, No. 4, pp. 52-7.

The impact of acidic precipitation on marble monuments was measured over a two year period from 1986 to 1988. The deterioration of monuments was evaluated from water runoff analysis. The calcium removed from the surface was measured by chemical differences between incident rain and monument runoff. The results showed that marble loss from complex, sculptural marble surfaces is several times greater than dissolution from flat, vertical materials of comparable petrography and similar exposure history. It is therefore recommended that weathering rates based on simple geometries need to be adjusted when applied to structures and monuments to take into account the increased erosion associated with greater geometric complexity. The shape and texture of the surface were found to influence the rate of dry deposition significantly. It is not clear from the paper whether the loss of calcium was measured for equal exposed surface area for different monuments.

126 Skerry, B.S., Johnson, J.B. and Wood, G.C. (1988) Corrosion in smoke, hydrocarbon and SO_2 polluted atmospheres – I. General behaviour of iron, *Corrosion Science*, Vol. 28, No. 7, pp. 657-95.

The effect of atmospheric pollutants on the corrosion characteristics of iron foil specimens is presented and discussed. The results of laboratory tests under controlled atmosphere have shown that: 1) sulphur dioxide significantly enhances corrosion in air; 2) smoke enhances corrosion in air; 3) the effect of ethane and acetylene in air is only marginal while ethylene has no apparent effect; 4) smoke worsens the effect of sulphur dioxide but sulphur dioxide appears to have the overall controlling influence; 5) ethane greatly enhances the effect of sulphur dioxide and smoke. The morphological effect of SO_2 was the change from relatively localised attack to complete coverage by corrosion products. Other interaction effects between the various pollutants have been observed. The authors present interesting discussions of the effects observed in the laboratory in terms of the chemical activity taking place on the surface of iron and the appearance of the corrosion products. Chemical processes taking place during corrosion of iron in the presence of atmospheric pollutants are proposed.

127 Skerry, B.S., Wood, J.C., Johnson, J.B. and Wood, G.C. (1988) Corrosion in smoke, hydrocarbon and SO_2 polluted atmospheres – II. Mechanistic implications for iron from surface analytical and allied techniques, *Corrosion Science*, Vol. 28, No. 7, pp. 697-719.

The paper presents the results of an experimental investigation using Auger electron spectroscopy (AES) and X-ray photo-electron spectroscopy (XPS) to study the composition of corrosion products on the surface of iron foils. Using mass spectroscopy and argon etching, the depth of penetration and profile of the corrosion layer, sulphur and carbon penetration were analysed. The tests showed that both oxygen and sulphur are distributed throughout the corrosion layer. It was found, however, that carbon does not penetrate beyond the surface of the corrosion layer. When corrosion takes place in the presence of SO_2, corrosion products on the surface of the corrosion layer occur as sulphates whereas sulphur species in the lower oxidation state (sulphides) are detected deeper in the corrosion layer. Other interesting findings with respect to the influence of hydrocarbons and smoke have been made which explained earlier observations by the authors about the effect of smoke and hydrocarbons on corrosion.

128 Skerry, B.S., Johnson, J.B. and Wood, G.C. (1988) Corrosion in smoke, hydrocarbon and SO_2 polluted atmospheres – III. The general behaviour of zinc, *Corrosion Science*, Vol. 28, No. 7, pp. 721-40.

Tests were performed on specimens of zinc foil exposed to various environments. Specimens were exposed in a modified industrial atmosphere test chamber to an air-based environment at 298 K with a relative humidity of 85 % and a CO_2 concentration of 5 500 ppm. Additions were made of SO_2, ethane, ethylene and acetylene and smoke. SO_2 was found to have a large and steadily increasing corrosion enhancement effect. Smoke was lightly corrosive to zinc and also promoted to some extent corrosivity in the presence of SO_2. Smoke in the presence of SO_2 represented the most corrosive test environment. Hydrocarbons only affected zinc corrosion marginally. However, when combined with SO_2 or smoke, hydrocarbons were found to affect significantly the corrosion process. Hydrocarbons were found to have an inhibiting or enhancing effect. The effects of smoke and some hydrocarbons are considered to be due to factors such as sorption, catalytic activity, interaction with moisture.

129 Southwell, C.R. and Bultman, J.D. (1982) Atmospheric corrosion testing in the Tropics, in *Atmospheric Corrosion,* (ed. W.H. Ailor), John Wiley and Sons, N.Y., pp. 943-967.

The authors discuss corrosion testing of bare metals at various tropical locations and attempt to evaluate the relative corrosiveness of different tropical atmospheres where temperature and relative humidity can be significantly greater than for temperate regions. The authors present a brief review of the work performed on tropical corrosion since the early 1950's. They also report on exposure studies conducted by ASTM at U.S. Naval Research Laboratory in Panama. The work included 54 metals and alloys among which six were pure metals, 24 were ferrous metals, 20 were non-ferrous alloys, and four were non-ferrous-coated steels. Two exposure sites were utilised: a site located 8 km inland from the pacific ocean in a semiurban location, and a marine site located near the Atlantic ocean. Limited exposure sites were also set at two other locations. Exposure periods were for 16 years. Tension tests were also run on the exposed specimens to determine any change in strength that might have taken place with corrosion. Although not very conclusive, the tensile tests on steel showed that the reduction in strength is directly proportional to the loss in cross-section area due to corrosion. Although the corrosion rates at the inland site were much lower than for the marine site during the first two years, the difference was significantly smaller on the long-term. The stabilised corrosion rates were used as a basis of comparison. The tropical open inland sites showed corrosion losses equal to or slightly higher than the U.S. rural site in Pennsylvania. The authors also present the result of galvanic corrosion obtained from coupling dissimilar metals.

130 Spence, J.W. and Haynie, F.H. (1988) *Theoretical Damage Function for the Effects of Acid Deposition in Galvanised Steel Structures*, Environmental Protection Agency, EPA/600/S3-88/027.

The authors present the development of a theoretical damage function for the prediction of the corrosion of galvanised steel structures by wet and dry deposition. The function was developed from thermodynamics and kinetics of atmospheric corrosion chemistry. The function expresses the corrosion rate of zinc in terms of the competing rate of build up and dissolution of the protective zinc carbonate with exposure time. The principal findings of the theoretical analysis are : SO_2 on the surface (either from dry or wet deposition) reacts stoichiometrically (reaction is proportional to the amount of SO_2 present) with zinc during periods of surface wetness; rain acidity reacts stoichiometrically with the zinc (reaction is proportional to the amount of dissolved H^+ in rain); the corrosion film of zinc carbonate is soluble in clean rain and the dissolution depends on the residence time of the rain on the surface; deposition velocity controls the rate of corrosion of galvanised steel structures by gaseous SO_2 during periods of wetness. Lab and field tests are in progress to evaluate the theoretical damage function.

131 Sussex, G.A.M., Scantlebury, J.D. and Johnson, J.B. (1984) Atmospheric corrosion and its accelerated testing, in *Proceedings of the 9th International Congress on Metal Corrosion, Toronto*, 3-7 June, Vol. 3, pp. 192-7.

The paper presents a literature review on atmospheric corrosion of metals. The first part of the paper reviews the important environment agents causing corrosion. These agents are water, oxygen, sulphur dioxide, chloride, particulate material. Other physical conditions such as temperature, light, sample geometry are also presented. The second part of the paper presents some empirical expressions for the time-of-wetness and corrosion rates of zinc, mild steel and weathering steel (about four functions are presented each for zinc and steel). The author states that those functions are essentially site specific and they conform in a general way with mechanistic ideas of corrosion, but they do not assist in understanding the processes. Finally, the author discusses the use of accelerated tests and present the various test methods used by researchers in that field. Wet/dry cycling tests are given particular attention since they are believed to represent more closely corrosion conditions in the atmosphere.

132 Swamy, R.N. and Tanikawa, S. (1991) Control of steel corrosion in chloride contaminated concrete through ARON WALL surface coating, in *2nd International Conference on Durability of Concrete, Montreal, Canada*, (ed. V.M. Malhotra), ACI SP-126, Vol. 1, pp. 371-91.

The authors present data on the performance of an acrylic rubber coating called ARON WALL in a corrosive environment. The information presented in this paper is essentially the same as the information presented in the paper summarised below.

133 Swamy, R.N. (1990) In-situ behaviour of galvanised reinforcement, in *Proceedings of the 5th International Conference on Durability of Building Materials and Components, Brighton, 1990*, (eds. J.M: Baker, P.J. Nixon, A.J. Majumdar and H. Davies), E & FN Spon, U.K., 7-9 November, pp. 299-312.

The author addresses the considerable controversy which exists with regard to the performance of galvanised rebars in reinforced concrete. While some studies show substantially improved durability over uncoated steel bars, others appear to indicate slightly better, equal, or sometimes even worse performance. In general, while some field and laboratory test results show that zinc coated reinforcement does not perform well, field experience contradicts such evaluations. Tests have shown that in dilute NaOH the minimum corrosion rate of zinc occurs at a pH of about 12,5. However, there seems to exist a threshold pH of 13,35±0,10 which defines the onset of active corrosion in galvanised steel. Although pore solution studies have shown that pore solution pH can exceed the threshold value, cores taken from structures exposed to the ambient environment show pH values less than the critical threshold value, providing a stable environment for galvanised steel. The author believes that the test solutions used for laboratory experiments are not representative of the pore solution found in concrete exposed to the atmosphere. Tests have shown that, as opposed to black steel which shows an increased rate of corrosion in carbonated concrete, the zinc coating on galvanised steel remains passive in carbonated concrete. By correlation between lab and field tests the author concludes that galvanised/chromated bars have excellent corrosion resistance in concrete with high levels of chloride. The author presents interesting references on corrosion of zinc and galvanised steel in concrete.

134 Swamy, R.N. and Tanikawa, S. (1991) A highly elastic surface coating to protect structures exposed to coastal/marine environments, in *Proceedings of ACI International Conference on Evaluation and Rehabilitation of Concrete Structures and Innovations in Design, Hong Kong*, (ed. V.M. Malhotra), ACI SP-128, Vol. I, pp. 1-18.

The authors present data on the performance characteristics of a highly elastic acrylic rubber coating called ARON WALL coating. The coating consists of a primer (chlorinated resin), a base coat (soft monomer) and a top coat (acrylic resin, acrylic urethane resin, or acrylic silicone resin) for an overall thickness of 1,1 mm. The coating was tested for its ability to bridge cracks, fatigue resistance, weathering resistance, resistance to ozone, UV exposure, temperature resistance, adhesion strength, carbonation depth, and chloride penetration. The coating showed a good performance in all areas. It should be noted that the maximum time of exposure to outdoor atmosphere is ten years. Seventeen references on the performance and use of various surface coatings are presented.

135 Tarricone, P. (1993) Plastic potential, *Civil Engineering*, Vol. 63, No. 8, pp. 62-3.

The author claims that recent concerns about the durability of epoxy coated rebars may open the door to the use of FRP. Some of the applications of FRPs are in places where electromagnetic neutrality of the reinforcement is necessary (magnetic resonance imaging test chambers in hospitals, guideways for magnetic levitation trains), and structures exposed to highly corrosive environments. The paper outlines some projects underway in the U.S. to develop the use of FRP. Some of the experimental projects are: a bridge in South Dakota, prestressed with composite cables, has been in service for two years; two projects will be constructed within one year as part of the U.S. Army Corps of Engineers CPAR programme; one project will also be constructed within one year through funding by the Intermodal Surface Transportation Efficiency Act (ISTEA); the West Virginia DOT and the University of West Virginia are joining with the Corps' Construction Engineering Research Laboratories (CERL) to construct a bridge deck with FRP rebars; a concrete bridge deck prestressed with composite cables has been in use for two years in Rapid City, S.D. Lawrence Bank, professor of Civil Engineering at the Catholic University of America, is developing pultruded two-dimensional grating bar mat that will be tested by the Washington D.C. Department of Public Works. It is believed that one way to offset the relatively high cost of FRP is to innovate on the ways it is manufactured and used (i.e. it should not be manufactured to duplicate rebars).

136 Timmins, F.D. (1974) Protection of steel – paint, metal spray, hot dip galvanize, weathering steel – which and why?, in 7th *International Metal Spraying Conference, London*, Welding Institute, September, Vol. 1, pp. 232-44.

The author reviews the various protection systems listed in the title. The use of metal spraying is strongly favoured over the other protection systems. The complexity of the paint systems and the lack of quality control generally encountered during their application is the basis for the rejection of the paint systems. Hot dip galvanising is rejected based on the difficulty to get other coatings to adhere to the zinc coating, the low resistance of zinc to corrosion in highly polluted areas and the potential heat related problems encountered during the hot dip galvanising process. Weathering steels show low resistance to corrosion in wet environments. Aluminium spray followed by painting is said to provide the best protection. (Notes: It is fundamentally correct that metal spraying followed by paint coating provides a very good protection from corrosive environments. However, this does not provide a basis for rejection of other protection systems. It is the writer's opinion that the best protection system is the cheapest system which will adequately protect the structure in the specific environment where the structure is intended to be used. For this reason some cheaper and maybe less performing systems are preferred in mildly corrosive environments.)

137 Tolstoy, N., Andersson, G., Sjöström, C. and Kucera, V. (1990) Statistical field survey of exterior building materials degradation, (refer to section D).

138 Tuthill, L.H. (1991) Long service life of concrete, *Concrete International*, Vol. 13, No. 7, pp. 15-7.

The author discusses some of the aspects to consider if a long service life is to be obtained from concrete. Coarse aggregates should be chosen by uniformity of grading and minimum variation of the percent of significant undersize. In most cases normal portland cement can be used except in those rare cases where sulphates may be present or when the most favourable aggregates for a project contain alkali-reactive elements. The addition of pozzolans is recommended for reduced permeability (use of silica fume) or to reduce the heat of hydration in massive structures. The importance of good curing is also emphasised.

139 Tuutti, K. (1982) *Corrosion of Steel in Concrete*, Swedish Cement and Concrete Research Institute, CBI Research, Fo 4.82.

This report is the most complete and detailed research report on corrosion processes of reinforcement in concrete. The process of corrosion is illustrated with a schematic model where the service life is divided into a period of initiation and a period of propagation of corrosion. Theoretical models have been proposed to estimate the time of initiation from the flow of penetrating substances into the concrete cover and the attainment of a threshold concentration. The rate of corrosion in the propagation period has been defined in terms of the moisture content of the concrete and temperature. Different models for this stage have been proposed depending on the initiation mechanism. The final stage of corrosion, cracking of the concrete cover and reduction of the steel cross-sectional area is discussed in the model. The author also presents a method for service life prediction and comparison with laboratory investigations.

140 Tuutti, K. (1980) Service Life of structures with regard to corrosion of embedded steel. performance of concrete in marine environment, in *Publication SP-65, American Concrete Institute*, pp. 223-36.

The service life of concrete structures with regard to reinforcement corrosion is broken down into an initiation stage and a propagation stage. The initiation stage is characterised by the carbonation of the concrete cover and/or the ingress of chloride ions in the concrete. The author discusses the factors affecting the rate of carbonation and the rate of chloride ingress in the concrete. During the propagation stage active corrosion of the reinforcing steel is taking place. Two factors are considered to be important in that stage, namely, the electrolyte, which establishes the contact between anode and cathode surfaces, and oxygen, which must be supplied to the cathode if the reactions are to proceed. The service life is taken as the time to develop visible cracks in the concrete cover (0,1 mm) since the corrosion rate at that time is greatly increased by the flow of oxygen in cracks.

141 Wolf, J. and Lahnert, B. (1990) Performance of latex-modified thin-set mortars in applications of thin-brick veneer, in *Proceedings of the 1st Materials Engineering Congress on Serviceability and Durability of Construction Materials, Denver, CO.*, 13-15 August, Vol. 1, pp. 508-16.

The paper discusses the application specifications and procedures for thin brick. Latex modified mortars are used to increase adhesion to the building facade and to the thin brick. In all the failure cases investigated by the authors it was found that the application had not been done in accordance to the manufacturer's specifications. Poor bond between the substrate and the mortar was found to be the primary reason for failure by debonding. Inadequate hydration of the mortar near the substrate was found to be at the origin of the poor bond. Wetting of the substrate has been recommended by ANSI 108.5 prior to the application of the substrate. Grouting of the joints along with strict quality control was recommended. Grouting of the joints provides additional support to the thin bricks.

142 Wright, J.R. and Frohnsdorff, G. (1985) Durability of building materials: durability research in the United States and the influence of RILEM on durability research, *Materials and Structures / Matériaux et Constructions*, Vol. 18, No. 105, May-June, pp. 205-14.

The paper reviews some of the research on the durability of building materials which has been carried out at the US National Bureau of Standards and other US laboratories in the recent years. It also draws attention to the role of RILEM in promoting international collaboration in research on the durability of building materials. In 1978 an ASTM building materials standard of a new type was established. The standard formalised the logic of accelerated testing and indicated the steps which should be taken in setting up credible tests for service life prediction. Using the methodology outlined in the standard NBS developed an approach to service life prediction of protective coatings for steel. NBS has explored the possible application of infrared thermography and the pulse echo technique to detect flaws in seams of single-ply roofing membranes. The Forest Products Laboratory is concerned with the performance and protection of wood. Research is under way to find wood preservatives which are more environmental acceptable. Organisations which have been major contributors to research on concrete durability for many decades include the US Army Waterways Experiment Station, the Bureau of Reclamation, and the Portland Cement Association. The initial research on epoxy coatings for protection of reinforcing steel against corrosion was carried out by NBS in the 1970's. The importance of the establishment of feedback systems which can capture the information from those responsible for the maintenance, repair and rehabilitation of facilities is emphasised. The areas where research is needed have been identified as: development and application of techniques for elucidating the degradation mechanisms; characterisation of environments (including micro climates); probabilistic mathematical modelling of degradation; service life prediction; condition assessment of materials in existing structures; and construction of large data bases with evaluated data.

143 Yeomans, S.R. (1991) Comparative studies of galvanised and epoxy coated steel reinforcement in concrete, in *2nd International Conference on Durability of Concrete, Montreal, Canada*, (ed. V.M. Malhotra), ACI SP-126, Vol. 1, pp. 355-70.

Black steel, galvanised steel and epoxy coated steel rebars were tested under a salt fog environment and under wet and dry conditions. Half cell potential measurements were made to assess the state of corrosion of the test specimens. Two series of tests, consisting of rebars embedded in concrete cylinders of different sizes, were conducted. In both series the black steel specimens performed badly showing early signs of corrosion. In the first series of tests the epoxy coated rebars performed well with no sign of corrosion after 142 days. However, the galvanised rebars showed early signs of zinc corrosion (half cell potential of -1050 mV). The potential of the galvanised bars slowly changed as the zinc coating depleted. At the end of 142 days the zinc was still providing protection. In the second test series the ends of the rebars were left unprotected. Early corrosion of the epoxy coated rebars was detected where the coating had been damaged. However, the zinc coating on the galvanised rebars protected the exposed steel surfaces adequately. The bond strength of galvanised and epoxy coated deformed rebars was found the be the same as that for the black steel. However, for plain reinforcement, the ultimate bond strength of the epoxy coated steel is 17 % less than that of black steel, while the weathered galvanised steel is 31 % higher.

C Components and assemblies

1 ACI Committee 546 (1980) Guide for repair of concrete bridge superstructures, *Concrete International*, Vol. 2, No. 9, Report No. ACI 546.1R-80, pp. 69-88.

ACI Committee 546 presents guidelines for repair of concrete bridge superstructures (pier caps, beams, decks, curbs, sidewalks, and rails). More specifically, guides are provided for the evaluation of damage and selection of repair method, methods of surface preparation, a description of portland cement concrete, latex modified portland cement and polymer concrete. A list of references containing 46 references is also presented.

2 Akoz, F. and Akman, M.S. (1990) Service life estimation for multi-ply flat roof membranes, in *Proceedings of the 5th International Conference on Durability of Building Materials and Components, Brighton, 1990*, (eds. J.M: Baker, P.J. Nixon, A.J. Majumdar and H. Davies), E & FN Spon, U.K., 7-9 November, pp. 353-8.

A method of predicting the service life of bituminous multi-ply roofing elements is presented. The damage factors investigated were wetting-drying, wetting-partial drying, freezing-thawing, surface heating-cooling and ultraviolet radiation. Tests were conducted both in laboratory and in the field. The tensile strength of samples was measured at various numbers of cycles to assess the level of deterioration. After expressing the serviceability by a measurable property (tensile strength) it was possible to predict the life based on the regional characteristics of the damage factors. A correction factor was applied to the accelerated test results to account for the discrepancy between the lab observations and the field observations.

3 Allan, J.A. (1992) Retrofit ties for brick veneer, (refer to section B).

4 Amoroso, G.G. and Fassina, V. (1983) *Stone Decay and Conservation – Atmospheric Pollution, Cleaning, Consolidation and Protection*, (refer to section B).

5 Andersen, E.Y. (1989) Re-evaluation of structural load carrying capacities, in *Durability of Structures, IABSE Symposium, Lisbon*, 6-8 September, Vol. 57, pp. 731-6.

The author describes how reliability analysis can be used to include the information on measured structural characteristics for re-evaluation purposed. It is shown that the uncertainties about the material properties can be reduced by testing material samples extracted from the built structure. The lower uncertainty results in an increase of the reliability index. A sample example of a bridge shows that determining the concrete and reinforcing steel strength can give rise to a substantial increase in the reliability index. Since the increase of reliability index is a function of the load level, the technique can be use to justify increasing the load on the bridge while maintaining an acceptable reliability index. Load testing an existing bridge is also shown to be potentially beneficial. The author shows, however, that the benefits of load testing is strongly dependent on the certainty with which we know the applied loads, i.e. the standard deviation of the variable load applied during bridge service. The benefits of load testing decrease as the standard deviation of the variable load increases. Finally, the author has a brief look at costs analysis, comparing expected costs as the variable load is increased, given three alternatives: using the bridge unaltered, strengthening the bridge, or replacing the bridge. As expected, the analysis shows that, for low load levels, leaving the bridge unaltered is more economical, at high load levels it is more economical to replace the deficient bridge, and at intermediate loads it is more economical to strengthen the existing bridge.

6 Anon (1993) Removing chloride ions from reinforced concrete, (refer to section E).

7 Anon (1983) *External Masonry Walls Insulated With Mineral Fibre Cavity-Width Batts: Resisting Rain Penetration*, Building Research Establishment Housing Defects Prevention Unit, Defect Action Sheet DAS 12, February.

This bulletin describes the action to take to prevent water penetration to the inner leaf of an exterior wall when the wall cavity is completely filled with mineral fibre insulation batts and mortar is extruding in the horizontal joints between batts. Although the batts are treated to make them water repellent, when there is mortar in the horizontal joint between the batts, rainwater can penetrate to the inner leaf. The mortar can either be mortar droppings or extrusions from bed joints.

8 Anon (1980) Water leaks plague museum, (refer to section D).

9 Anon (1980) Facades: errors can be expensive, (refer to section D).

10 Anon (1979) GAO decries bridge deck corrosion, (refer to section D).

11 Anon (1973) LBJ library needs $1,8 million repair, (refer to section D).

12 Arbogast, D. (1990). Problems affecting the service life of exterior sandstone: case study, the Burlington, Iowa Free Library, (refer to section D).

13 Arumala, J.O. (1992) Water penetration in laterally loaded brick-wall panels, *Journal of Materials in Civil Engineering*, ASCE, Vol. 4, No. 4, November, pp. 432-6.

Water permeance tests were performed on six full-scale brick veneer / steel stud wall systems after exposing the samples to various levels of lateral loading. A lateral load was applied on full-size walls using a vacuum chamber mounted on the test specimens. The magnitude of the lateral pressure corresponded to zero, one, two and three times the design load. The results did not show any correlation between the amount of water permeating the wall and the magnitude of the applied load. The water permeance test was conducted for three days following lateral loading. The results of the permeance tests are somewhat misleading because leaks developed between the test chamber and the surface of the wall specimens. The test results presented are the amount of water loss and the amount of water collected at the flashing. It is not clear whether the reported amount of water loss was corrected adequately for the large losses reported to take place between the chamber and the wall. Some of the conclusions don't seem to be relevant to the work presented in the paper (e.g. it is reported that the weep holes at the bottom of the cavity may be obstructed by mortar droppings, and that the water penetrating the wall may soak the wall ties, and that weak brick veneer lets more water penetrate).

14 Arup, H. (1983) Prospects concerning corrosion of steel in concrete, in *Durability of Concrete Structures, CEB-RILEM International Workshop, Copenhagen*, (ed. S. Rostam), 18-20 May, pp. 345-350.

The author discusses the use of half cell potential measurement for assessing the state of corrosion of steel embedded in concrete. The importance of half cell potential measurements as a non-destructive testing/inspection technique is emphasised. The electrochemical potential mapping as described in the ANSI/ASTM C 876-77 has been used especially for highway bridge decks and should be used with caution in other types of structure. A laboratory investigation of bridge deck corrosion is presented. The parameters which were under examination were epoxy coating of one or both rebar layers and the application of a low permeability coating on slabs where corrosion had already been initiated. It was found that the coating of the underside of the slab reduced the current much more than a coating of the top layer (the coating of the underside reduces the amount of oxygen available to the bottom steel for the cathodic reaction). It was found that most of the corrosion of the top layer was contributed by the contact with the bottom layer. If the bottom layer is prevented from reacting with the oxygen e.g. by epoxy coating, this contribution can be prevented.

15 Attwood, D., Nessim, M.A., Ghoneim, A., Cormeau, A. and Cheung, M.S. (1991) Application of reliability theory to in-service monitoring and maintenance of parking garages, *Canadian Journal of Civil Engineering*, Vol. 18, pp. 781-8.

The paper presents an example of the application of reliability theory to in-service monitoring and maintenance. The time-dependent deterioration of a reinforced concrete indoor parking structure's driving surface is considered. Based on an estimated rate of slab delamination, crack width in the concrete cover, and the effect of temperature on the rate of corrosion, the authors have derived a model which relates the amount of delamination to the age of the garage, crack width, temperature, and time to deterioration (which in turn is dependent on the depth of the cover, water-cement ratio, and chloride ion concentration). The concepts used in the assessment of parking garage reliability described in this work may be applied to a variety of structures and building systems in order to make decisions regarding different aspects of in-service maintenance and repair.

16 Babaei, K. and Hawkins, N.M. (1988) Evaluation of bridge deck protective strategies, *Concrete International*, Vol. 10, No. 12, pp. 56-66.

The authors present the existing knowledge of different common strategies for protecting bridge decks against deterioration caused by de-icing salts. The strategies considered are: increased cover to the top bars; low-slump, dense concrete overlay; latex-modified concrete overlay; interlayer membrane/asphalt concrete system; and epoxy coated bars. The authors propose practical guidelines to make each strategy acceptable from a point of view of durability. To achieve a fifty-year or more effective service period, designs must consider the severity of salt application, the water/cement ratio and the cover thickness. A cost effectiveness study of the protection strategies (either single or double protection) was performed. For single protection, provision of a concrete cover of 9 cm is the least expensive. For double protection decks, provision of top and bottom mats' epoxy coated rebars was found to be the least expensive.

17 Baker, M.C. (1969) *Designing Wood Roofs to Prevent Decay*, Canadian Building Digest, Division of Building Research, National Research Council of Canada, CBD 112.

In an earlier digest by the same author (CBD 111) excessive moisture content of wood (above 35 % or so) was identified as the major cause of wood decay which can be most easily controlled. In order to prevent wood decay the moisture content of wood must be maintained below 20 %. The author deals with some of the moisture conditions to be considered in the design of wood roofs. In normal building environments the moisture content of wood varies between 5 % in the winter and 15 % in the summer. Higher moisture contents which can lead to rotting of wood can however be reached when building systems are improperly designed or built. Venting on the cold side of insulation would provide a means of disposing of the moisture which will reach the cold side of the deck. Under normal conditions it is believed that condensation of moisture is not a problem when properly dried wood is used for the construction. The wood absorbs the moisture and releases it again when the temperature rises. The use of vapour barriers can be a problem in cases where the deck is enclosed between the vapour barrier and the roofing membrane. In such a case the moisture in the deck will get trapped and may cause decay, especially if the deck was exposed to rain before the roofing and the vapour barrier were installed.

18 Baronio, G., Berra, M., Binda, L. and Fontana, A. (1989) Durability of masonry strengthening by injection techniques, in *Durability of Structures, IABSE Symposium, Lisbon*, 6-8 September, Vol. 57/2, pp. 755-60.

Brick masonry prisms, prepared with different types of mortar, were subjected to compression tests up to failure. The cracked prisms were injected with epoxy resin or cement-polymer grout and retested. Injection of the failed prisms restored an average of 85 % of the original strength. The stress-strain curve of the injected specimens tend to remain similar to those of the prisms under the first compression test. A correlation was found to exist between the ultrasonic pulse velocity and the strength of the prisms both in the virgin and injected conditions. It seems that the authors have a different concept of durability than everybody else. The problem of durability (i.e. how the repaired structure will stand with time) is not addressed

19 Barrett, N. (1993) Selecting the right wall tie, *Construction Repair*, Vol. 7, No. 3, May-June, p. 13.
The author discusses some of the problems and causes leading to wall tie replacement. The main two causes are corrosion and, especially in the London area, non existent ties. The bulk of the ties being installed in existing buildings are replacements for corroded ties. It is mentioned that on of the biggest wall tie replacement contract in the U.K. consists on replacing ties in some 400 houses. The short paper does not address the various types of ties available for replacement nor does it give guidelines for the selection of appropriate wall ties. The title is misleading in that respect.

20 Barrett, P. (1983) Rain penetration through masonry walls, *Insulation Journal*, Vol. 27, No. 11, November, pp. 35-6, 38-9.
A significant change in the construction industry which took place over the years is believed to be partly responsible for water penetration problems in buildings. The change from local builders to the national builder and the bricklayer becoming a subcontractor is responsible for the loss of knowledge of local climatic conditions and the loss of skills of recognition and remedial techniques. The change in composition of the mortar (lime being replaced by plasticisers) has resulted in much more permeable mortar being used nowadays than was used previously. The author discusses problems of water penetration through masonry cavity walls. A brief discussion of the development of exposure zones developed in terms of driving rain and used in BS 5618 (Code of Practice for Thermal Insulation of Cavity Walls) indicates that the present exposure system provides a realistic guide to suitable building construction when used in conjunction with UF foam cavity wall insulation. The influence of cavity insulation on wall resistance to rain penetration is also discussed.

21 Beall, C. (1990) Building joint movement, in *Proceedings of the 1st Materials Engineering Congress on Serviceability and Durability of Construction Material., Denver, CO.*, 13-15 August, Vol. 1, pp. 517-26.
The factors affecting joint movement in masonry cladding and sealant failure are discussed as parameters for sizing and detailing brick expansion and concrete masonry control joints. Control joints in masonry should provide room for contraction and expansion due to temperature changes and moisture variation. In order to minimise the strain on the joint sealant, the sealant should be placed at mean temperature. In order to minimise the stress on the joint sealant the joint depth should be less than or equal to the joint width.

22 Beasley, K.J. (1990) Leaking brick-clad walls: causes, prevention, and repair, *ASCE Journal of Performance of Constructed Facilities*, Vol. 4, No. 2, May, pp. 124-33.
The author discusses some common problems which cause leaks in brick-clad walls. Those are: improper flashing, improper drainage of the cavity, presence of mortar droppings in the cavity which bridge the cavity and allow access of water to the backup wall, deteriorated of improper mortar joints, deterioration of sealants in expansion joints. The author does not discuss any specific field experience but rather presents a guide to proper design and workmanship. Recommendations for repair are given. The paper does not present any new material that has not already been discussed in many other papers.

23 Beasley, K.J. (1988) Use and misuse of exterior travertine cladding, (refer to section B).

24 Beeby, A.W. (1983) Cracking, cover, and corrosion of reinforcement, (refer to section B).

25 Binda, L. and Baronio, G. (1989) Performance of masonry prisms repaired by grouting under various environmental conditions, *Masonry International, Journal of the British Masonry Society*, Vol. 3, No. 2, pp. 74-9.
The authors have investigated a repair technique by injection of epoxy resins or cement-polymer grouts into masonry prisms. The description of the test programme and the findings have been described in Baronio et. al. (1989) and have been summarised above. No new information is presented in this paper. Salt decay was not stopped by the technique. Temperature variations influenced the behaviour of masonry repaired by resins: freezing increases the material stiffness and brittleness, thawing, on the contrary, decreases both stiffness and strength.

26 Binda Maier, L., Rossi, P.P. and Landriani, G.S. (1983) Diagnostic analysis of masonry buildings. strengthening of building structures – diagnosis and therapy, in *IABSE Symposium, Venezia*, IABSE reports Vol. 46, pp. 131-8.

The authors describe operative criteria for the stress analysis of masonry buildings for which data on local geometrical configuration, physical properties of materials and loading history are incomplete. The authors discuss the use of in-situ non-destructive tests to obtain stress and strain states in masonry buildings. In-situ tests based on the insertion of special flat jacks in the masonry represent a useful tool to determine the mechanical behaviour of the material without extracting samples.

27 Bjegovic, D., Ukraincik, V. and Beus, Z. (1990) Evaluation and repair of concrete structure in urban environment: case study, (refer to section D).

28 Blades, J.T. and Perl, G.C. (1989) Durability provisions for prestressed concrete, in *Durability of Structures, IABSE Symposium, Lisbon*, 6-8 September, Vol. 57, pp. 517-23.

The authors look at the code provisions for prestressed concrete and normal reinforced concrete structures. They note that some codes have the same provisions for both types of concrete. Some codes, however, have recognised the greater susceptibility of prestressed concrete to corrosion in terms of consequence of corrosion and also in terms of the corrosion process involved. Those codes, such as ACI, require greater cover thickness. Other codes (CEB-FIP and British codes) also require higher cement contents and higher compressive strength. None of the codes reviewed by the authors required lower water to cement ratio. The authors present two field case studies where inadequate protection of the prestressing steel lead to the failure of structures. The authors conclude by saying that durability problems in concrete structures result from inadequate detailing and specification at the design phase and/or poor work practices during the construction phase of a structure.

29 Brandt, E. (1984) Assessment of the conditions of buildings, in *Proceedings of the 3rd International Conference on Durability of Building Materials and Components*, *1984 Espoo, Finland*, 12-15 August, Vol. 2, pp. 433-9.

The paper discusses in very general terms different types of maintenance and inspection. It also gives suggestions on how to treat the information collected during inspections in order to make inspection routines and evaluation of rest of life simple. Guidelines about reporting inspection observations are given. It is suggested that rest of life time predictions should be made subjectively by experienced engineers.

30 Bright, K.D. (1991) Testing cathodic protection systems, *Concrete International*, Vol. 13, No. 7, pp. 37-9.

The author reports the results of a two year study of various cathodic protection systems for rehabilitating a parking garage. The parking structure showed signs of deterioration ten years after its construction. Three years after the repairs the slab was again showing distress. A total of 11 different cathodic protection systems were implemented on a section of the slab to assess the performance of the systems. In general, the performance of the systems was disappointing. It appears that any top system that inhibited moisture from wetting the surface had poor performance. The most effective system was estimated to cost US\$40/m^2. The author expressed the opinion that anyone considering a cathodic protection system should first test it on the actual structure to measure its potential performance.

31 British Standard Institution (1992) *Guide to Durability of Buildings, and Building Elements, Products and Components*, BS 7543: 1992.

The Standard gives guidance on durability, required and predicted service life and design life of buildings and their components and/or parts. It applies primarily to new buildings rather than to alterations and repairs. Guidance is also given for presenting information on the service and design life of buildings and parts.

32 Browne, R.D. (1982) Design prediction of the life for reinforced concrete in marine and other chloride environments, *Durability of Building Materials*, Vol. 1, No. 2, pp. 113-25.

Corrosion generally occurs when the electrical resistivity of the concrete cover to steel is low, when atmospheric oxygen can diffuse through the cover layer, and when the chloride level of the steel surface is 0,4 % by weight of cement. Where any one of the three factors is absent, damage does not occur. A relationship between the rate of chloride diffusion and the chloride diffusion coefficient is proposed from which the time required for the Cl^- concentration at the steel reaches a critical value can be predicted. The time of onset of corrosion is predicted from the diffusion rate of the chlorides from the environment through the concrete cover. This time scale is used as a basis for predicting the service life of the structure. The additional time required to cause spalling of the concrete cover is believed to be from 6 months to 5 years depending on the availability of oxygen and the quality of the concrete. For corners it is believed that the life may be approximately halved because of the two-directional diffusion path from the two sides. Evidence is emerging that the use of cement replacement materials (pulverised fuel ash and blast furnace slag) can reduce the chloride diffusion coefficient by significantly reducing the size of the interconnected pores in the hardened cement. Various field measurements are proposed which can assess the condition of the concrete structure before spalling starts. Simple repairs are also proposed to extend the service life depending on the state of deterioration of the concrete structure. (This seems to be a promising technique of assessing the existing condition of concrete bridge decks and predicting their remaining service life before damages become visible. However, the threshold chloride concentration is not as well established as the author believes. The problem of carbonation of concrete was not addressed).

33 Building Research Association of New Zealand (1984) *Sealed Joints in External Claddings – 2. Sealants*, Building Information Bulletin 239.

This bulletin gives a description of various types of sealants used in the construction industry for sealing glazing and cladding. The various sealants are classified by chemical type and by movement type. A table is provided in which 12 different types of sealants have been described in terms of the substrate suitability, the maximum recommended movement, the expected service life and their typical uses. The durability of the listed sealants range from five years for bituminous sealants to 20 years for silicone an polysulphide sealants. Guidelines are given for the proper joint preparation, sealant application and maintenance.

34 Building Research Establishment (1992) *Flat Roof Design: Waterproof Membranes*, BRE Digest 372, June.

Various roofing systems are reviewed. A description of built-up roofing with its various components is given. This system consists of two or three plies of bituminous felt. Polymeric single-ply membranes are also described. The types of polymers, the methods of forming the joints and the methods of attachment are discussed. Liquid-applied membranes are used for remedial work or to prolong the life of existing membranes. Mastic asphalt consists of two 10 mm layers of asphalt mix. The mastic asphalts are vulnerable to movements since the mastic is relatively hard and brittle once it sets. Durability of roofing membranes is briefly discussed. The durability of membranes seems to be closely related to their fatigue resistance. Fatigue test results on various membrane systems are presented.

35 Building Research Establishment (1991) *Why do Buildings Crack?*, BRE Digest 361, May.

The digest examines the causes of cracking in buildings and shows the visible results of a wide range of problems. The causes of cracking outlined are: differential movements of various parts of a building, temperature changes, drying of moisture and wetting and drying, freezing and thawing, sub-surface crystallisation of soluble salts, sulphate attack, corrosion of embedded steel, moisture expansion of fired clay products, alkali silica reaction, imposed load effects, foundation movement, and vibration.

36 Building Research Establishment (1977) *Repairing Brickwork*, BRE Digest 200, April.

This digest is concerned with damage to small buildings. Various causes of damage are referred to briefly. The causes of damage reviewed are: ground movement resulting from movement of the water table, frost heave, vibration; thermal movement ; drying shrinkage; fire; roof spread where pitched roofs have been inadequately tied; sulphate attack; expansion on wetting; corrosion of embedded steel; unsound materials; frost when the brick is very wet; salt which produces efflorescence. Guidelines are given to repair damage. For brick veneer showing signs of tie corrosion, it is recommended that the old ties be removed and new corrosion resistant (stainless steel, carefully protected steel, non-ferrous metal) ties be placed.

37 Building Research Station (1966) *Cracking in Buildings*, Digest 75, Second Series, October.

This digest examines the causes of cracking in buildings and shows, with the help of illustrative examples, how an understanding of the factors responsible is necessary for correct diagnosis and repair. The main cause of cracking of masonry walls is the lack of provision in the design for expansion of the bricks that takes place when brickwork is exposed to moisture. Frost heave, differential settlement, sulphate attack, thermal movements, and deflection of floors are also identified as causes of cracking.

38 Cady, P.D. and Weyers, R.E. (1983) Deterioration rates of concrete bridge decks, *Journal of Transportation Engineering*, Vol. 110, No. 1, pp. 34-44.

Procedures are presented for estimating the parameters required for life-cycle costing of the elements of maintenance and rehabilitation of concrete bridge decks subject to the deteriorative effects of de-icing chemicals. As background information, the authors discuss the nature of the deterioration process taking place in deck slabs. They also discuss expressions used to evaluate the time of cracking as derived by Bazant and time to initiation of cracking. The authors present a method to estimate the time at which rehabilitation of the deck is likely to be necessary and the time at which beginning of maintenance is necessary. The methodology is neither applicable to bridges containing epoxy coated reinforcement, nor to overlaid or cathodically protected decks for which chloride penetration rate and corrosion rate are not known.

39 Carlson, A.R. (1991) Computer simulation of wall condensation problems, (refer to section E).

40 Carrier, R.E. and Cady, P.D. (1973) Deterioration of 249 bridge decks, (refer to section D).

41 Carruthers, J.F.S. (1982) Classifying the performance of components by usage, (refer to section E).

42 Cassady, L.W. (1990) Incompatibility of building components, *ASCE Journal of Performance of Constructed Facilities*, Vol. 4, No. 1, February, pp. 21-3.

The author reports about a storage facility consisting of a steel frame with precast panel walls and a roof with a ballasted single-ply membrane with galvanised metal edge flashing and gutters. After only three years of construction the gutters had been perforated by corrosion. The expected service life of the gutters was 20 years. The problem was found to be the roof ballast which contained significant amount of iron, aluminium, magnesium, and silicon. The minerals were being washed away by rain and deposited in the gutters where it would set an electrolytic cell with the gutters causing accelerated corrosion of the zinc coating on the gutters.

43 Chrest, A.P. (1990) Structural system performance in parking structures, *Concrete International*, Vol. 12, No. 8, pp. 30-4.

The author considers cast-in-place conventionally reinforced concrete structural systems, and thin slab cast-in-place conventionally reinforced concrete structural systems, to be less suitable than other structural systems for parking structures. The author recommends cast-in-place post-tensioned concrete one way slab and beam or precast, pretensioned, pretopped concrete double-tee systems. Structural systems that require surface coatings or other relatively expensive protective measures to make them durable in corrosive environments should be avoided.

44 Cohen, J.M. (1991) Cladding design: whose responsibility?, *Journal of Performance of Constructed Facilities*, ASCE, Vol. 5, No.3, pp. 208-18.

The author looks at some of the issues involved in the design of building cladding. Some of those issues consist of durability and integrity of cladding materials, analysis and structural design, education, practice, and future uses of cladding. The author reports that, in a 1990 survey conducted by the ASCE's Research Committee of the Technical Council on Forensic Engineering, durability of man-made building materials was noted as one of the most important research topics. The paper discusses mostly aspects related to the use of natural stones for cladding. It is noted that there is a lack of communication and collaboration between the architects and structural engineers and there is a greater lack in teaching on the building envelope. Cladding are required to carry wind loads and their anchors must resist the same loads and the weight of the cladding and, as such, their design requires the expertise of structural engineers. Finally the author observes that there is growing trend towards structural engineers looking at cladding as a structural element. Research work quoted by the author indicated that cladding not only carries the environmental loads to the framework, but also contributes significantly to the stiffness of buildings and plays an important role in frame members force distribution.

45 Cohen, J.M. and Monteiro, P.J.M. (1991) Durability and integrity of marble cladding: a state-of-the-art review, (refer to section B).

46 Comité Euro-International du Béton (1989) *Durable Concrete Structures – CEB Design Guide*, (refer to section E).

47 Comité Euro-International du Béton (1990) *CEB – FIP Model Code 1990. 1st Draft Chapters 1-5*, CEB Bulletin d'Information No. 195, March.

Section 1.5 of the Model Code presents a discussion of the principles of design versus durability. It is believed that if a structure is designed, executed and maintained according to the requirements of the Model Code, there is a high probability that it will withstand the expected conditions of use for a long period of time (50 years or more).

48 Comité Euro-International du Béton (1990) *CEB – FIP Model Code 1990. 1st Draft Chapters 6-14*, CEB Bulletin d'Information No. 196, March.

Chapter 8 of the Model Code covers aspects of durability. Design strategies to adopt in order to obtain a structure with adequate durability are outlined. Design criteria for durability take the form of recommendations with respect to: the structural form; composition of the concrete material and the cover thickness for reinforcing steel and prestressing steel; detailing; nominal crack width limitation; and, special protection such as use of structural protection such as roof projection, use of surface coating, increase of concrete cover, modification of the micro-environment, coating of reinforcement, cathodic protection, etc.

49 Corr, D. (1983) *Cavity Wall Ties – A Bibliography*, LIB/176, The National Institute for Physical Planning and Construction Research, St. Martins House, Waterloo Road, Dublin.

This is a bibliography containing 33 references. The database source is not specified. It seems to result from a rather fast search of the literature in a database since the word tie is common to most titles. Although the number of bibliographic entries is small some titles are of interest. No annotation is provided with the references.

50 Cuoco, D.A. and Velivasakis, E.E. (1989) Aluminium curtain wall panel failure, assessment and repair, (refer to section D).

51 Davies, H. (1990) Studies of the performance of fusion bonded epoxy coated reinforcement during the construction process, (refer to section B).

52 de Vekey, R.C. (1989) *Corrosion of Steel Wall Ties: Recognition and Inspection*, British Research Establishment, Information Paper IP13/90, November, 4 p.

The paper discusses some methods of detecting corrosion of wall ties. Corrosion of ties can be recognised either from circumstance (type of construction and age of construction are often used as an indication for the likelihood of corrosion problems developing in masonry walls), from cracking of the walls at the mortar joints where the ties are embedded, bulging and movement of some parts of the building such as the roof edge, or from collapse of the outer leaf. The two methods of inspection recommended by the author are direct examination by removal of brick or the use of a borescope. The author mentions that the absence of corrosion in the cavity is not conclusive since the worst corrosion usually occurs in the mortar joint. The reader is referred to BRE Digest 329 for more information on inspection methods.

53 de Vekey, R.C. (1990) *Corrosion of Steel Wall Ties: History of Occurrence, Background and Treatment*, British Research Establishment, Information Paper IP12/90, Nov., 4 p.

The paper discusses problems of wall ties corrosion and the action to take if corrosion is detected. Corrosion of wall ties results in a loss of serviceability (bulging, cracking or leaking), but occasionally the walls have actually collapsed under wind load. The factors which are likely to have an influence on the life of masonry ties have been identified as: steel alloy used, the mortar type, standard of workmanship, the severity and duration of driving rain for the area and the particular local exposure condition of the affected wall. Observation of buildings have shown that the corrosion is more pronounced in the outer leaf than in the inner leaf, which indicates that the level of exposure is an important factor. Since corrosion of steel is accompanied by a four-fold increase in volume, cracking of the masonry leaf is often observed when corrosion of the ties takes place. Short guides are given to assess the state of corrosion of masonry ties. A problem arises when corrosion of the wall ties is detected and the remaining life estimated to be less than required. In that case a decision has to be made on the preferred method to adopt to replace the corroding ties. The author discusses some of the alternatives.

54 de Vekey, R.C. (1989) The durability of steel in masonry, *British Ceramic Transactions and Journal*, Vol. 88, No. 5, September-October, pp. 201-3.

Problems and failures of steel in brickwork exposed to outdoor conditions are quite common. The paper reviews the mechanism of failure, and the way to specify to avoid failure in relation to some of the key factors such as the exposure condition, type of steel alloy and protective coating system if used. Factors such as the chemical nature of the environment, the effectiveness of any protective systems and type of steel alloy are briefly discussed. Carbonation of mortar is noted as the principal cause for corrosion of steel embedded in mortar, along with ingress of chlorides. Zinc and epoxy coatings are the two most widely used protective coatings for steel in masonry. The paper also presents a list of 27 references related to the durability of metals in masonry.

55 Dreger, G.T. (1989) Cementitious-cladding failure – a building façade collapse: an odyssey of failure and lessons learned, (refer to section D).

56 El-Sayed, H.A., Abd El-Wahed, M.G. and Ali, A.H. (1987) Some aspects of the corrosion of reinforcing steel in concrete in marine atmospheres, (refer to section D).

57 Fookes, P.G., Comberbach, C.D. and Cann, J. (1983) Field investigation of concrete structures in south-west England, part I, (refer to section D).

58 Fookes, P.G., Comberbach, C.D. and Cann, J. (1983) Field investigation of concrete structures in south-west England, part II, (refer to section D).

59 Freyermuth, C.L. (1991) Durability of post-tensioned prestressed concrete structures, *Concrete International*, Vol. 13, No. 10, pp. 58-65.

The author presents a brief summary of the research in the area of durability of post-tensioned prestressed concrete, experience on the durability of post-tensioned bridges (bridge decks, segmental concrete bridges, stay cables) and buildings. The various research projects showed that post-tensioning can provide substantial improvement of durability by eliminating cracks and limiting crack width. The various case studies reported by the author indicate that the cause of deterioration and low durability of existing post-tensioned structures can be traced to deficient design and substandard workmanship. ”Recent improvements in corrosion protection materials for tendons and stay cables provide a very high degree of assurance of long-term durability in future”.

60 Freyermuth, C.L., Klieger, P. and Stark, D.C.(1970) Durability of concrete bridge decks – a review of cooperative studies, (refer to section D).

61 Fujiwara, M. and Minosaku, K. (1989) Measure against deterioration of concrete bridges due to chloride ions, in *Durability of Structures, IABSE Symposium, Lisbon*, 6-8 September, Vol. 57, pp. 841-6.

The paper presents an outline of a Guideline for coastal structures prepared by the Ministry of Construction in Japan. The information required to prepare the guideline was obtained from a survey of 920 concrete bridges situated within 500 m from the coastline. The areas where deterioration had indicated that special measures are required to insure durability were identified and amount of airborne chloride ions was measured at 266 sites. As a result of the findings from the survey, the guideline was drafted to cover four basic concepts: 1) the shape of the bridge members is important and the use of corners was discouraged since corners provide more front area from which chlorides can penetrate the concrete; 2) the use of appropriate cover is recognised as an effective means of protecting the steel reinforcement; 3) the concrete material should be controlled to minimise the amount of chlorides in the concrete; and, 4) dense concrete of proper composition (cement content and water-cement ratio) was found to help reduce the chloride penetration.

62 Green, P. (1988) Structures need a low-sodium diet, (refer to section D).

63 Green, P. (1986) Owners reclad damaged masonry, (refer to section D).

64 Grimm, C.T. (1985) Corrosion of steel in brick masonry, (refer to section E).

65 Grimm, C.T. (1976) Metal ties and anchors for brick walls, *Journal of the Structural Division, Proceedings of the ASCE*, Vol. 102, No. ST4, April, pp. 839-58.

The paper presents an extensive description of metal ties and anchors used for brick walls. The various types of ties and anchors, the materials used for their manufacture, and the tested capacity of the various types of connectors are discussed. The estimated allowable axial load on typical wall anchors varied from 36 to 418 kg.

66 Gulikers, J. (1989) Influence of local repairs on corrosion of steel reinforcement, in *Durability of Structures, IABSE Symposium, Lisbon*, 6-8 September, Vol. 57/1, pp. 151-6.

A description is given of a concrete corrosion cell which has been developed to investigate the effect on the corrosion process of the mutual influence between rebars in carbonated concrete and rebars embedded in repair mortar. It was found that rebars embedded in mineral mortars can develop into macro cathodes, the rate of their reactivity being mainly determined by the diffusion of oxygen. When these macro cathodes are electrically connected with rebars in carbonated concrete an acceleration of their anodic processes will occur. For rebars partly embedded in repair mortar and partly in carbonated concrete it is likely that the small macro anode will occur next to the large macro cathode.

67 Halsall, R. (1989) New building designs incorporating lessons from failures, in *Durability of Structures, IABSE Symposium, Lisbon*, 6-8 September, Vol. 57, pp. 549-54.
The author addresses the problem of durability of parking garages. A list of causes of corrosion reinforcement and structural distress is presented. It is believed that there are about 5000 parking structures in Canada which have been constructed within 30 years. By the late 1970's serious problems were obvious in many of these structures but new structures built in the 1980's were built the same way without recognition of the failures that had already taken place with existing structures. It is believed that the major contributing factor for those problems was the fierce competition which existed for the construction of cheap parking structures. Because of that competition, bonded post-tensioned tendons gave way to unbonded tendons which then showed much more problems than the bonded tendons. The author discusses briefly the importance of the CSA Standard for parking structures which first appeared in 1987. An interesting table is presented in which, for different types for parking structure constructions, the framed area, the year complete, the year when major repairs were required, the direct costs to date for repairs and the estimated additional initial costs which would have avoided the needed repairs. From the table it is obvious that relatively small increases in initial costs could have avoided very costly repairs. The costs for repair of post-tensioned structures is about five times the costs of repair of other types of construction.

68 Haver, C.A. (1989) Corrosion of steel embedded in masonry walls, (refer to section B).

69 Haver, C.A., Keeling, D.L., Somayali, S., Jones, D. and Heidersbach, R.H. (1990) Corrosion of reinforcing steel and wall ties in masonry systems, (refer to section D).

70 Heidersbach, R. and Lloyd, J. (1985) Corrosion of metals in concrete and masonry buildings, (refer to section D).

71 Heidersbach, R., Borgard, B. and Somayaji, S. (1987) Corrosion of metal components in masonry buildings, (refer to section D).

72 Hillemeier, B. (1989) New methods in the rehabilitation of prestressed concrete structures, in *Durability of Structures, IABSE Symposium, Lisbon*, 6-8 September, Vol. 57, pp. 311-6.
The author briefly describes some of the inspection and repair methods used for prestressed concrete structures. The radar technique is used to detect the location of steel in concrete by detecting the location of interfaces between materials of different density. Safe techniques for opening prestressing sheathings along with a method of visually inspecting tendons in the sheathing using a borescope are briefly discussed. Finally, a method of injecting grout in ungrouted prestressing ducts and a method of reprofiling repaired sections using a wet shotcrete method are presented. The paper presents only a very limited number of techniques which can be used to inspect prestressing steel in structures.

73 Höglund, I., Espling, R. and Hilding, G. (1989) Functional studies of flat roofs covered with membranes of butyl rubber, (refer to section B).

74 Karni, J. (1982) The durability of prefabricated reinforced concrete external walls and cladding in buildings, *Durability of Building Materials*, Vol. 1, No. 2, pp. 141-60.
An investigation was initiated to assess the durability of prefabricated reinforced concrete external wall panels used in several industrialised building systems throughout Britain. To compare the degree of exposure, the Index of Exposure to Driving Rain was used. The survey was planned to include visits to prefabricated-element factories and to building sites, as well as tests at B.R.S. laboratories on materials obtained from the factories. An analysis of the preliminary observations showed that location of the most common weakness of the exposed buildings was at the panel-to-panel joints rather than in the panels themselves. The quality of the panels included in the survey was found to be good and deterioration after 8 years of observation was found to be non-existent. However, more observation, over many more years, are required to assess the panels' durability.

75 Keller, H., Trestain, T.W.J. and Maurenbrecher, A.H.P. (1992) The durability of steel components in brick veneer/steel stud wall systems, (refer to section D).

76 Klaver, E. and Mijnsbergen, J. (1993) *Modelling the Durability of Brickwork Masonry: The Concept of the Relative Performance*.

The use of probability concepts is made to predict the change of relative performance of brick masonry with time. The necessary data required to estimate the probability of failure by various modes as a function of important factors were obtained by interviewing experts. Using probability theory it was possible to predict the loss of performance (equivalent to the increase in probability of failure) with time. A computer programme was written to calculate the probabilities of failure with various key parameters influencing the performance (type of pointing, location on the building, brick type, etc.) and accounting for various failure mechanisms. The important failure mechanisms identified from other surveys were: freeze/thaw, efflorescence, soiling, and corrosion of cavity ties. The paper gives an example of the use of the programme for assessing the loss of performance of the pointing with time. Although other failure mechanisms were identified it is not clear which are accounted for in the computer programme. This is an interesting design tool which can be used to assess the effect of various key parameters on the durability of brick masonry. However, since the data was obtained in The Netherlands, it is not known how reliable the software would be for other countries where climatic conditions are different and, consequently, probabilities of failure by certain mechanisms are different.

77 Lin, C.Y. (1980) Bond deterioration due to corrosion of reinforcing steel, in *Performance of Concrete in Marine Environment*, ACI Publication SP-65, pp. 255-69.

The author presents the results of an experimental investigation conducted to determine the effect of flexural cracks on the rate of corrosion of reinforcement and the reduction in bond strength due to longitudinal corrosion cracks. The tests were performed on concrete beams exposed to sea water under impressed currents. The tests showed that, for a given flexural crack size, the higher the impressed current, the faster the longitudinal crack will start. Under constant impressed current, the corrosion rate is not affected by the flexural crack width. If cracking results from the application of an overload and the crack is allowed to close, the corrosion rate is significantly reduced compared to the condition where the crack remains open. For beams with continuous reinforcement, the moment capacity was reduced from 12 to 50 % under impressed currents of 5 to 10 mA/cm^2. About 35 % of the bond stress was found to be lost from corrosion in beams designed with lapped splices. The resistivity of the concrete was found to play an important role in the susceptibility to corrosion of the reinforcement.

78 Litvan, G.G. (1990) Performance of parking garage decks constructed with epoxy coated reinforcing steel, (refer to section D).

79 Litvan, G.G. (1982) *Deterioration of Indoor Parking Garages*, Canadian Building Digest, Division of Building Research, National Research Council Canada, CBD 224.

The total of repair of deteriorating parking garages in Canada is estimated at 1500 million dollars. The author outlines some of the most common techniques for protecting the steel from corrosion in parking garages. The use of epoxy coatings creates an electric discontinuity in the corrosion cell and would therefore stop corrosion. Exclusion of reactants such as oxygen, water, and chlorides can be achieved by the use of good quality cover concrete, the use of membranes, overlays, or sealers. Cathodic protection can also be used (two common forms are the use of a sacrificial anode or imposed current with an inert anode) whereby the reinforcing steel is made the cathode in the corrosion cell. Corrosion inhibitors such as calcium nitrite can also be used, although the duration of the protection is not well known.

80 Litvan, G.G. (1982) *Evaluation and Repair of Deteriorated Garage Floors*, Canadian Building Digest, Division of Building Research, National Research Council Canada, CBD 225.

This digest describes the methods used for the assessment of damages and how these tests can aid in the selection of the repair method. Visual inspection is performed to detect possible structural defects, cracks in the deck, the conditions of beams and columns and to identify as accurately as possible the cause of deterioration. The drainage conditions and condition of expansion joints is also assessed during the visual inspection. A delamination survey using a chain dragged over the surface of the deck identifies location where corrosion has progressed to a point where the concrete cover has failed. A cover meter survey using a pachometer identifies the thickness of the concrete cover. Measurement of corrosion potential can identify locations where corrosion is taking place but cannot assess the corrosion rate. Measurement of chloride content in the concrete cover can help identifying the cause of corrosion. The repair strategy should include repair of concrete in all delaminated areas, the installation of a water proofing membrane, the repair of expansion joints, and necessary upgrading of the drainage system. The concrete repair technique to be adopted, and maintenance programme to adopt are briefly discussed.

81 Malhotra, S.K. (1989) Factors which influence durability of wooden structures, (refer to section E).

82 Manning, D. (1987) A rational approach to corrosion protection of the concrete components of highways bridges, in *Concrete Durability – Katharine and Bryant Mather International Conference*, SP-100, American Concrete Institute, Vol. 2, pp. 1527-47.

The paper traces the development of corrosion protection requirements in Ontario over a thirty year period. Problems of durability of bridges were identified as early as 1963. At that time, however, problems were severe scaling of the concrete. After adoption of air entrainment agents in concrete the problem of durability moved to corrosion of reinforcement. The importance of low permeability concrete and adequate cover is emphasised by the presentation of a chart indicating the number of daily salt application which will lead to corrosion of reinforcement for a given concrete cover and water-cement ratio. The use of membranes have been demonstrated for a long time to improve the durability of concretes exposed to road salts. It is also believed that epoxy coating improves considerably the durability of reinforced concrete. The problems associated with the used of a coated top mat with a black steel bottom mat don't seem to be of concern to the author. It is believed that the large cathode to anode ratio is of significance only if the bare area of coated bars exceeds 0,24 % and even with that much damaged areas, the service life is projected to be almost 20 times the service life of decks with two mats of uncoated reinforcement. Various components for bridges other than the deck are prone to durability problems due to chlorides. These are the components where chloride contamination can occur from surface runoff, or by splash from adjacent roadway surfaces.

83 Manning, D.G. (1984) Accelerated corrosion in weathering steel bridges, (refer to section D).

84 Manning, D.G. (1983) The need for international co-operation in future work on repair and strengthening, in *Durability of Concrete Structures, CEB-RILEM International Workshop, Copenhagen*, 18-20 May, pp. 389-98.

Various areas where international collaboration is needed for research are identified. Those areas are directly related to deterioration of highway bridges. It is believed that the methods of detecting deterioration were inadequate at that time and further research was needed. The techniques discussed by the author are infrared thermography and radar. Both methods were found to be unreliable for detection of delamination (radar would falsely identify sound areas as delaminated areas whereas infrared thermography would miss some delaminations). International co-operation is also required to develop an accurate method of measuring the amount of chlorides in concrete which contributes to corrosion. A consensus needs also to be reached on the chloride threshold level. When assessing the effectiveness of concrete repair methods, it is important to be able to predict the remaining service life for the various repair strategies. It is important that the performance of various repair and strengthening techniques be monitored and reported. Problems with inspection of prestressing cables and state of deterioration of prestressed structures are believed to be related to grouting. Research is needed to develop better grouts and grouting techniques, quality assurance practices and to locate areas of defective grouting in existing structures.

85 Maurenbrecher, A.H.P. and Brousseau, R.J. (1992) *Review of Corrosion Resistance of Metal Components in Masonry Cladding on Buildings*, National Research Council Canada, Institute for Research in Construction Report IRC-IR 640, February.

This report presents a valuable summary of existing information on the corrosion resistance of metal connectors in masonry cladding. The corrosion of the metal connectors is associated with the carbonation of concrete/mortar and chloride ingress. Generally, as the porosity of the mortar increases with increase in water/cement ratio the carbonation becomes more severe. The effect of chlorides in mortar is not well known. It may affect corrosion in various ways: 1) act as an hygroscopic material on the surface; 2) may provide a more corrosive environment by promoting pitting. The authors report rates of corrosion of zinc on connectors in various buildings. Rates of zinc corrosion are based on the measured thickness of zinc at the time of inspection and the age of the building. The zinc loss is estimated by comparison with a tie which shows no sign of corrosion. An extensive list of references is also presented. (Note: The method used to estimate the corrosion rate assumes that the conditions which led to the deterioration observed at the time of inspection were present throughout the life of the building. It is likely that those conditions are now more severe than they were (because of the deterioration of the building envelope), indicating an increasing rate of corrosion. Such predictions of the corrosion rate of zinc can be significantly unconservative when used to predict the remaining life of the component. On the other hand, the rate of corrosion is known to decrease with time under certain environmental conditions. This decrease in corrosion rate with time has been associated to the protective nature of the corrosion products. Large variation in zinc coating thickness has been observed by others. This would have the consequence of making corrosion loss measurements quite imprecise. Although the data presented by the authors is of interest, it should be used with caution.)

86 McFarland, B. (1993) Multi-storey car parks – ”Their real worth”, *Construction Repair*, Vol. 7, No. 3, May-June, pp. 6-9.

The author gives a brief history of multi-storey car parks in the U.K. and indicates that there are from 6000 to 7000 multi-storey car parks in the U.K. today. It is estimated that 50 % of the car parks built since the Second World War suffer serious defects. This is partly due to the low standards that were used in the peak of construction of this type of structures. The ”real worth ” of parking structures is only discussed in one sentence in the conclusions where the author states that those structures are open invitations to business people and shoppers to enter retail and commerce districts. A simplified procedure to assessment and repair of parking structures is presented. A short list of some of the defects which are commonly encountered in parking structures is presented along with some commonly used tests to assess the condition of existing parking structures. Finally, a few words are said about the most common repair techniques: patch repairs, full repairs, replacement, re-alkanisation, de-salination, cathodic protection, coatings, and waterproofing.

87 McLean, R.C., Galbraith, G.H. and Sanders, C.H. (1990) Testing building materials – moisture transmission testing of building materials and the presentation of vapour permeability values, *Building Research and Practice, The Journal of CIB*, Vol. 18, No. 2, pp. 82-91.

The authors present a test procedure to evaluate the differential permeability (permeability under a humidity differential) of building materials. The test materials used were plywood, plaster board, wood, insulation, and brick. The tests showed that for plaster board and polystyrene the permeability is practically constant with changes humidity. However, with wood, plywood and brick, extremely large variations of permeability was observed with the relative humidity. The permeability can be as much as 20 times greater at RH of 95 % than at low humidity. It was found that, for brick, the permeability only increases slightly at RH lower than 75 %. However, at RH greater than about 75 %, the permeability increases rapidly with changes in RH. A study of the effect of temperature on the measured permeability showed that the temperature effect is negligible. It is the authors' opinion that, despite the fact that much data has been published on permeability of building materials, much of this data are of little use unless the test conditions are reported.

88 Millard, S.G., Gowers, K.R. and Gill, J.S. (1991) Reinforcement corrosion assessment using linear polarisation techniques, (refer to section B).

89 Miller, N.L. and Cherry, B.W. (1975) The Corrosion of Stressing Steel Embedded in Concrete, in *Proceedings of the 6th International Congress on Metallic Corrosion, Sydney, Australia*, December, pp. 1496-502.

The paper presents an investigation of a mechanism for the destruction of the protective nature of the mortar coating together with the development of testing techniques to assess the corrosion state of structures in the field, with particular application to prestressed concrete pipelines. Leaching of calcium hydroxide from the mortar by the action of acidic waters is believed to be the cause of prestressing steel corrosion. It was found that the mortar coating, when subjected to a leaching environment of carbon dioxide bearing waters, appears to show a critical initial permeability below which the densification due to hydration and carbonation is sufficient to protect the inner layers of the mortar coating from the effects of leaching.

90 Moore. J.F.A. (1981) *The Performance of Cavity Wall Ties*, Building Research Establishment Current Paper CP 3/81.

This report addresses the problem of structural and durability performance of wall ties. A section on the structural performance discusses the requirements, the function and the forces in ties. Construction workmanship and safety factors are also discussed along with the required strength and stiffness. In the section dealing with durability the author briefly presents the codes approach to durability which consists in specifying the zinc thickness. Evidence of corrosion problems of ties is then presented. Unsollicited reports sent to the author indicated that premature corrosion of wall ties is a problem. Detailed results of two surveys conducted in the U.K. are presented. Houses managed by the Property Services Agency (PSA) were part of the first survey and local authority (LA) property was covered in the second survey.

91 Morinaga, S. (1990) Prediction of service lives of reinforced concrete building based on the corrosion rate of reinforcing steel, in *Proceedings of the 5th International Conference on Durability of Building Materials and Components, Brighton, 1990*, (eds. J.M: Baker, P.J. Nixon, A.J. Majumdar and H. Davies), E & FN Spon, U.K., 7-9 November, pp 5-16.

The service life of reinforced concrete structures was predicted based on the assumption that the deterioration of the concrete results from corrosion of the reinforcing steel. The life was defined as the point at which the amount of corrosion is sufficient to crack the concrete cover. Corrosion of the rebars is assumed to result uniquely from carbonation of the concrete and chloride ingress. Empirical equations were obtained to predict the rate of corrosion of rebars in a chloride environment as a function of chloride content, water/cement ratio, diameter of reinforcing steel, and thickness of concrete cover. The concrete type (normal weight and light weight) and the orientation of the reinforcing steel were found to have a negligible effect on the rate of corrosion. The effects of temperature, relative humidity and oxygen concentration were investigated separately at various levels of chloride impregnation. Empirical equations were also obtained to predict the rate of carbonation as a function of CO_2 concentration, ratio of carbonation velocity with finished surface to carbonation velocity without finished surface, temperature, relative humidity, water/cement ratio, and time. The rate of corrosion of steel in carbonated concrete was also investigated experimentally as a function of temperature, relative humidity, and oxygen concentration. In general, it was found that the portion of the total life expanded by the carbonation process was significantly greater than the portion of the total life expanded corroding the steel in the carbonated concrete. The carbonation rate was demonstrated to be independent on the chloride content. The life prediction was therefore based on the smaller of corrosion in a chloride environment or the carbonation time. Finally, in order to predict the life of the concrete structure an empirical equation was presented to predict the amount of corrosion required to cause cracking as a function of the concrete cover and the diameter of the reinforcing steel.

92 Oshiro, T., Tanikawa, S. and Goto, N. (1991) A study on durability of structures exposed to marine environment, in *Proceedings of ACI International Conference on Evaluation and Rehabilitation of Concrete Structures and Innovations in Design, Hong Kong*, (ed. V.M. Malhotra), ACI SP-128, Vol. I, pp. 433-47.

A test building was constructed in 1984 and has been exposed to a marine environment under sub-tropical weather for six years. A new corrosion monitoring method, developed by Nippon Steel Corporation, was used to monitor corrosion potential, polarisation resistance and concrete resistance. The correlation between the corroded area ratio and polarisation resistance indicates the possibility to evaluate quantitatively the corrosion of the steel reinforcement in concrete. But there still remains uncertainties and more efforts are needed to clear those uncertainties. The surface coating (acrylic rubber) applied to the experimental building was effective in protecting the steel reinforcement from corrosion due to chloride penetration.

93 Plewes, W.G. (1977) Failure of brick facing on high-rise buildings, in *Canadian Building Digest, CBD 185*, April, pp. 185–1-4.

Buckling and spalling failures of brick cavity walls and brick veneer cladding on high-rise buildings have occurred in recent years owing to differential movements between the frame and cladding. This occurs as the brick cladding swells in a moist environment while the reinforced concrete frame shrinks and creeps in a relatively dry environment. The author discusses the mechanism of failures and points out common construction features and design details that contribute to problems. Designing to accommodate dimensional changes and attention to details such as ties and anchors is recommended. Reference is made to sources of design data on movements and the strength and installation of ties.

94 Porteous, W.A. (1992) Classifying building failure by cause, in *Building Research and Information*, Vol. 20, No. 6, pp. 350-6.

The author states that there are essentially two causes of failure of buildings. One of the causes is the result of natural causes, or the effect of the environment on the deterioration of the materials and building components. In that respect a distinction is made between cause and source of failure. The author presents a table listing some 18 sources which are all elements of three different causes, namely, dampness, movements, and chemical/biological changes. The other cause of building failure is mistakes people make. In this respect the author argues that the use of the generic term ”negligence” does little to assist understanding of the type of mistake made. From a survey of the literature, the author has identified the most common mistakes made by building professionals. Those are: the use of defective materials, overlooked site conditions, ignorance, over-emphasis on first cost, defective documentation, unanticipated consequences of change, specialist contractors' lack of knowledge, design too difficult to build well, negligence, and poor communication. The natural causes of failure may sometimes be exacerbated in their effect by human error.

95 Rostásy, F.S. and Bunte, D. (1989) Evaluation of on-site conditions and durability of concrete panels exposed to weather, in *Durability of Structures, IABSE Symposium, Lisbon*, 6-8 September, Vol. 57/1, pp. 145-50.

A model for the prediction of durability of concrete panels exposed to weather is presented. The model combines a carbonation law and on-site measurement of the permeability of the concrete cover. The model can be used for quality control or durability assessment of older structures in the course of maintenance. The model presented by the authors is a tentative model and further evaluation is needed. It is assumed that the durability of a structure can be expressed in terms of carbonation of concrete.

96 Sawada, E. (1990) Repair method for salt-damaged reinforced concrete structures, *Concrete international*, Vol. 12, No. 3, pp. 37-41.

Coating of the concrete surface is currently attracting attention as a salt damage repair method. The author describes such a system, the Asano Refresh (AR) process, developed by Nihon Cement Co., Ltd. Prepacked aggregate grouted with SBR (styrene-butadiene rubber) or other polymer cement mortar is used when a large part of the concrete cross-section must be replaced. The SBR mortar was found to give particularly good results compared to the other polymers tested. The effect of the polymer cement mortar thickness on the vapour transmission rate and chloride ion penetration was investigated. The system is further improved by the application a surface coating and a salt damage prevention finish material. The system has shown high resistance to chloride ion and water vapour penetration. The performance of the system is not compared with more conventional systems and the performance over time is not discussed.

97 Schupack, M. (1993) Bonded tendon debate, (refer to section D).

98 Shiv Kumar, S., Heidersbach, R. and Lloyd, J. (1986) The corrosion of metal components in masonry and stone-clad buildings, in *Proceedings of the 4th Canadian Masonry Symposium*, Vol. 2, pp. 826-39.

The authors discuss corrosion problems in masonry and stone building facades. References are given for field cases where problems of the exterior facade were traced back to corrosion of steel connectors or anchors. Unfortunately several of the references cannot be found in the journals cited. General recommendations to prevent corrosion related problems are given. ”Corrosion control for new buildings should emphasise adequate drainage and selection of coatings or corrosion resistant materials. ” The authors are not specific on the type of corrosion resistant material to use. A word of caution is given about the use of epoxy injection used to repair cracks. Epoxy injection can lead to increased cracking in some instances. The same can happen from the use of sealants on exterior walls. The causes of that effect are not given.

99 Stockbridge, J.G. (1978) Evaluation of terra cotta on in-service structures, (refer to section D).

100 Swamy, R.N. and Jones, R. (1991) Plate bonding technology – the painless technique of structural rehabilitation, in *Proceedings of ACI International Conference on Evaluation and Rehabilitation of Concrete Structures and Innovations in Design, Hong Kong*, (ed. V.M. Malhotra), ACI SP-128, Vol. II, pp. 1385-405.

Plate bonding has been used as an effective technique of strengthening reinforced concrete beams in many countries. A review of the role and effectiveness of the bonded plate on the deformation, stiffness, strength and failure behaviour of strengthened reinforced concrete beams is presented. Extensive testing of beams strengthened with bonded plates have shown that plating contributes to a significant reduction of deflection and cracking. High localised stresses are found to exist in the adhesive at the edges of the plate which can lead to premature debonding of the plate. The use of bonded anchor plates was found to be effective end anchorage to the bonded plates enabling the bonded plates to reach yield when the details are properly designed. Tests have shown that significantly damaged beams, when plated in the unloaded or loaded condition, can be restored to stiffness and strength values superior to those of the original unplated beams. Design guidelines are given for the design of bonded plate reinforcement. Emphasis must be placed on close supervision, high quality control and good workmanship on site. Furthermore, regular inspection, and maintenance of steel protection are also essential. The important aspect of durability is not addressed.

101 Takewaka, K., Matsumoto, S. and Khin, M. (1991) Non-destructive and quantitative evaluation for corrosion of reinforcing steel in concrete using electro-chemical inspection system, in *Proceedings of ACI International Conference on Evaluation and Rehabilitation of Concrete Structures and Innovations in Design, Hong Kong*, (ed. V.M. Malhotra), ACI SP-128, Vol. I, pp. 339-57.

The authors have performed a theoretical and an experimental analysis of the effectiveness of the half-cell potential method for inspection of rebar corrosion. The analysis showed that an approximate corrosion weight loss of reinforcement can be estimated from the potential and concrete resistivity distribution. However, the technique developed by the authors could not be applied to doubly reinforced sections (i.e. slabs with top and bottom reinforcement) because of the galvanic cell formation between the top and the bottom steel (it is impossible to determine whether the estimated weight loss occurs on the top or the bottom steel).

102 Tankut, A.T. and Ersoy, U. (1991) Behaviour of repaired / strengthened reinforced concrete structural members, in *Proceedings of ACI International Conference on Evaluation and Rehabilitation of Concrete Structures and Innovations in Design, Hong Kong*, (ed. V.M. Malhotra), ACI SP-128, Vol. II, pp. 1257-76.

The authors have investigated the performance of repaired and strengthened members of reinforced concrete structures. (Repaired members are restored to their original strength as opposed to strengthened members which have their original strength increased.) Jacketed column behaviour under uniaxial load or combined axial load and reversed cyclic bending was found to be quite satisfactory under the conditions tested if the jacket was made while the structure was unloaded. If the jacket was made under load the performance was rather poor, displaying an axial load capacity of around half of that of a monolithic reference specimen. The behaviour of beams strengthened for flexure under monotonic, repeated, and reversed cyclic loading was found to be successful with the capacity of the strengthened specimens being not less than 90 % of a reference monolithic specimen. Strengthening by epoxy glued steel plates was found to be much less promising since meticulous work was required to obtain satisfactory behaviour (ageing of the system was not investigated). Initial test results on repaired and strengthened slabs showed that the strengthened specimens performed satisfactorily, but the performance of the repaired specimens was rather poor. Reinforced concrete infills were found to increased both the strength and stiffness of reinforced concrete frames. The observed increase in strength was not as large with masonry infills. Finally the strength of epoxy anchorage for rebars in existing concrete was found to be adequate for embedment length of 15 to 20 bar diameters.

103 Tarricone, P. (1993) Plastic potential, (refer to section B).

104 Thomasen, S.E. and Ewart, C.S. (1983) Techniques for testing, analyzing and rehabilitation of terra-cotta, in *IABSE Symposium on Strengthening of Building Structures – Diagnosis and Therapy, Venezia*, IABSE reports Vol. 46, pp. 139-46.

The authors discuss observations made during field investigation of terra-cotta failures and tests of the material. The causes of terra-cotta failures have been related to expansion of the material when exposed to moisture, leading to spalling of the glaze, corrosion of ties and anchors, or buckling of part of a facade. In-situ pressure tests can be used to test the terra-cotta anchors. The same frame used for air pressure tests can also be used for water permeance tests. Stresses in various parts of a facade can be measured by mounting strain gauges on the surface and removing a section of the facade on which the strain gauges are mounted. Discussion of the use of lab tests as an assessment tool is also presented. Finally, rehabilitation techniques are briefly discussed. Those include: provision of expansion joints; replacement of corroded or missing shelf angles and ties; sealing of joints to prevent water penetration; replacement with new terra-cotta or other materials.

105 Thomasen, S.E. and Searls, C.L. (1990) Assessment of building facades in masonry and stone, in *Service Life of Rehabilitated Buildings and Other Structures*, (eds. S.J. Kelley and P.C. Marshall), ASTM STP 1098, ASTM, pp. 108-16.

Original building enclosures and past restorations and repairs to building facades require periodic evaluations to assess their safety, durability, performance, and aesthetics. Inspection techniques (non-destructive and destructive) used to assess the condition of building facades are discussed. Visual inspection should be used to determine the location where invasive inspection should be practised. The use of inspection openings and a fibre-optic borescope is recommended to minimise damage to building facades. A pachometer can be used to determine the location of masonry ties. A copper-copper sulphate half cell can then be used, with an electrical lead connected to a tie or metal conductor backup, to assess the corrosion activity in the wall ties. (If the ties are not attached to a conductive backup the technique is of limited use since each tie would have to be exposed to connect the electrical lead). Field monitoring is also important to assess whether cracks are still moving or stopped. The authors also discuss some destructive field tests. Accelerated lab tests are recommended to assess the durability of new materials for which previous service records are not available.

106 Tuutti, K. (1978) *Cracks and Corrosion*, CBI research Fo 6:78, Swedish Cement and Concrete Research Institute, 52 p.

The author demonstrates with the use of diffusion and capillary models that cracks in the concrete cover accelerate the corrosion initiation stage. In order to reconcile his theoretical findings with experimental evidence that cracking of the concrete cover does not substantially affect the corrosion of reinforcing steel, he argues that, although corrosion starts sooner, the corrosion rate is very slow due to a limiting process.

107 Vaysburd, A.M. (1993) Some durability considerations for evaluating and repairing concrete structures, *Concrete International*, Vol. 15, No. 3, pp. 29-35.

The author presents, among other things, a fairly long discussion of the effect of chlorides on corrosion of steel in reinforced concrete structures. A better discussion of the same subject has been presented by Pfeifer et. al. (1992). The existence of a chloride threshold level is not believed to exist (repeated several times). The effect of cracking of concrete on the durability of reinforced concrete is also discussed. It is believed that localised cracking is not detrimental to corrosion of reinforcing steel since only small quantities of corrodents can access the steel. However, durability of the reinforced concrete will be affected if cracking is widespread to the point where the permeability of concrete is increased. It is also believed that a good bond between the steel and the concrete is essential to prevent the accumulation of corrosion products and to ensure proper protection from the concrete (corrosion is not believed to take place when good bond is maintained). The problem here is that bond is usually broken when corrosion starts, so one can ask: ”is debonding the cause or the result of corrosion? ” The paper is interspersed with little “glitches” such as the misconception that the corrosion film on rebars in alkaline concrete passivates the underlying steel (it is the corrosion layer that is passive and it protects the steel),.

108 Waldum, A.M. (1993) Restoration of masonry facades, renders and final coats in a severe climate, (refer to section D).

109 Yuan, Y.S. and Marosszeky, M. (1991) Major factors influencing the performance of structural repair, in *Proceedings of ACI International Conference on Evaluation and Rehabilitation of Concrete Structures and Innovations in Design, Hong Kong*, (ed. V.M. Malhotra), ACI SP-128, Vol. II, pp. 819-37.

An experimental investigation of the performance of structural repair on reinforced concrete is presented. Beams with preformed cavities to simulate spalling were cast and three types of polymer modified concrete were used as repair material. The test results showed that the effect of shrinkage of polymer cement concrete on the stress and serviceability of a repaired structure is quite significant. The restrained shrinkage can lead to cracking in the repair patch and to additional tensile stresses in the substrate. Acrylic modified concrete was found to behave less favourably than styrene modified concrete. SBR showed expansion during the first 24 hours and low shrinkage strain subsequently. Consequently, SBR did not show cracking. Tests and analyses showed that the major factors influencing the performance of structural repair include free shrinkage, creep coefficient and tensile strength and ultimate tensile strain in early age properties of the repair material. Stiffness of the member and moment redistribution in the repaired structure are also important.

D Case studies

1 Akers, D.J. (1990) Evaluation of reinforced concrete masonry in a highly corrosive environment, in *Paul Klieger Symposium on Performance of Concrete*, (ed. D. Whiting), ACI SP-122, pp. 451-67.

The deterioration of a reinforced concrete masonry structure resulted from the change of usage of the building from a general warehouse to chemical storage. The corrosion of the concrete reinforcement was found to be associated to various causes: 1) the walls did not appear to be specifically designed for exposure to an aggressive environment. (no special surface coating was applied onto the absorbent concrete blocks); 2) the cover over the vertical reinforcement varied a lot due to the lack of quality control during construction (in addition, the horizontal steel should have been placed in grouted bond beams rather than placed in the mortar joint where inadequate cover results); 3) the walls were repeatedly exposed to corrosive chemicals due to carelessness when the chemicals were handled; and, 4) the raised tank slab was constructed without end drains thus allowing the spilled chemical to get in contact with the walls.

2 Anon (1993) Amoco building recladding, *Masonry Construction*, Vol. 6, No. 4, April, p. 150.

This is a follow-up story on J. Trewhitt and J. Tuchman's paper in ENR (refer to Trewhitt and Tuchman (1988) in Case Studies). The thin marble panels on Amoco Building failed after only 17 years of service. The 32 to 38 mm panels (44 000 panels) are being replaced by 5-cm thick white granite at the cost of US$60-80 million. As explained in the paper by Trewhitt and Tuchman, the panels failed by excessive bowing. No reasons are given to explain the cause of bowing of the panels. This expensive recladding job focused attention on the need for conservative design and thorough testing of thin stone exterior panels.

3 Anon (1980) Water leaks plague museum, *Engineering News Record*, Vol. 204, No. 21, 22 May, p. 39.

Less than four years after its opening, the Smithsonian Institution's US$31,5 million Air and Space Museum in Washington, D.C., experienced problems of leaking. The building is faced with marble and glass. The area in trouble is on or above grade and the water was believed to bypass flashing or come in through the marble siding. The estimated cost for repair was US$2-4 million and consisted of plugging leaks and to install insulation where needed. The reason for the needed insulation is not explained and the relation between the needed insulation and the water leaks is not explained in the paper.

4 Anon (1980) Facades: errors can be expensive, *Engineering News Record*, Vol. 204, No. 5, 24 January, pp. 30-2.

The paper reports some cases of building facades failures which have caused, or could have caused serious injury to passer-by. The cases reported consist mostly of marble slabs with some brick and stone facades. The cause of failure is reported to be ignorance, carelessness, negligence and greed. One of the cause of the ignorance with respect of building cladding is the lack of research being conducted in that area mostly because cladding does not have all the glamour that the framing and primary structure of the building has. Consequently, engineers receive little or no formation in the area of cladding behaviour and design. The need for a better understanding of cladding behaviour and better quality control during construction of building facades is emphasised. A programme of periodic inspection along with proper preventive maintenance is believed to be the key to safer cladding and prevention of costly repairs and replacement of building facades.

5 Anon (1979) GAO decries bridge deck corrosion, *Engineering News Record*, Vol. 202, No. 5, 1 February, p. 11.

According to a report by the General Accounting Office (GAO) nearly one-third of the road bridge decks in the U.S. are seriously deteriorated because of de-icing salts corroding the reinforcement. It is estimated that it will cost US$6600 million to restore them. GAO identified 162 622 bridges in 32 states with deterioration of deck problem. Many of these decks, built to last 40 years, need repair after five to ten years of operation. Research by the Federal Highway Administration (FHWA) identified four fairly successful methods to protect new concrete bridges from corrosion: membranes with asphalt overlays; epoxy-coated rebars; special concrete admixtures, including dense and latex modified overlays to slow the penetration of salts; internally sealed concrete using wax beads.

6 Anon (1973) LBJ library needs $1.8 million repair, *Engineering News Record*, Vol. 191, No. 4, 26 July, p. 11.

After two years of operation, the Lyndon Baines Johnson (LBJ) Library at the University of Texas had about 10 % of its travertine facing panels loose or cracking. Several panels came loose after only about eight months of opening. The panels are held to the reinforced concrete building with Z-type anchors. Although the reports of the investigating engineers were not released, alleged defects in the exterior stone and masonry work cited in the complaint included displacement of slabs, sealant deficiency, stone breakage, mortar that did not meet specifications, and deficiencies in the preparation of the stonework.

7 Arbogast, D. (1990) Problems affecting the service life of exterior sandstone: case study, the Burlington, Iowa Free Library, in *Service Life of Rehabilitated Buildings and Other Structures*, (eds. S.J. Kelley and P.C. Marshall), ASTM STP 1098, American Society for Testing and Materials, Philadelphia, pp. 95-104.

The author examines the causes of deterioration, the current condition , and efforts to restore the integrity of a red sandstone facing. The case study consists of the facing of the Burlington Free Library in Iowa. The library was constructed in 1896-1898. Portions of the exterior have suffered deterioration of various sorts (rising damp, spalling, exfoliation, powdering, and efflorescence). Since the time of construction, site drainage and ground water appear to have been major factors in deterioration. In 1971 the building facade was cleaned by sandblasting and a protective coat of clear silicone sealant was applied. The silicone was later found to have trapped natural salts in the surface resulting in swirls of efflorescence. Damaged areas from rising damp were later repaired by trimming back the damaged surface and parging. This shifted the problem of rising damp to the region above the applied parging. Although the sandstone was found to be of good quality with siliceous minerals as a bonding agent, excessive moisture resulted in damage of the building facade. Elimination of the source of rising damp (by altering the drainage pattern on one side of the building) was strongly recommended.

8 Beal, D.L. and Heywood, R.J. (1991) Story bridge rehabilitation study – concrete deck slabs, in *Proceedings of ACI International Conference on Evaluation and Rehabilitation of Concrete Structures and Innovations in Design, Hong Kong*, (ed. V.M. Malhotra), ACI SP-128, Vol. I, pp. 395-410.

The Story Bridge spans the Brisbane River in Australia. Built in the 1940's, the bridge showed some signs of concrete deterioration. The authors describe the methods used to evaluate the concrete quality, the level and extent of corrosion, and discuss the repair options that were considered for the bridge. The investigation of the bridge was limited to the footways on all sections of the bridge, and the roadway on the steel approach spans. In general, the concrete was found to be of variable quality. The footways, being very thin, were carbonated through their thickness. Poor detailing of the reinforcement had led to zero cover in some critical locations. The asphalt cover on the deck had protected the deck from carbonation.

9 Bjegovic, D., Ukraincik, V. and Beus, Z. (1990) Evaluation and repair of concrete structure in urban environment: case study, in *Paul Klieger Symposium on Performance of Concrete*, (ed. D. Whiting), ACI SP-122, pp. 427-50.

The authors describe the repair work that was performed on the reinforced concrete structure on the east stand of the FC ”Dinamo” stadium in Zagreb. The stand was constructed in early 1960, and 25 years later the bearing capacity was impaired. Visual inspection and laboratory tests revealed that carbonation had taken place and, in the worst areas, had reached such depths that the concrete surrounding the steel had become affected. The concrete cover was found to be very small in most placed and the concrete was excessively porous and segregated in many places. Several mortar compositions were investigated for the repair work, namely, a reference mortar, mortar with added acrylic dispersion, a mortar with added latex dispersion, a mortar with added silica fume, a mortar with added acryl and silica fume, and a mortar with added latex and silica fume. Although all the mortars performed well, the mortar with added silica fume was selected due to its lower cost.

10 Carrier, R.E. and Cady, P.D. (1973) Deterioration of 249 Bridge Decks, in *Highway Research Record No. 423*, Highway Research Board, pp. 46-57.

A survey of 249 four-year-old bridge decks was performed in the state of Pennsylvania. The objectives of the study were: to discover the extent and nature of deterioration of a large sample of fairly new bridge decks; and to evaluate the relative importance of factors causing deterioration on these decks. Fracture planes and spalls, the most serious form of deterioration from a repair cost standpoint, were found on 22 % of the decks. 95 % of the decks exhibited surface deterioration, with 97 % of the affected area attributed to wear and only 3 % attributed to disintegration of weak mortar. A total of 10,8 km of cracks were encountered on the 34,6 lane-km of deck surface observed. In essentially every case, the cracks were transverse and occurred directly above the transverse reinforcing bars. A total of 68,7 % of the decks exhibited noticeable popouts. Map cracking appeared in 20 % of the decks. Insufficient reinforcement cover was detected in nearly all cases where spalls were observed. Although other factors were identified as contributing factors to the decks deterioration, inadequate concrete cover was believed to be the governing factor.

11 Cuoco, D.A. and Velivasakis, E.E. (1989) Aluminium curtain wall panel failure, assessment and repair, in *Durability of Structures, IABSE Symposium, Lisbon*, 6-8 September, Vol. 57/2, pp. 689-94.

The authors summarise an investigation of the failure of an aluminium curtain wall panel of a high-rise building in New York City. A review of the original design and drawings indicated that the panels were not unusual. A metallurgical investigation of the failed panels indicated that the failure was due to fatigue failure of poorly welded clips. Since the building contains 9000 panels, replacement of all the panels was not practical. The panels were all inspected in the field and the loose panels, indicating failure of some of the clips, and those needing repair were repaired and all the panels were fastened at their base with stainless steel screws. A finite element analysis of a typical panel indicated that the strength of the clips was adequate w.r.t. overload provided the weld was adequate. An inspection programme was instated to detect failing panels and implement repairs before the detachment of the panels.

12 Dreger, G.T. (1989) Cementitious-cladding failure – a building façade collapse: an odyssey of failure and lessons learned, *Building Research and Practice: The Journal of CIB*, Vol. 17, No. 6, pp. 337-41.

The author gives a brief presentation of case histories of building facade failures which took place in the past decade. Facade failures have run a close second only to roof failures in overall building systems failures. The author presents a case study of a recent facade failure on the public facility in the tropical climate of Fort Myers, Florida. The Lee County Judicial Complex, completed in 1984 had a facade of painted cement-plaster stucco cladding. The cladding was furred and secured to the masonry and concrete substrate with steel ”Z” channels. The steel fasteners used to hold the vertical Z channels to the concrete blocks and concrete frame were of the same type as those used to fasten steel decks to steel framing. The fasteners were lightly galvanised to prevent corrosion before their installation. Inspection of the fasteners revealed severe corrosion due to the presence of chlorides. The chlorides were believed to come from the atmosphere and the masonry backup.

13 Eiselstein, L.E. and Caligiuri, R.D. (1988) Atmospheric corrosion of the suspension cables on the Williamsburg Bridge, in *Degradation of Metals in the Atmosphere*, (eds. S.W. Dean and T.S. Lee), ASTM STP 965, American Society for Testing and Materials, pp. 78-95.

The Williamsburg Bridge spans the East River in New York City. The main span of the bridge was opened in 1903. The bridge carries automobile and train traffic. The corrosion protection of the main cables have caused problems since the beginning of operation. At the time of construction it was decided that the 7696 wires which make up each of the cables were not going to be galvanised since the galvanising process anneals the surface of the wire and reduces its strength. Various methods of protection were tried but most were found to be inefficient. Finally, the wire were coated with linseed oil and the cables were filled with a mixture of slushing oil and graphite. The cables were then wrapped with water proof cotton and sheat iron. Given the problem plagued history of the cables, the cables were subjected to an extensive inspection and rehabilitation programme between 1980 and 1985. The authors present some of the experimental analysis and observations made during this investigation. A 91-m long segment of one cable was unwrapped to exposed the wire. Some samples of 91-m long wires were extracted from the cables. It was found that the top wires were relatively uncorroded but severe pitting was observed on the bottom wires. No explanation is provided for the gradual transition taking place from the top to the bottom wires. The breaking strength of the wires was found to be substantially reduced by the presence of pits but no signs of stress corrosion cracking were detected. The rate of localised and pitting corrosion was estimated to be between 15,2 and 63,5 μm/year.

14 Eldukair, Z.A. and Ayyub, B.M. (1991) Analysis of recent U.S. structural and construction failures, *ASCE Journal of Performance of Constructed Facilities*, Vol. 5, No. 1, February, pp. 57-73.

The authors have studied a total of 604 structural and construction failures in the United States during the period of 1975-1986. All the failure cases studied were reported in the Engineering News Record. The failures occurred either during the construction process or during the service life of the project. Over 56 % of the failure cases were associated with collapse while 4 % and 39 % were associated with loss of safety, and loss of serviceability, respectively. Most of the loss of serviceability types of failures were associated with delamination of structural composites, water penetration, and corrosion. The proportion of failures associated to commercial, bridge, and residential projects constituted 48, 21, and 18 %, respectively. It was estimated that 44 % of the failures occurred during the construction process while 56 % occurred during the utilisation phase. It is also interesting to note that over 86 % of the failure cases involved reinforced concrete elements. Although not stated by the authors, the large proportion of failure cases attributed to reinforced concrete is probably the result of the widespread use of reinforced concrete in severe atmospheres. The study also identified the primary and secondary causes of failure.

15 El-Sayed, H.A., Abd El-Wahed, M.G. and Ali, A.H. (1987) Some aspects of the corrosion of reinforcing steel in concrete in marine atmospheres, *Durability of Building Materials*, Vol. 5, No. 1, pp. 13-25.

Seven deteriorated reinforced concrete structures exposed to marine environmental conditions in Egypt were inspected. The structural members most affected were ceilings and those less affected were the beams and columns. The corrosion rate was found to vary between 0,06-0,1 mm/year. The main factors responsible for the premature deterioration of the investigated buildings were: 1) poor mix design with low cement content in the concrete; 2) inadequate thickness of the concrete cover; 3) ineffective water-proof membranes on the upper floors; 4) use of ordinary portland cement concrete instead of dense concrete; 5) use of salty water for concrete mixing; 6) use of sand and aggregates contaminated with salts; and 7) susceptibility of the used steel to pitting corrosion that lead to transgranular corrosion cracking when forces are acting.

16 Fischetti, D.C. (1990) The Montague Building and Watauga Hall: a comparison of predicted service life based on building materials, in *Service Life of Rehabilitated Buildings and Other Structures*, ASTM STP 1098, American Society for Testing and Materials, pp. 117-24.

The author discusses the renovation of two historic buildings from North Carolina. The two buildings were of similar age, size and construction. The methods used in the renovations were entirely different for the two buildings. The paper discusses the two methods in detail, and which method was more successful in terms of the predicted service life. One of the two buildings was renovated by complete replacement of the existing timber floor system by a two-way flat plate concrete floor system. In the second case, because of tax credit incentives, the existing timber floor system was reinforced with steel plates to meet the code requirements. No prediction of the anticipated service life has been presented. It only seems obvious that the complete replacement of the existing floor system by a reinforced concrete flat plate is more satisfactory on the long run than reinforcing to meet existing building codes.

17 Fookes, P.G., Comberbach, C.D. and Cann, J. (1983) Field investigation of concrete structures in south-west England, part I, *Concrete*, Vol. 17, No. 3, March, pp. 54-6.

A programme of field investigations was carried out on several structures in south-west England in order to help assess causes and types of cracking, their structural significance and remedial measures. The field investigation procedures and the rating system used for the structures is presented. One structure, a multi-storey car park, was chosen for detailed study.

18 Fookes, P.G., Comberbach, C.D. and Cann, J. (1983) Field investigation of concrete structures in south-west England, part II, *Concrete*, Vol. 17, No. 4, April, pp. 60-5.

Part I of this paper outlined a field classification system for concrete structures, and detailed mapping of members with cracks or other defects. The results of the investigation of a multi-storey car park are presented in this paper. Labour intensive crack mapping of some of the members in the structure was performed. This helped provide evidence to help identify the cause or causes of cracking. Cracking due to alkali-aggregate reaction was identified. The authors concluded that the structures or concretes at greatest potential risk are those retaining fluids or in exposed positions. The design and detailing should therefore pay attention to joints, quick water shedding and efficient weather proofing and limiting passage of moisture across or through the concrete.

19 Frauenhoffer, J. (1992). Masonry wall and window system leakage investigation for university building, *ASCE Journal of Performance of Constructed Facilities*, Vol. 6, No. 2, May, pp. 107-15.

A two-story reinforced concrete frame building with infill masonry and bands of triple-pane windows developed leakage problems shortly after its construction but the problem became reasonably evident ten years after construction of the building. The windows were leaking and condensation of moisture took place inside the units. This problem led to an investigation by the author of conditions inside the masonry wall system. The investigation revealed that a lot of moisture got behind the brick cladding and stayed there because of poor drainage resulting from bad flashing detailing. Details such as the use of a brick coping to cap the cavity at the top of the parapet, the absence of effective weep holes, short flashing which were not properly embedded in the backup wall at bolt heads protruding from the shelf angles, and mortar in the cavity were found to be at the source of much of the problems. The seal for the triple glaze windows was found to be inadequate. Recommendations were made to place adequate flashing, replace the brick coping by a conventional aluminium coping flashing, to replace the shelf angles at the top of the windows with stainless steel angles, to place additional control joints, to replace the triple glaze windows with maintainable separate pane units, and to remove the mortar squeeze from the back of the brick veneer.

20 Freyermuth, C.L., Klieger, P. and Stark, D.C. (1970) Durability of concrete bridge decks – a review of cooperative studies, in *Highway Research Record No. 328*, Highway Research Board, pp. 50-60.

The paper summarises the results of concrete bridge deck durability studies made by the Portland Cement Association in co-operation with the U.S. Bureau of Public Roads and ten state highway departments. Condition surveys were made of over 1000 bridge decks. Attention was given to scaling, spalling and cracking. Scaling was not found to be a problem for the bridges surveyed. The observed scaling was found to be related to deficiencies in the air entrainment and inadequate deck drainage. Among the various types of cracking observed, transverse cracking was the form most encountered. Cracks over bars, shallow concrete cover over bars, and permeable concrete allow the de-icing chemical solutions to reach the bars and cause corrosion. Transverse cracking was found to be associated with shrinkage of the concrete. Spalling was found to be the result of corrosion of the top layer of reinforcement as a result of the use of de-icing chemicals. The authors conclude by saying that inadequate construction practice has played a major role in the development of many present durability problems. Examples of shallow cover, ponding of water in gutters, high water-cement ratio pastes at the wearing surface, excessive variation in air content, improper finishing, inadequate curing, and other durability reducing practices have all been observed.

21 Gardner, L.L. and DeWitt, C.A. (1992) Moisture damage in South Carolina housing, *Building Research and Information*, Vol. 20, No. 3, May-June, pp. 166-70.

The results of a survey of 6000 homeowners/occupants in South Carolina indicated that 69 % of the 1329 respondents complained about moisture problems in their home. It is reported that estimates of annual damages to homes in the United States from moisture and moisture-related insects include over US$2000 million in 1978-80 and US$15600 million by the year 2000. The survey conducted by the authors identified several causes of substructure and superstructure problems. All the listed causes of moisture problems are found to be the result of: violation of residential building codes (shallow crawl space with no vapour barrier on the floor of the crawl space); poor workmanship by the builder (no flashing or improperly installed flashing); lack of maintenance of the home; poor or inadequate design details (water draining towards the house).

22 Green, P. (1988) Structures need a low-sodium diet, *Engineering News Record*, Vol. 220, No. 2, 24 March, pp. 28-31.

It is estimated that the backlog of corrosion induced distress in bridges would cost the U.S. between US$16000-24000 million. However, each year adds another US$400 million of deteriorated decks to the backlog. The author discusses the use of cathodic protection as a desirable system for protection of steel reinforcement against corrosion. Although the system has been used extensively on bridge decks, its use is still limited on other parts of bridges. The system consists of placing an inert anode and passing a small low voltage direct current between the external anode and the rebar to charge the steel negatively, making it the cathode. The Federal Highway Administrator has commented the system as the only rehabilitation system that has proven to stop corrosion in salt-contaminated bridge decks. Research programmes aimed at developing guidelines to quantify, analyse and repair bridges are briefly discussed. The efforts of individual states at developing and experimenting with cathodic protection are also briefly discussed.

23 Green, P. (1986) Owners reclad damaged masonry, *Engineering News Record*, Vol. 216, No. 10, 6 March, pp. 10-1.

The author discusses problems arising from the use Dow Chemical's mortar additive Sarabond. The additive has been found to leach out chloride ions which then cause accelerated corrosion of unprotected steel. Many brick building facades had to be replaced because of the accelerated corrosion of the ties embedded in the mortar. It is reported that over 100 building owners had engineers investigate problems related to the use of Sarabond in mortar for building facades. See related paper by Rosenbaum and Powe (1989) in Case Studies.

24 Haver, C.A., Keeling, D.L., Somayali, S., Jones, D. and Heidersbach, R.H. (1990) Corrosion of reinforcing steel and wall ties in masonry systems, in *Masonry: Components to Assemblages*, (ed. J. H. Matthys), ASTM STP 1063, American Society for Testing and Materials, pp. 173-93.

The authors present the results of experimental work on two different masonry walls. The first was a concrete block wall situated near the beach and exposed to sea water. The second was a 12-year old section of veneer wall from the Howe Development Center in Tinley Park, Ill. Both walls were examined for corrosion activity using half cell potential readings, chloride sampling, pH measurements to determine the extent of carbonation in the mortar, and energy dispersive X-ray analysis to determine the composition of the corroded surface. The half cell potential measurements on the beach wall indicated potentials significantly lower than the potential for which there exists a probability of 90 % for corrosion. Chloride contents near the rebars (0,45 %) were significantly over the commonly accepted threshold (0,20-0,35 %). The mortar and grout surrounding the reinforcement was found to be carbonated with a pH of 8,5 to 9,5. Although all the measurements indicated that significant corrosion should have been taking place, very little corrosion was observed in the wall. The veneer wall was inspected because it had to be replaced due to excessive cracking and corroding wall ties. The tests showed only a nominal amount of chloride in the mortar in which corroded galvanised ties was embedded. The pH values ranged from 9,5 near the surface to 10,5 in the centre. EDX analysis indicated that the only possible source of corrosion must have been moisture. As a result of their investigation, the authors recommend the use of more resistant wall ties since moisture inside wall cavities and brick facades cannot be avoided.

25 Heidersbach, R. and Lloyd, J. (1985) Corrosion of metals in concrete and masonry buildings, in *Corrosion/85, Boston, Massachusetts*, March 25-29, paper no. 258.

The authors begin by outlining some actual field cases where problems in buildings have been associated to corrosion of metals. Most of the paper discusses specific problems related to corrosion of metals in concrete and masonry buildings. Control of corrosion is basically done by providing adequate ventilation inside wall cavities, especially near connectors, and providing adequate drainage in order to reduce the time of wetness. The paper does not present any new research but only refers to already published literature. Cathodic protection, an increasingly popular corrosion control technique for highway structures is believed to have only very limited applications in buildings.

26 Heidersbach, R., Borgard, B. and Somayaji, S. (1987) Corrosion of metal components in masonry buildings, in *Proceedings of the 4th North American Masonry Conference, Los Angeles, California*, August, paper no. 68.

The first part of this paper is almost identical to the above paper by Heidersbach and Lloyd (1985). Talking about galvanic couples, the authors guard against using galvanised bolts with a stainless steel shelf angle and recommend the use of stainless steel shelf angle and stainless steel bolts in preference of galvanised shelf angle and bolts. The effect of anode to cathode area ratio is not mentioned. In a later section the authors advocate an increase in use of zinc coating and recommend the use of galvanised shelf angles over plain carbon steel shelf angles. It is believed that atmospheric corrosion tests conducted with flat specimens at various exposure sites cannot be used to estimate the corrosion rate of metals inside walls. (The authors do not recognise the potential use of those test results once the nature of the environment inside walls has been defined). Referenced cases of corrosion problems in masonry buildings with metal-stud backup systems are reported. Although stiffer structural design have since been recommended for metal studs to prevent excessive cracking of the facade, the reported cases of metal stud corrosion did not occur on buildings with stiffness problems. Examples are given where galvanised metal studs severely corroded and galvanised masonry ties failed due to corrosion. It is apparent that the most viable way of preventing corrosion related problems is to minimise corrosion by adequate ventilation and drainage. The paper illustrates that the use of metallic coatings such as zinc can delay corrosion but cannot prevent it if environmental conditions are severe.

27 Hergenroeder, M. (1990) Long-term behavior of prestressed girder slabs in cattle stables, in *Proceedings of the 5th International Conference on Durability of Building Materials and Components, Brighton, 1990*, (eds. J.M: Baker, P.J. Nixon, A.J. Majumdar and H. Davies), E & FN Spon, U.K., 7-9 November, pp. 325-31.
The author reports observations of an investigation of the structural state of prefabricated prestressed girder slabs in cattle stables. Two such slabs collapsed in the early 80's. The collapses were found to be of a brittle nature with no cracking or signs of distress prior to the collapse. Stress corrosion cracking at the areas where insufficient concrete cover existed was found to be the cause of fracture of the susceptible prestressing tendons. Applying Bayes decision theory, the amount of damaged slabs was estimated based on the results from a very limited number of inspections. A greater number of slabs were then investigated using magnetic field measurements to detect wire fractures and infra-red thermography to detect imperfections in the concrete. The results of the statistical analysis where thus confirmed.

28 Hookham, C.J. (1992) Service life prediction of concrete structures – Case Histories and Research Needs, *Concrete International*, Vol. 14, No. 11, pp. 50-3.
The service life prediction of reinforced concrete structures is based partly on experience and mathematical formulation of transport of chlorides and carbonation of the concrete. Although the area has been researched quite well there are still areas where further research is needed in order to obtain better and more reliable predictions of service life. The author presents three case studies where service life prediction was required and existing knowledge was used. In the first case a wharf structure, subjected to the impact of a crane, was deteriorating quite severely under the action of sea salt. In order to allow for long-term plans for the facility the service life had to be determined after repair of the damaged elements was made and protection was provided to reduce the rate of corrosion. The accuracy of current mathematical models was questioned and need for more research to characterise the micro climates identified. The other case studies (which are of a general nature) consisted of a power plant structure and radioactive waste repositories. Future research needs as identified by others are listed.

29 Keller, H. (1993) Personal conversation, 4 August.
Heinz Keller, from Keller Engineering Associates Inc., during a telephone conversation with G.Y. Grondin, conveyed the following information. The building envelope, with roofing and exterior walls, is the element causing the most problems with durability. Parking garages are also a major problem. Balcony slabs and their metal railings are believed to be as bad as parking garages. Although brick cladding has many problems, poor detailing, resulting in water leakage, is the main cause of those problems. Materials are not as much at fault as poor workmanship. Other specific problems mentioned were: on low rise buildings, the back entrance door threshold suffers of premature rotting in many town houses (hundreds of them had to be replaced not too long ago); wooden window frames also suffer from poor detailing and rot after 10-15 years of service (metal frame windows do not pose problems other than air leakage of aluminium frame windows which need to have their gasket replaced). In general, most durability problems are caused by poor workmanship. Repair techniques make use of knowledge that was available over 20 years ago through DBR reports. The main problem seems to be the dissemination of that information to construction workers and contractors.

30 Keller, H., Trestain, T.W.J. and Maurenbrecher, A.H.P. (1992) The durability of steel components in brick veneer/steel stud wall systems, in *6th Conference on Building Science and Technology, Toronto*, March, pp. 83-104.
The paper presents the results of a field investigation of the durability of metal components in brick veneer/steel stud (BV/SS) exterior walls. A total of eight buildings were inspected using destructive methods. Two buildings each from St-John's Newfoundland, Montreal, Toronto, and Calgary were selected. The wall ties, screws and bottom tracks were found to be the most deficient components of the BV/SS system.

31 Kellermeyer, K.B. and Chin, I.R. (1986) Lessons learned from investigations of over 500 distressed masonry and stone facades, in *Building Performance: Function, Preservation, and Rehabilitation*, (ed. G. Davis), ASTM STP 901, American Society for Testing and Materials, pp. 152-64.

The authors and their colleagues have investigated over 500 buildings with distressed masonry and stone facades. Although the location of the these buildings is not specified, it is expected that all are from the U.S. The vast majority of the distressed conditions observed in masonry and stone facades occurred as a result of : 1) inadequate provisions within the facade to accommodate volumetric changes of materials; 2) inadequate provisions within the facade to accommodate differential movements between the facade and the structural frame of the building to which it is attached; 3) improper design and installation of expansion and control joints; 4) inadequate detailing and construction to reduce the entry of water into the facade to acceptable levels and to prevent the entry of water into the interior of the building; 5) the use of wind suction forces in the design lower than the actual wind suction forces.

32 Kudoh, P., Hirotani, A., Moriwake, A. and Yasuda, M. (1991) Evaluation and rehabilitation of concrete structures and innovations in design, in *Proceedings of ACI International Conference on Evaluation and Rehabilitation of Concrete Structures and Innovations in Design, Hong Kong*, (ed. V.M. Malhotra), ACI SP-128, Vol. II, pp. 1293-307.

Extensive repair work was performed on a 12 year old jetty. The cause of damage was chloride ingress. The protective system for the repairs consisted of rebar coating with anti-corrosive material (the exact nature of the material is not mentioned) the use of polymer cement for patching, and the whole deck was painted. The authors report the condition of the repairs five years later. It was found that the surface coating had satisfactorily protected the deck from chloride ingress.

33 Liam, K.C., Roy, S.K. and Northwood, D.O. (1992) Chloride ingress measurements and corrosion potential mapping study of a 24-year-old reinforced concrete jetty structure in a tropical marine environment, *Magazine of Concrete Research*, Vol. 44, No. 160, September, pp. 205-15.

A corrosion survey was made of a 24-year old reinforced concrete jetty structure in Woodlands in the northern part of Singapore. The survey of two piles covered the mean tidal zone, the upper tidal zone at 0,8 m above the mean tide level, and the splash zone at 1,3 m above the mean tide level. The chloride content in the concrete increased with height above the mean tide level. The chloride concentration profiles measured at different locations in two piles were in good agreement with those calculated using the diffusion model. The values of the chloride diffusion coefficient D, which were calculated from the chloride profile measurements, ranged from $2{,}13 \cdot 10^{-8}$ cm^2/s at the mean tidal zone to $5{,}50 \cdot 10^{-8}$ cm^2/s at the splash zone. The upper tidal zone and the splash zone, those zones with the highest chloride concentration, were found to be most susceptible to corrosion. The authors discuss in some detail the mechanisms of chloride ingress, the diffusion model, and the chloride corrosion threshold.

34 Litvan, G.G. (1990) Performance of parking garage decks constructed with epoxy coated reinforcing steel, in *Proceedings of the 5th International Conference on Durability of Building Materials and Components, Brighton, 1990*, (eds. J.M: Baker, P.J. Nixon, A.J. Majumdar and H. Davies), E & FN Spon, U.K., 7-9 November, pp. 421-32.

It has been established that suspended garage decks constructed with epoxy coated steel appear to be prone to excessive cracking. The author summarises some of the observations made while inspecting some parking garages. Extensive cracking of the slabs has been reported even for a garage built in 1988 where no cars had parked on one of the floors. The author gives an historical review of the use of epoxy coated reinforcing steel. The cause of the excessive cracking is not known. However, it appears to be reasonable to accept the hypothesis that the defect is related to the decreased adhesion between the epoxy-coated steel and the concrete matrix compared to that existing between bare steel and concrete. A protective membrane is recommended to protect the bottom bare steel mat from the chloride and water ingress through the numerous cracks.

35 Litvan, G.G. (1991) Deterioration of parking structures, in *2nd International Conference on Durability of Concrete, Montreal, Canada*, (ed. V.M. Malhotra), ACI SP-126, Vol. 1, pp. 317-34.

The author makes some observations from a survey performed on 49 parking garages as part of a five year research project. Condition surveys, where they were made, indicate that prior to repairs delaminations in the suspended decks ranged between 0-60 % in terms of the floor area, the average per garage being 6,9 %. Polyurethane has been the most frequently used membrane, comprising 31 % of the applications, followed by mastic asphalt and rubberised asphalt 23 % and 15 %, respectively, latex neoprene 15 %, and coal tar epoxy, 5 %. In the course of field survey excessive cracking of suspended deck constructed with epoxy-coated rebar was noted in recently built garages. A cause of the excessive cracking is not given. Preliminary results of the survey indicate waterproofing membranes are an effective means to reduce the moisture content in the slab.

36 Litvan, G. and Bickley, J. (1987) Durability of parking structures: analysis of field survey, in *Concrete Durability – Katharine and Bryant Mather International Conference*, SP-100, American Concrete Institute, Vol. 2, pp. 1503-25.

It is estimated that the cost of repairs of parking garages in Canada exceeds CD$3000 millions. Although the technology required to avoid those problems is available from the bridge construction industry, it is believed that it would be unnecessarily expensive to try to use the same technology for parking garages. In other to try to develop techniques of construction more suitable to parking structures, a survey of parking structures was performed. The authors are reporting some observations for the survey of some 215 structures. It was found that durable parking structures can be built without using exotic methods, and the poor condition of existing structures is mainly attributable to design and construction practice that fall short of those required by the environment. It is estimated that almost all garages built until very recently by conventional methods will require rehabilitation. Repair by patch and waterproof method was found to reduce the rate of deterioration substantially but does not stop deterioration. The use of a waterproof membrane on top of chloride contaminated concrete was not found to cause detrimental effects. No relation was found to exist between extent of delamination and that of cracking, no relation was found between the compressive strength of concrete and the extent of delamination. It was found that the chain drag technique is as good in detecting delamination as half-cell potential measurement.

37 Manning, D.G. (1984) Accelerated corrosion in weathering steel bridges, in *Canadian Structural Engineering Conference*.

The paper describes an investigation of the performance of weathering steel in the highway environment where de-icing salts are used extensively in winter maintenance operations. A review of some research projects on corrosion of weathering steel is presented. Based on the review of the literature and field inspection of some 61 bridges in southern Ontario, it is concluded that most of the steel work in weathering steel bridges is performing satisfactorily. Problem areas are locations were debris and moisture can accumulate such as on horizontal surfaces near expansion joints. Blast cleaning was recommended in order to remove the mill scale which retards the formation of the protective patina on the steel surface. Mill scale, because it is cathodic, could lead to pitting of the steel in areas where the mill scale has been damaged.

38 Maurenbrecher, A.H.P. and Suter, G.T. (1989) A loadbearing clay brick masonry deterioration problem: monitoring of temperature and moisture, in *Proceedings of the 5th Canadian Masonry Symposium, Vancouver, B.C.*, 5-7 June, pp. 771-9.

The damaged exterior walls of two five-storey apartment buildings were monitored for a period of nine months. The temperature and moisture were monitored on the brick surfaces. The continuous monitoring indicated that both faces of the brick were subjected to freeze-thaw action. The moisture content of the brickwork was found to be strongly influenced by rain. The contribution from air exfiltration was not really noticeable. (see also Suter and Maurenbrecher (1989) in Case Studies).

39 Morishita, K., Sato, Y. and Fuse, T. (1989) Deterioration diagnosis and repair techniques for NTT buildings, in *Quality for Building Users Throughout the World, CIB 89, XIth International Congress, Paris*, 19-23 June, Theme II: Lifespan of Buildings, Vol. 1, pp. 167-76.

The Nippon Telegraph and Telephone Corporation (NTT) owns approximately 30 000 buildings with some being more than 30 years old. Some of those buildings suffer from deterioration. The paper introduces a method of diagnosing the durability and repair techniques for reinforced concrete buildings. A procedure is outlined which is used by NTT to evaluate the durability of existing buildings. The procedure consists of field surveys to assess the present state of deterioration and then the deterioration progress is assessed. Parameters used to assess the state of deterioration are cracking, depth of carbonation, depth of chlorides penetration, reinforcing bar corrosion, etc. Future deterioration is estimated using the time when a certain proportion of reinforcing steel are corroded. This time is determined by predicting the depth of concrete carbonation and chloride infiltration. Criteria for reinforcing steel corrosion based on experimental results are presented. Deterioration of existing building is evaluated using: infrared inspection to detect exfoliation, gas leakage measurement to detect water leak, and use of sample photographs to evaluate the state of surface deterioration. Some guidelines are given to repair deteriorating buildings. The repairs address only two problems; carbonation of concrete and consideration to restrain it (no method given), and methods to seal cracks.

40 Nanni, A. and Lista, W.L. (1988) Concrete cracking in coastal areas: problems and solutions, *Concrete International*, Vol. 10, No. 12, pp. 67-72.

The authors present case studies of concrete cracking caused by reinforcement corrosion due to a coastal environment in combination with improper workmanship. Identified causes of concrete damage were: 1) lack of adequate cover for both or either of the longitudinal and transverse steel; 2) incorrect selection and installation of balcony parapets (e.g. selection of aluminium for posts embedded in concrete); 3) use of poor quality concrete (e.g. use of sea water contaminated aggregate, use of low strength and high permeability concrete). The proposed repair process is local demolition, cleaning and protection of the reinforcing steel, and patching to sound concrete.

41 National Institute of Standards and Technology (1990) *Performance of Structures During the Loma Prieta Earthquake of October 17, 1989*, (ed. H.S. Lew), NIST Special Publication 778,. U.S. Department of Commerce, January.

This is report relating some of the damages caused to structures by the Loma Prieta Earthquake. Numerous masonry building facades and veneers collapsed as a result of the earthquake. One of the causes of collapse was loss of anchorage due to excessive corrosion of the brick ties (common nails). Another instance was cited where the wood frame backup had rotten. Although the reporter does not mention whether ties were present and pulled out from the rotten wood, it is possible that this could have been a contributing factor.

42 Nehil, T.E. (1991) Rehabilitating parking structures with corrosion-damaged button-headed post-tensioning tendons, *Concrete International*, Vol. 13, No. 10, pp. 66-73.

Many of the early long-span concrete parking structures were built using the paper-wrapped button-headed wire post-tensioning system which was brought to the U.S. from Europe. After 10 to 15 years of service, many of these parking structures have been found to be in a deteriorated condition with varying degrees of damage to the post-tensioning system. The author reviews some of the problems experienced with unbonded paper-wrapped button-headed post-tensioning systems in parking garages and presents general guidelines for rehabilitation strategies. Typically, the unbonded prestressing tendons were greased and wrapped in Kraft paper. Corrosion protection was provided only by the concrete. Near the dead end anchorage the wires were unprotected and the concrete cover at the dead anchor could be less than 15 mm. Many other deficiencies are identified at the live anchorage, splices, etc… The lack of internal protection against corrosion in the paper wrapped tendon often requires the installation of some external form of protection against the intrusion of water and chlorides. The author discusses some of the non-destructive techniques of inspecting a parking garage and concludes that the only reliable technique consists of exploratory excavation into the slab at locations most susceptible to corrosion attack. The exploratory work must be conducted very carefully since the prestressing system is unbonded.

43 Page, A.W. (1992) The design, detailing and construction of masonry – the lessons from the Newcastle earthquake, *Australian Civil Engineering Transactions*, Vol. CE34, No. 4, pp. 343-53.

The author presents some observations made on masonry buildings following the December 1989 earthquake in Newcastle, Australia. Many problems were found in masonry buildings mainly because of the poor workmanship and substandard materials. Inspection of many buildings showed that mortar joints were badly executed, poor mortar to brick bond was observed, many buildings had several ties which were not embedded in the brick whyte. In older structures, wall tie corrosion was identified as being a significant factor in the damage incurred to buildings. Corrosion was found to be worst in cavity construction in the mortar joint of the outer whyte of exposed walls. The only effective way of detecting corrosion is to identify the tie location, and remove the appropriate brick in the outer skin to expose the tie.

44 Roper, H. (1989) Durability aspects in maintenance, repairs and rehabilitation, in *Durability of Structures, IABSE Symposium, Lisbon*, 6-8 September, Vol. 57/2, pp. 651-62.

The author proposes various definitions for the term durability. The design life of a structure is considered in terms of repair requirements. Investigative procedures are considered and this leads to results of surveys on concrete structures in Australia and suggestions for repair procedures. The author reports that in an Australia wide survey on repair problems, 621 reported cases were related to structural problems, whereas 484 were related to diminution of functional efficiency and surface aesthetics of the structures. Some 590 structures were reported to have been the object of concern to the owner and public, but not to the engineer.

45 Rosenbaum, D.B. and Powe, C. (1989) Dow wins in court at last; jury finds its mortar additive Sarabond is safe, *Engineering News Record*, Vol. 222, No. 20, May 18, pp. 7-8.

A court decision is presented where Dow Chemical Co. was found not liable for the cracking of building facades on two high-rise buildings in Denver. The buildings were built in the early '70s and latter showed signs of corrosion of the anchor bolts for prefabricated brick panels. Sarabond is used to increase the strength of the mortar. It is an organic chloride additive which tends to break down as it weathers, releasing salts. According to published news reports, Dow has paid out roughly US$100 million against Sarabond claims. The jury decided that Sarabond makes mortar more corrosive, but at too slow a rate to do damage. The additive was taken off the market in 1982. See related paper by Green (1986) in Case Studies.

46 Sarkar, S.L., Chandra, S. and Rodhe, M. (1992) Microstructural investigation of natural deterioration of building materials in Gothenburg, Sweden, *Materials and Structures / Matériaux et Constructions*, Vol. 25, No. 151, August/September, pp. 429-36.

Site investigation of some concrete and masonry structures in Gothenburg, Sweden, revealed carbonation to have affected number of concrete structures. The synergistic freeze-thaw action was found to have accelerated the deterioration process in some cases. The masonry structures investigated have undergone a different type of deterioration mechanism. Sulphate efflorescence had affected the brickwork to various degrees resulting in loss of aesthetic beauty to delamination between the brick and the rendering. Water is required for both mechanisms to be activated. The deterioration process and the degradation products were identified using microstructural study.

47 Schupack, M. (1993) Bonded tendon debate, *Civil Engineering*, Vol. 63, No. 8, pp. 64-6.
Bonded tendons have been banned in bridge construction in the U.K. as a result of a recent collapse of a prestressed concrete bridge (the Yns-y-Gwas Bridge). The main problem which lead to its ban is the difficulty of inspecting and replacing tendons. Corrosion has been found to occur when air pockets are created during the grouting operation. The author has demonstrated in the past that the voids are the result of grout bleeding which leads to the creation of water pockets in the post-tensioning ducts. The author has developed an admixture which prevents the water from separating from the grout and will prevent the formation of voids. The author reports about an investigation that was made for the Connecticut DOT. A 14 spans post-tensioned bridge had to be demolished because of the necessary widening of the roadway. Longitudinal cracks had been detected along 41 of the 112 T-beam sections. An examination of some of the beams showed that various types of grout had been used. Some grouts were expansive grouts which incorporate iron and chloride. Extensive corrosion of the prestressing steel was detected where the chloride containing grout had been used. Corrosion was also intermittent which indicates that spot checks only could have missed detecting the corroding wires. Inspection of wires in non-contaminated grout showed very good performance of the system even where high water-cement grout had been used. Corrosion in that case occurred only where air voids had formed either between the individual wires or between the wires and the post-tensioning duct. Concrete cracks parallel to the reinforcement was found to cause corrosion of the steel even if the concrete was not carbonated.

48 Schupack, M. (1991) Evaluating buildings with unbonded tendons, *Concrete International*, Vol. 13, No. 10, pp. 52-7.
The author discusses some of the problems existing with buildings where unbonded tendons have been used. His observations are similar to those of T.E. Nehil (1991). A preliminary evaluation of the condition of the anchorages is done using a vacuum cell on the surface of the concrete at the anchorage zone. This essentially gives an assessment of the porosity of the concrete protecting the anchors.

49 Schupack, M. (1991) Corrosion protection for unbonded tendons, *Concrete International*, Vol. 13, No. 2, pp. 51-5.
The author concludes that unfortunate detailing and construction practices of the past have caused a number of problems which have reflected on the ability of unbonded tendons to provide corrosion-free structures. Such factors include but are not limited to: 1) grossly inadequate cover; 2) poor concrete quality; 3) loose sheathing; 4) construction abuse of the tendons; 5) improper filling of the stressing pockets; 6) improper location of stressing anchors; 7) cracking due to inadequate design consideration of structural restraints; and, 8) inadequate drainage.

50 Shoya, M., Tsukinaga, Y. and Sugita, S. (1991) Assessment of deterioration in sea-side concrete bridges located in cold regions by in-situ tests on the surface layer of concrete, in *Proceedings of ACI International Conference on Evaluation and Rehabilitation of Concrete Structures and Innovations in Design, Hong Kong*, (ed. V.M. Malhotra), ACI SP-128, Vol. 1, pp. 69-83.
The authors describe the results of field investigations on the state and the causes of deterioration in three sea-side concrete bridges located in the most northern district of Japan. Two of the bridges investigated were 14 years old and the third bridge was 52 years old at the time of inspection. The visual damage of the two 14-year old bridges consisted of cracking and some efflorescence. The older bridge showed some scaling and peeling of surface mortar resulting in exposure of the coarse aggregates, longitudinal cracks along the main reinforcement, and efflorescence. In all the three bridges, corrosion of the reinforcing steel resulted in partial loss of cross-sectional area of the rebars and spalling of the cover. The deterioration which resulted in severe pattern cracking was identify to be due to alkali-silica reaction in some members. The authors describe some simple field tests which showed some potential to assess concrete quality.

51 Stockbridge, J.G. (1978) Evaluation of terra cotta on in-service structures, in *Proceedings of the 1st International Conference on Durability of Building Materials and Components, Washington DC 1978*, ASTM STP 691, ASTM, pp. 218-30.

Although terra cotta is not used very much nowadays on new buildings, it was used extensively as decorative trim and facade cladding in the period 1850 to 1950. As a result, several buildings in use today have a terra cotta facade. The author reports some incidents where failure of terra cotta cladding has lead to loss of life. He also reports on an inspection of 2458 terra cotta clad buildings in Chicago which showed that 45 % of the buildings inspected had loose and potentially unsafe pieces of terra cotta. The prime causes of those poor conditions were identified to be: built-in problems in the original design of the facades (no drainage path for the water penetrating the cladding, lack of details to allow contraction of the structural frame and expansion of the cladding, lack of expansion joints); infrequent inspections and unqualified inspections, and; need for technical support for qualified inspectors (the author outlines some of the tests developed to assess terra cotta facades in the field and points out that better techniques are required to make the inspections more reliable and efficient).

52 Suter, G.T. and Maurenbrecher, A.H.P. (1989) A loadbearing clay brick masonry deterioration problem: case study, in *Proceedings of the 5th Canadian Masonry Symposium, Vancouver, B.C.*, 5-7 June, pp. 757-69.

The paper presents the findings of an investigation on a deteriorating exterior loadbearing masonry wall in a building constructed in 1973-74. The deterioration consisted of extensive brick spalling, particularly on the north-east elevation. As a measure of durability, the saturation coefficient (C/B ratio) was determined for bricks obtained from the deteriorated walls. The bricks were found to comply with the 1965 edition of the Standard but not with the 1987 edition. The resulting low freeze-thaw durability is believed to be the cause of the deterioration. The exterior coating applied at an earlier time to reduce the rate of moisture ingress is also believed to have caused an acceleration of the deterioration by slowing down the drying process of the brick. As a remedial measure, exterior insulated cladding was suggested. (see also Maurenbrecher and Suter (1989) in Case Studies).

53 Tolstoy, N. (1989) The design of field investigations for estimating the extent of building failures, in *Quality for Building Users Troughout (sic.) the World, CIB 89, XIth International Congress, Paris*, 19-23 June, Theme II: Lifespan of Buildings, Vol. I, pp. 187-96.

The importance of knowing the extent of failure in building elements and materials is outlined. It is from such knowledge that research and development can be directed towards the areas where it is most needed. The author outlines some statistical considerations in the selection of buildings for the survey. It is believed that one of the main difficulties in these investigations is to define what constitutes a failure. Failure has been defined as need for unexpected maintenance. In certain investigations a questionnaire is distributed before inspection. Useful tips on the preparation of such a questionnaire are given and the follow-up inspection are given. Some results of field survey are briefly presently. A survey conducted in 1984 showed that in low-rise housing the sub-structure and mechanical installations were the building elements demanding the greatest expenditure. In high-rise buildings the external walls and windows require the greatest expenditure. Repairs to concrete balconies, replacement of built-up roofing felt, replacement of timber windows and rerender were listed as the most common repair works on high-rise buildings.

54 Tolstoy, N. (1984) Field investigations of moisture in buildings, in *Proceedings of the 3rd International Conference on Durability of Building Materials and Components*, *1984 Espoo, Finland*, 12-15 August, Vol. 2, pp. 422-32.

The author outlines the various causes of water and moisture problems in roofs, walls, and floors of buildings. Recommendations and guidelines are given for the field investigation of buildings showing moisture problems. It is important that a standard procedure be used to investigate such problems in order to avoid the tendency of certain site investigators to always look for the same source of problem since moisture problems can result from several sources.

55 Tolstoy, N., Andersson, G., Sjöström, C. and Kucera, V. (1990) Statistical field survey of exterior building materials degradation, in *Proceedings of the 5th International Conference on Durability of Building Materials and Components, Brighton, 1990*, (eds. J.M: Baker, P.J. Nixon, A.J. Majumdar and H. Davies), E & FN Spon, U.K., 7-9 November, pp. 133-8.

The authors present an investigation of external materials on buildings in the greater Stockholm area. One of the main goals of the investigation was to inspect and account for the observed deterioration of the materials. For roof, windows and walls, an assessment was made of the status of surface finish and underlay. Both were evaluated on a three point scale as either intact, with minor damage, or in need of repair. Record was also made of cause of status and age of material as well as surface treatment. Visual inspection and study of environmental factors such as SO_2 level, proximity to traffic, local pollution sources, proximity to salt water and NO_x formed the basis for the evaluation of the status and causes of deterioration. The surface finish status of wood, metal and rendering was shown to be inferior in areas with heavier sulphur dioxide concentrations. Proximity to salt water revealed no connection with the status of materials (probably because of the small number of buildings in exposed areas inspected). A strong connection was found between the degree of fouling and the SO_2 concentration, the distance from road traffic and the distance from a local pollution source. The maintenance periods were statistically estimated by using the classification of the condition of surface finishes and materials distributed in different age classes.

56 Trewhitt, J. and Tuchman, J. (1988) Amoco may replace marble on Chicago headquarters, *Engineering News Record*, Vol. 220, No. 12, March 24, pp. 11-2.

Some 43 000 marble panels are used for exterior cladding on the Amoco Building in Chicago. It is estimated that 30 % of the panels have bowed between 13-25 mm. The rest of the panels have bowed by as much as 13 mm or not at all. The panels are 127×112 cm^2 and are either 32 or 38 mm thick and weigh about 125 kg. As a temporary safety measure, the panels are being restrained by stainless steel straps. Investigations are being carried to assess the safety of the panels and to identify possible alternatives for the replacement of the panels. It is estimated that the necessary repairs could cost up to US$20 million. See related paper by Anon. (1993) in Case Studies.

57 Waldum, A.M. (1993) Restoration of masonry facades, renders and final coats in a severe climate, *Building Research and Information*, Vol. 21, No. 1, pp. 51-5.

The author describes the restoration work on an old masonry building facade with a render located in Norway. A brief description of the weather conditions acting on buildings in Norway is presented. The main problem with this type of facade is believed to be the incompatibility between the original material and the material used for repair. The use of high strength or normal strength mortar for repairs has been found to cause rapid deterioration of the renders since more stress is put on the old render as the patch work is stiffer. The use of low strength mortar was found to give satisfactory results. It was found that both organic and inorganic decorative coatings can be used successfully on such facades.

58 Winkler, E.M. (1991) Weathering of crystalline marble at the Field Museum of Natural History, Chicago, *APT Bulletin, The Journal of Preservation Technology*, Vol. 23, No. 4, pp. 43-7.

The Field Museum of Natural History in Chicago was closely studied for weathering effects, specifically, surface attack by dissolution and cracking of the column ribs. The building was built seventy years ago and is showing signs of deterioration on the surfaces exposed to rain. Loss of marble surface was measured and it was found that the maximum erosion on the ribs of the columns was 3,6 mm on the north side and 4,2 mm on the south-west side. It was found that the faces sheltered from rain, although accessible to dry deposition, had suffered very little deterioration. Photogrammetric techniques were used to measure the difference between the weathered and unweathered stone. Stress analysis of the columns showed that the microcracks observed on the columns result from the maximum principal stresses caused by the load of the building carried along the axis of the columns.

59 Woodward, R.J. and Williams, F.W. (1988) Collapse of Ynys-y-Gwas Bridge, West Glamorgan, in *Proceedings, Institution of Civil Engineers*, Part 1, Vol. 84, Paper No. 9357, August, pp. 685-99.

The collapse of the Ynys-y-Gwas Bridge represents the first collapse of a prestressed concrete highway bridge in the U.K. The bridge had a simply supported segmental post-tensioned deck with a clear span of 18,3 m. The nine internal beams of the deck consisted of eight precast I-sections stressed together both longitudinally and transversely. The cross sections consist of a multiple cell box section with longitudinal joints along the length of the bridge and transverse joint at each segment. The bridge was constructed in 1953 and collapsed in 1985. Although several inspections were performed on the bridge prior to collapse, no sign of trouble was detected. The cause of the bridge collapse was identified as corrosion of the post-tensioning strands due to: lack of an situ slab over the beams, ineffective water proofing, inadequate protection to the tendons resulting from the presence of several joints and improper grouting, opening of the joints between the segments under live load, poor workmanship and the damp environment over the river.

60 Wyman, M.R., Gumina, T.V. and Reins, J.D. (1990) Repair of an improperly detailed masonry facade: a case history, in *Proceedings of the 1st Materials Engineering Congress on Serviceability and Durability of Construction Materials, Denver, CO.*, 13-15 August, Vol. 1, pp. 500-7.

Extensive cracking of a brick facade of a 18 story building took place because of the lack of horizontal joints in the facade. The crushing strength was exceeded due to the weight of the brick and additional stresses induced by restrained expansion. The problem was solved by providing horizontal joints by saw-cutting the brick facade and providing additional shelf angles.

61 Ye, Y. (1989) Analysis of structural failure events in China, in *Quality for Building Users Troughout (sic.) the World, CIB 89, XIth International Congress, Paris*, 19-23 June, Theme II: Lifespan of Buildings, Vol. 1, pp. 197-206.

The author discusses the cause of failures of 285 structures in China. Over 83 % of the failures which resulted in total collapse occurred during construction. The causes of collapse have been identified as foundation failures resulting from errors in design, poor construction quality, unstable conditions existing during construction, poor construction practice such as lack of temporary bracing and early removal of formwork, overload after construction (although this could reflect a deficiency in the codes since the floor of an assembly hall should not collapse under the weight of people), and corrosion failure. Once case of roof collapse due to the corrosion failure of the suspension cables was reported. Although the author discusses only a few cases in particular, it gives the impression that poor design and quality control are at the source of the failures.

E Environment

1 Anon (1993) Removing chloride ions from reinforced concrete, *Concrete Repair Digest*, February/March, pp. 11-4.

This article is based on an earlier paper by D.G. Manning and F. Pianca of the Ontario Ministry of Transportation. It reports the results of an investigation by the OMT on a technique to remove chloride ions from concrete. The technique consists of applying an anode to the surface of a reinforced concrete structure and passing current between the anode and reinforcing steel, which acts as the cathode. Since chloride ions are negatively charged, they are pulled toward the positively charged anode. The anode is temporary and the current density applied is about 100 times that used in most cathodic protection systems. The system was maintained in place for 8 weeks and the cellulose fibres used to cover the anode is kept wet during the treatment. The results of a test on a bridge pier have shown that the chloride concentration is effectively reduced to a depth equal to the concrete cover. Corrosion activity was also found to have been reduced significantly to a passive level. The effectiveness of the method however is uncertain when chlorides penetrate deeper than the concrete cover. It is possible that the chlorides in the concrete beyond the steel reinforcement will not be removed since they may not be able to move past the negatively charged cathode. The cost effectiveness of the technique still needs to be assessed.

2 Amoroso, G.G. and Fassina, V. (1983) *Stone Decay and Conservation – Atmospheric Pollution, Cleaning, Consolidation and Protection*, (refer to section B).

3 Arya, C. and Newman, J.B. (1990) Problem of predicting risk of corrosion of steel in chloride contaminated concrete, in *Proceedings of the Institution of Civil Engineers*, Part 1, Vol. 88, October, pp. 875-88.

The authors examine three different approaches to measure the chloride content in concrete. The three techniques measure different levels of chloride: the total chloride content (acid-soluble), the water-soluble chloride content, and the chloride/hydroxyl ion concentration ratio in the pore solution. It is shown that, although the total chloride content can be determined readily, it may be a poor indicator of corrosion risk since no allowance is made for mix composition or source of chloride contamination. The water-soluble chloride content may be an inappropriate parameter to relate to corrosion risk since it provides only an approximation to the free chloride content over a limited range of mixes. The free chloride/hydroxyl ion ratio is a more realistic indicator of corrosion risk, but there are difficulties associated with its determination in actual structures. An alternative procedure is proposed whereby the free chloride content is measured by way of total chloride, using empirical relationships derived from results obtained by laboratory extraction of free chloride.

4 Asakura, S., Moriyama, M. and Matsumoto, M. (1991) A study on the distribution of salt concentration in the atmosphere at coastal areas, *Energy and Buildings*, Vol. 16, Nos. 3-4, pp. 1069-77.

The authors present the result of measurements of salt concentration in the atmosphere as a function of distance from the coastline and height. The field measurements showed that the chlorine concentration reduced with increasing distance from the coastline. The variation with height showed that a maximum concentration exists at some height above the ground surface (about 12 m) The lower concentration near the ground is believed to be caused by the absorption of the chlorine by the soil. The results of numerical calculations of salt concentration are also presented. The calculated vertical concentrations were found to be in good agreement with the measured values. However, the calculated horizontal distribution of the concentration was found to be lower than the measured distribution.

5 Ashton, H.E. (1970) *Irradiation Effects on Organic Materials*, (refer to section B).

6 Ashton, H.E. (1970) *Radiation and Other Weather Factors*, Canadian Building Digest 122, National Research Council of Canada, Ottawa, February.

The digest discusses how radiation affects organic building materials when its action is associated with the action of water and oxygen. Because the severity of the attack is increased with exposure to more than one agent, preventive action must be taken to reduce de gradation. The author discusses in particular the use of polymers unaffected by UV. The polymers may be formed with molecules which possess chemical bonds resistant to UV attack such as silicone polymers which have a silicone oxide backbone with organic side groups. Incorporation of compounds that reflect or absorb UV is another alternative. The latter, called UV absorbers, must be able to reflect the visible light (for clear polymers) and absorb UV light. Carbon black is effective at absorbing UV light but its colour is not always desirable. UV absorbers must also be able to resist degradation from UV light by dissipating the energy harmlessly as heat. The digest presents a good introduction to deterioration of polymers.

7 Ashton, H.E. and Sereda, P.J. (1982) Environment, microenvironment and durability of building materials, *Durability of Building Materials*, Vol. 1, No. 1, pp. 49-65.

The authors attempt to demonstrate that the micro environment defines the precise conditions of solid materials and the immediate layers of liquid or gas prevailing at the site where chemical or physical processes of deterioration are taking place. It emphasises the importance of measuring the conditions of the micro environment as a guide to the direction of future studies and investigations. The authors show that short-term movement of building components is predictable from surface temperature data but not from ambient temperature data. Surface moisture, defined as the time-of-wetness, can be monitored to provide the reaction time for corrosion and surface deterioration processes. The paper identifies the need to monitor the total deposition of pollutants on the surface of materials in service.

8 Atteraas, L. and Haagenrud, S. (1982) Atmospheric corrosion in Norway, (refer to section B).

9 Baker, A.J. (1980) Corrosion of metal in wood products, (refer to section B).

10 Barneyback, R.S. Jr. and Diamond, S. (1981) Expression and analysis of pore fluids from hardened cement pastes and mortars, *Cement and Concrete Research*, Vol. 11, No. 2, pp. 279-85.

The authors describe an apparatus that has been used for expression of pore solution from hardened portland cement pastes and mortars. Particulars with respect to the design, fabrication, and operation of such equipment are given,. Methods for the analysis of the resulting small volumes of pore solutions are discussed. Although questionable, it is believed that the composition of the pore solutions obtained are representative of that of the bulk of the pore solution within the paste or mortar from which the solutions have been obtained.

11 Barton, K., Knotkova, D., Strekalov, P., Kemhadze, V., Kozhukharov, V., Sobor, A., Zaydel, M. and Bestek, T. (1980) Atmospheric corrosion of metallic systems II. Analysis of the corrosion aggressiveness of the media at the atmospheric testing stations of member-nations of COMECON, according to the results of five-year tests on steel, zinc, copper, and aluminium, (refer to section B).

12 Benarie, M. and Lipfert, F.L. (1986) A general corrosion function in terms of atmospheric pollutant concentrations and rain pH, *Atmospheric Environment*, Vol. 20, No. 10, pp. 1947-58.

The atmospheric corrosion of metals has been described by a linear bi-logarithmic function of the form $M = at^b$ where a and b are constants determined by analysis of test results. The constant a was expressed in terms of the time of wetness and the SO_2 and Cl^- deposition on the surface. The exponent b, which was found to characterise the diffusivity of the corrosion product layer was expressed in terms of the rain pH. Although the simple exponential model worked well with the results obtained by various investigators, it is, however, not a complete solution because it does not take into account all the complexities of the corrosion process. The value of b in the model was found to be closely correlated to the rain pH for the results obtained on zinc, but the correlation was less clear for the steel samples. It is believed that the differences between the steel samples used by various investigators is at the origin of the lack of correlation observed for steel.

13 Berke, N.S. (1991) Corrosion inhibitors in concrete, (refer to section B).

14 Bomberg, M.T. (ed.) (1991) Moisture research in North America, in *Symposium on Research and Development in Building Physics During the Last 25 Years*, Department of Building Physics, Lund University, Sweden, 13 September, pp. 57-93.

Bomberg, along with several other authors from various research organisations in Canada and the United States, have outline some of the research being conducted in the area of moisture in buildings. Research has covered materials, components and systems. Computer models and full scale tests have been used to develop standards for test methods, design tools and guidelines for rehabilitation. Each contributor of the paper gives a brief summary of the research performed at their facilities and gives a short list of publications giving more details about the research being done. This is an excellent paper which gives a general view of the state of research in Canada and the U.S. on moisture in buildings. Some of the research work outlined consisted of monitoring the moisture, temperature, and pressure differential conditions inside real building walls.

15 Brown, P.W. and Masters, L.W. (1982) Factors affecting the corrosion of metals in the atmosphere, (refer to section B).

16 Building Research Establishment (1985) *Corrosion of Metals by Wood*, (refer to section B).

17 Carlson, A.R. (1991) Computer simulation of wall condensation problems, in *Water in Exterior Building Walls: Problems and Solutions*, (ed. T.A. Schwartz), ASTM STP 1107, American Society for Testing and Materials, pp. 210-28.

The author presents cases of field investigation of moisture condensation problems in exterior walls. A computer analysis on spreadsheet is presented whereby moisture condensation problems inside walls can be predicted from measured relative humidity and temperature on either sides of the wall and thermal and vapour penetration characteristics of the building materials. The importance of a good vapour barrier is demonstrated. It appears that a 150 µm polyethylene is one of the better vapour retarders. The importance of proper ventilation and moisture control inside air-tight houses is also emphasised.

18 Carruthers, J.F.S. (1982) Classifying the performance of components by usage, in *Proceedings of the 3rd ASTM/CIB/RILEM Symposium on the Performance Concept in Building, Lisbon*, 29 March - 2 April, pp. 161-72.

The author describes a classification system adopted in the U.K. for windows, doors, and walls. The classification is based on exposure to human activity and environment. The exposure to human activity accounts for the degree of abuse to which a building component can be subjected. Five categories have been proposed varying from public areas where little incentive to exercise care is present, to locations that cannot be reached by people (e.g. walls above 3 m). The classification categories for exposure to environment are based on wind pressure for outdoor exposure and aggressiveness of the environment for indoor component. The classification only is discuss. The purpose for the classification is to help in assessing the level of performance to expect or require for windows, doors, and walls.

19 Carter, J.P., Linstrom, P.J., Flinn, D.R. and Cramer, S.D. (1987) The effects of sheltering and orientation on the atmospheric corrosion of structural metals, (refer to section B).

20 Chawla, S.K. and Payer, J.H. (1990) Atmospheric corrosion: comparison of indoor vs. outdoor, in *11th International Corrosion Congress on Innovation and Technology Transfer for Corrosion Control, Florence, Italy*, 2-6 April, Vol. 2, pp. 2.17-24.
The authors discuss the differences between indoor and outdoor environments with respect to atmospheric water, atmospheric ionic sources (CO_2, SO_2, HCl, NO_x), atmospheric catalysts (Fe and Mn) and solar radiation. They provide a table with normal ranges of the various factors both in indoor and outdoor environments. The values presented were obtained from various cited references. The effect of the environments (both indoor and outdoor) on the corrosion rate of copper are then discussed. The paper is a good source of information for the nature of the atmospheric factors influencing the corrosion of metals.

21 Cole, I.S. (1993) Wall cavity microclimate and material durability parameters – an Australian survey, in *Proceedings of the 6th International Conference on Durability of Building Materials and Components, Omiya, Japan, 1993*, (eds. S. Nagataki, T. Nireki and F. Tomosawa), E & FN Spon, U.K., 26-29 October, Vol. 1, pp. 627-36.
The results of micro-climate monitoring in wall cavities of three houses in Australia are presented. The houses were located across three Australian climatic zones, namely, tropical, sub-tropical and temperate. The houses were brick veneer with tile roofs, typical of Australia. Temperature and relative humidity were monitored in a sheltered and in an exposed wall cavity. An analysis of the calculated absolute humidity in the cavities exposed to the rain indicated that there was possibility for rain penetration. A study of the time-of-wetness (using ISO definition of time during which RH>80 % while above freezing) was performed. Comparison of the time-of-wetness inside the wall cavities with outdoor conditions indicated that there is little correlation between the two. The TOW within the cavities fluctuated less but the conditions were found to be potentially more severe than outdoor and potential corrosion problems were observed. An equilibrium moisture content (EMC) was estimated from measured RH and temperature. Wood decay and deterioration of adhesives were the factors being investigated. It was found that the EMC was above 20% long enough to cause potential problems of wood decay. Guidelines from UK and US were used to estimate whether problems existed w.r.t. adhesives deterioration. It was found that potential problems existed according to U.S. codes but not according to U.K. code which is somewhat less stringent than the U.S. code. No description of the instrumentation and their location is given.

22 Cole, I.S. (1992) Corrosion in wall cavities – occurrence, conditions and prevention, in *2nd National Masonry Seminar, Melbourne, Australia*, November.
The problem of corrosion of metallic components in wall cavities is believed to be a widespread phenomenon. Evidence of the serious nature of the problem in Australia was discovered after the Newcastle earthquake which, although small, caused a lot of damage. Part of the problem was found to be corrosion of the wall ties. The author reports the results of a monitoring programme (this is also reported in Cole (1993) above) which indicated that the moisture conditions inside the cavity of exposed walls were favourable to corrosion activity during a significant portion of the time. A short discussion of potential problems with embedded components is given. It is not known whether wind driven rain can cause increased corrosion of embedded components in mortar joints. The author concludes with a brief discussion of corrosion prevention by proper drainage of cavities, reduction of moisture ingress and use of corrosion resistant materials (stainless steel, brass, galvalum, or increased zinc thickness).

23 Cole, I.S. (1992) Newcastle earthquake and cavity corrosion: implications for the rest of Australia, in *14th Annual Conference of the Australian Building Surveyors Queensland Chapter, October, Gold Coast Australia*.
This paper presents the same material as the above two papers by the same author. In the discussion the author states that the rate of corrosion of copper and steel exposed to the open atmosphere in Australia is comparatively the same as that measured in the U.K. It is therefore believed that the same problems of corrosion in the wall cavities can be expected in both countries. Also comparison of conditions in Newcastle with those elsewhere in Australia indicate that corrosion problems which lead to unsafe masonry structures can exist throughout Australia.

24 Comité Euro-International du Béton (1989) *Durable Concrete Structures – CEB Design Guide*, CEB Bulletin d'Information No. 182, Second Edition.

The Comité Euro-International du Béton provides a design guideline for durable concrete. The guide provides a theoretical background to concrete deterioration (corrosion of steel in concrete, causes of concrete cracking, and transport mechanisms in concrete). Processes of deterioration of concrete (physical (in the form of cracking, frost and de-icing agents, erosion), chemical (acid, sulphate, and alkali attacks) and biological) are discussed. The mechanisms of protection and deterioration of steel in concrete and the effect of corrosion are also discussed. Recommendations are given for the classification of the environments, design, construction and maintenance of concrete structures, and protection measures for the concrete and the steel reinforcement.

25 Costa, J.M. and Vilarrasa, M. (1987) Corrosion mapping for Catalonia, Spain, in *Proceedings of the 10th International Congress on Metallic Corrosion*, Vol. 1, pp. 35-44.

Results of three-year tests of atmospheric corrosion of carbon steel, copper, aluminium, zinc, brass and stainless are reported. The test specimens were standard plate specimens. The tests were conducted at 42 exposure sites across Catalonia. Weight loss and atmospheric factors such as temperature, relative humidity, SO_2 and Cl^- concentration in the air were measured. The main purpose of the test programme was to develop corrosivity maps for the various metals tested. The test results indicated corrosion rates several times larger at coastal sites than at the inland sites. The results were presented on a map (corrosion rate, SO_2, Cl^-, temperature, and relative humidity). The authors report that statistical analysis on the test results indicated that the atmospheric parameters were not independent. It was also reported that the relative effects of the various pollutants on the corrosion rate was material dependent (i.e. the controlling parameter is not the same for different metal).

26 Cramer, S., Carter, J.P., Linstrom, P.J. and Flinn, D.R. (1988) Environmental effects in the atmospheric corrosion of zinc, (refer to section B).

27 Doyle, D.P. and Wright, T.E. (1982) Rapid methods for determining atmospheric corrosivity and corrosion resistance, in *Atmospheric Corrosion*, (ed. W.H. Ailor), John Wiley and Sons, New York, pp. 227-43.

The authors deal with the CLIMAT (Classification of Industrial and Marine Atmospheres) test, which is used, as the name implies, to classify atmospheric corrosivity for a given location. Experience is mainly with aluminium and its alloys but some work with other metals has also been done. The CLIMAT test is also called the wire-on-bolt test whereby the corrosion characteristics of a wire can be investigated either alone or in a galvanic couple. The authors also describe the SIMAT (Simulation of Industrial and Marine Atmospheres) test, which has been developed in order to obtain accelerated test data on various aluminium alloys, relating to atmospheric applications.

28 Dugan, C.J. and Dolske, D.A. (1991) A twelve-year study of precipitation chemistry in Philadelphia, *APT Bulletin, The Journal of Preservation Technology*, Vol. 23, No. 4, pp. 33-6.

A record of precipitation chemistry has been collected in a mixed residential and light industrial area in north central Philadelphia over a period extending from 1978 to 1991. The rain sampling method is described. Rain pH has been monitored closely. The results show that the rain pH has gradually decreased from the early to mid 1980's but have since increased. Results of precipitation chemistry versus wind direction are also presented. The constituents investigated were potassium ions, sulphates, nitrates, and chlorides. The potassium level was found to be associated with possibly industrial sources while sulphate and nitrate levels were relatively independent of wind direction. Chlorides were found to be associated with winds coming from the Atlantic coast.

29 Duncan, J.R. and Ballance, J.A. (1988) Marine salts contribution to atmospheric corrosion, in *Degradation of Metals in the Atmosphere*, (eds. S.W. Dean and T.S. Lee), ASTM STP 965, American Society of Testing and Materials, pp. 316-26.

The amount of chloride deposition has been found by several researcher to reduce considerably within the first kilometre of the shoreline. Most of the research in that area has mainly focused on that short distance from the shore. The authors present the result of a study performed to investigate the chloride deposition rate at distances up to 90 km from the shore. Corrosion tests and measurements of the levels of chloride deposited by wind onto sheltered surfaces were performed at 18 sites in two regions of New Zealand. CLIMAT test specimens were used along with small plate specimens to evaluate the rate of corrosion. Dry-deposited chloride was collected using filter paper sheltered from rain. A good correlation was found to exist between the steel corrosion rate, measured from steel coupons and the CLIMAT test, and the logarithm of the deposited chloride. The deposition rate of chloride was found to vary considerably from month to month, indicating that the corrosivity of a site should be assessed based on several months of survey. Regression equations are proposed for the corrosion rate determined from coupons with the chloride deposition rate, for the corrosion rate of coupons in terms of the corrosion rate from the CLIMAT test, and the corrosion rate from the CLIMAT test and the chloride deposition rate. The cube root of the corrosion rate of mild steel was found to be well correlated with the distance from the shore up to 5 km. The paper does not discuss how well it correlates at greater distances.

30 Duncan, J.R. and Spedding, D.J. (1973) The effect of relative humidity on adsorption of sulphur dioxide onto metal surfaces, *Corrosion Science*, Vol. 13, pp. 993-1001.

The authors present the results of a series of experiments on metal samples to evaluate the effect of relative humidity on the adsorption rate of SO_2. When their results were compared with the test results reported by others, it was found that sample preparation before exposure has a considerable effect on the results achieved. The authors have supplemented the results of previous studies of the effect of relative humidity on the uptake of SO_2 onto metals by using a continuous flow method to examine uptake rates on iron and zinc. It was found that there exists a "critical humidity" above which appreciable corrosion occurs. This observation was also confirmed by many other researchers. A measurable desorption of SO_2 from a zinc surface was found after the removal from an atmosphere containing SO_2. There was an irregular desorption from an iron surface under these conditions.

31 Dutra, A.C. and de O. Vianna, R. (1982) Atmospheric corrosion testing in Brazil, in *Atmospheric Corrosion*, (ed. W.H. Ailor), John Wiley and Sons, New York, pp. 755-74.

The authors report that field testing for the evaluation of atmospheric corrosion of materials in Brazil started in 1972. The first field tests were set by Companhie Siderúrgica Nacional (CSN) in co-operation with the U.S. Steel Corporation. Since then different organisations initiated test programmes in various parts of the country. The authors describe the various test programmes under way and outline some of the test results. The influence of the exposure angle, chloride content in the atmosphere and distance from the sea is discussed in light of the test results presented. A total of six major test programmes are described.

32 Fassina, V. (1988) Environmental pollution in relation to stone decay, (refer to section B).

33 Flori, J.-P. (1992) *Influence des conditions climatiques sur le mouillage et le séchage d'une façade verticale*,. Cahiers du CSTB, cahier 2606, Centre Scientifique du Bâtiment, Paris CEDEX.

The CSTB carried out an experimental study on a wall exposed to actual weather conditions in order to gain a better understanding of the fluctuations in the water content of construction materials as a function of weather conditions. The testing apparatus employed provided measurements of the weight of water in the wall element, driving rain, temperature and humidity of the air next to the wall, as well as the surface temperature of the material. A reference weather station measured the environmental climate: precipitation, sunshine, temperature, humidity, and wind. A relationship was established between the water quantities absorbed by the exterior wall and the simultaneous amounts of precipitation. The influence that solar radiation and convection conditions near the wall had on the thermal exchanges and surface masses made it possible to forecast how weather conditions affected the external wall drying behaviour.

34 Flori, J.-P. (1989) Caractérisation des conditions climatiques de mouillage et de séchage d'une façade verticale, in *Quality for Building Users Troughout (sic.) the World, CIB 89, XIth International Congress, Paris*, 19-23 June, Theme II: Lifespan of Buildings, Vol. 1, pp. 285-95.

This is a description of the experimental work presented in a latter paper by the same author (Flori, 1992).

35 Garden, G.K. (1978) Design determines durability, in *Proceedings of the 1st International Conference on Durability of Building Materials and Components, Washington DC 1978*, ASTM STP 691, American Society for Testing and Materials, pp. 31-7.

No material is, of itself, durable or non-durable; it is the interaction of elements of environment with the material that determined its durability. Water is recognised to be a major factor affecting the performance of building assemblies. The forces that could move water inward through the openings are the kinetic energy of the rain drop, capillary suction, gravity, and an air pressure difference producing infiltration. It is shown in the paper, by means of practical examples, how these forces can be controlled through simple changes in the design of assemblies. Changing the position of materials in a wall makes it possible to the wall to fulfil its total function in such a way that only the least critical sub functions are imposed on each material. The importance of including the environment into the design (by considering changes that can be made to make the environment less severe for the critical materials) is outlined. Finally, it is emphasised that with the advent of polymeric materials, the designer is now confronted with the dilemma of chemical incompatibility. The paper is a useful paper to help identify and understand the transport mechanisms that can control the micro-environment in buildings.

36 Garrecht, H., Hilsdorf, H.K. and Kropp, J. (1990) Hygroscopic salts – influence on the moisture behaviour of structural elements, in *Proceedings of the 5th International Conference on Durability of Building Materials and Components, Brighton, 1990*, (eds. J.M: Baker, P.J. Nixon, A.J. Majumdar and H. Davies), E & FN Spon, U.K., 7-9 November, pp. 313-24.

Though high moisture concentrations in building elements occur due to capillary rise of water out of ground water, wet soil, or driving rain, sections only exposed to the free atmosphere will not exhibit high moisture concentrations over a prolonged period of time. High moisture concentrations found in elements of some masonry buildings may also be caused by a contamination of the building materials with hygroscopic salts which will absorb large amounts of water. Capillary rise per se is not the immediate cause of a high moisture content, dissolved salts are transported into the building materials by this mechanism. The authors are attempting to develop a numerical model taking into account the boundary conditions and the salt contamination to consider the influence of salts on the moisture behaviour of structural elements.

37 Gotfredsen, H.H. (1983) Survey of international activities, *Durability of Concrete Structures, CEB-RILEM International Workshop, Copenhagen*, 18-20 May, pp. 369-70.

The author presents, in table form, a summary of the research activities in Europe by CEB, RILEM, FIP, IABSE, OECD, CIB, and ISO, as well as by technical universities, public research, and private research. The technical aspects of research surveyed are: material composition, environment, structural form, and interactions between the basic parameters. It seems that the area receiving less attention at the time of the survey was the environment (macro- and micro-climatic conditions). Other aspects of concrete durability have been receiving ample attention at that time.

38 Graedel, T.E. (1988) The chemistry of precipitation: perspectives on potential impacts on the corrosion of metals, in *Degradation of Metals in the Atmosphere*, (eds. S.W. Dean and T.S. Lee), ASTM STP 965, American Society for Testing and Materials, Philadelphia, pp. 327-35.

The author looks at factors which can be used to assess the potential degradation of a metal upon atmospheric exposure, namely, the frequency of precipitation, the chemical constituents of the precipitation, and the susceptibility of the metal to those constituents. Frequency distributions of rain showers duration and rain shower depth for the Brookhaven National Laboratory in New York show that the precipitation events are of greatly differing length and intensity. Summary of dew duration from the Pendleton Experimental station shows that the occurrence of dew is strongly dependent on high relative humidity, with nearly all days on which dew formed having relative humidity greater than 90 % lasting for eight hours or more. Looking at the chemical constituents of precipitation, the author presents an interesting table containing ten constituents with typical concentration in fog, dew, and rain. The change of concentration of sulphates, nitrates and hydrogen ions with time of precipitation is also presented. Data showing the change in sulphur dioxide and nitrogen dioxide concentration from 1975 to 1983 at almost 300 U.S. sites indicate that the level of SO_2 has steadily dropped while the level of NO2 has not changed much. Very little is said about metals susceptibility to various atmospheric constituents.

39 Graedel, T.E. (1986) Corrosion-related aspects of the chemistry and frequency of occurence of precipitation, in *Proceedings of the Symposia on Corrosion Effects of Acid Deposition and Corrosion of Electronic Materials*, (eds. F. Mansfeld et. al.), April, Vol. 86-6, pp. 155-72.

The author discusses some of the corrodents which can be encountered in precipitation and be detrimental to corrosion of metals. Seven different constituents including hydrogen ion (pH), nitrates, sulphates and chlorides are discussed. The various forms of precipitation include rain, snow, dew, and fog. The results of surveys in the U.S. indicating the amount of the various constituents in the various types of precipitation are presented. The paper is interesting as it gives a good idea of the normal range of pollutants that can be expected as well as the normal ranges of precipitation quantity and duration. It is interesting to note that fog has the lowest pH of all the various forms of precipitation and the pH of fog in the LA area has been recorded to be somewhere near that of lemon and battery acid.

40 Graedel, T.E. (1984) Concentrations and metal interactions of atmospheric trace gases involved in corrosion, (refer to section B).

41 Graedel, T.E. and McGill, R. (1986) Degradation of materials in the atmosphere, (refer to section B).

42 Grimm, C.T. (1985) Corrosion of steel in brick masonry, in *Masonry: Research, Application, and Problems*, (eds. Grogan and Conway), ASTM STP 871, pp. 67-87.

The author presents a discussion of the causes and protection methods for steel in masonry. Water permeance of masonry is substantially greater than that of concrete and, consequently, the thickness of cover should be greater than required for reinforced concrete. A nominal cover of 100 mm is required if black steel is used in masonry structures. Another importance of water in wall cavities is condensation of water vapour on the interior surface of the exterior brick wythe. The use of flashing to redirect the water penetrating the wall cavity back outdoors is advocated. It is recognised that water is the main factor affecting corrosion of steel in masonry. Interesting references are made to case histories where corrosion of steel in masonry occurred prematurely and some case studies from the author are also presented. Hot dip galvanised steel or copper coated wire is recommended for ties, anchors, and joint reinforcement. Shelf angles and steel lintels should be hot dip galvanised.

43 Grondin, G.Y. (1993) *Damage Functions for Service Life Prediction of Zinc and Steel Components*, (refer to section B).

44 Guttman, H. (1982) Atmospheric and weather factors in corrosion testing, in *Atmospheric Corrosion*, (ed. W.H. Ailor), John Wiley and Sons, New York, pp. 51-68.
The paper reviews techniques for measuring climatic/atmospheric factors that are known to have an effect on the corrosion of metals. It discusses corrosion rate studies carried out in conjunction with measurements of atmospheric factors. The need to direct future research effort to take into account conditions of actual service is stressed. Such conditions include orientation, degree of shelter, effects of corrosion product and/or foreign materials, and others. The importance of time-of-wetness of a corroding substrate is discussed in some detail.

45 Guttman, H. (1968) Effects of atmospheric factors on the corrosion of rolled zinc, (refer to section B).

46 Guttman, H. and Sereda, P.J. (1968) Measurement of atmospheric factors affecting the corrosion of metals, (refer to section B).

47 Haynie, F.H. (1988) Environmental factors affecting the corrosion of galvanised steel, in *Degradation of Metals in the Atmosphere*, (eds. S.W. Dean and T.S. Lee), ASTM STP 965, American Society for Testing and Materials, Philadelphia, pp. 282-9.
The author first presents a brief discussion of theoretical considerations in the corrosion of metals in the atmosphere. The importance of the corrosion film in providing protection against further corrosion is emphasised. In polluted atmospheres zinc exhibits a linear corrosion rate with time indicating that the corrosion film dissolves as rapidly as it forms. Test data obtained in a test programme described elsewhere for the St. Louis area were used to compare the measured effect of air pollutants with their theoretical effect. Sulphur dioxide deposition on zinc specimens was evaluated from measured concentration in the air and measured wind velocities. The corrosion rate of zinc was found to be a linear function of time. The calculation of SO_2 deposition on the surface accounted for dry and wet deposition using wind speed, dew point, relative humidity, and temperature parameters. It is recognised that the critical relative humidity at which corrosion takes place is a function of the sulphur dioxide deposition. The accumulation of particles on the surface of zinc is also recognised to influence the effective time-of-wetness. Ozone, oxides of nitrogen, sulphate and nitrates were found to have insignificant effect on the measured corrosion rates.

48 Haynie, F.H. (1982) Economic assessment of pollution-related corrosion damage, in *Atmospheric Corrosion*, (ed. W.H. Ailor), John Wiley and Sons, New York, pp. 3-17.
The importance of corrosion as a product life limiting factor will increase with the necessity to conserve limited resources. Thus, an economic model to assess pollution related corrosion costs becomes a useful decision tool. Physical damage, geographic distribution, maintenance, replacement, substitution, and value lost due to decreased utility or appearances are incorporated into the model. These interrelated factors are subject to individual decisions, which can be assumed in calculating best, worst, and most likely cases for different pollution levels.

49 Haynie, F.H. (1982) Evaluation of the effects of microclimate differences on corrosion, (refer to section B).

50 Haynie, F.H. (1978) Theoretical air pollution and climate effects on materials confirmed by zinc corrosion data, (refer to section B).

51 Haynie, F.H. and Upham, J.B. (1974) Correlation between corrosion behavior of steel and atmospheric pollution data, (refer to section B).

52 Hechler, J.-J. (1991) Metal corrosion, wetness and deposition at the exterior of a building in Montréal, *APT Bulletin, The Journal of Preservation Technology*, V. 23, No. 4, pp. 20-5.

The author presents the results of an investigation of the corrosion rate of steel and copper at various locations on the exterior surface of Ecole Polytechnique in Montreal. Mini-racks were used at 44 locations on the building. Each rack was instrumented with a time-of-wetness sensor, a copper and a steel plate to measure the corrosion rate, and a nitration and a sulphation plate to measure the amount of dry deposition. The results of the investigation showed that corrosion rates can vary significantly from one point to another on the building. The trend of corrosion deterioration was different for copper and steel. It seems that the extrapolation of standard ASTM tests to real structures can be very difficult since no correlation was detected between the specimens mounted on the walls of the building and a standard rack installed on the roof of the building. The author recommends that more study of this nature be conducted.

53 Hechler, J.-J., Boulanger, J., Noël, D. and Pinon, C. (1993) Corrosion rates, wetness, and pollutants on exterior of building, *Journal of Materials in Civil Engineering*, ASCE, Vol. 5, No. 1, February, pp. 53-61.

The authors present the test programme previously discussed by the first author (Hechler, 1991). More observations are made in this paper. It is interesting to note that the corrosion rate of steel was faster during the fall than during winter, while the opposite was observed for copper. The corrosion of copper varied significantly around the external surface of the building (Ecole Polytechnique, Montreal) but not in the same way as for steel. The time-of-wetness and sulphur dioxide deposition rate were found to vary significantly around the building. No correlation could be observed between the results obtained on a standard exposure rack placed on the roof of the building and the results obtained at various locations on the building. It is interesting to note, however, that the standard rack exposure yielded more severe corrosion than the exposure specimens placed at various locations on the building surface. It is therefore recommended that standard exposure rack results can be used to estimate the highest corrosion rate on a building due to the general climatic environment.

54 Hosker, R.P., Smith, E.A., White, J.R. and Heathcote, E.A. (1991) Dry deposition to structures: configuration considerations, *APT Bulletin, The Journal of Preservation Technology*, Vol. 23, No. 4, pp. 26-32.

The authors look at the dry deposition of atmospheric pollutant on structures in terms of mass transfer. An analogy to Ohm's law is drawn whereby the potential difference is the concentration difference between the surface and the atmosphere, the current is the deposition flux, and the resistance is made up of the aerodynamic transfer resistance, the boundary layer transfer resistance, and the uptake resistance. The dry deposition of pollutants is believed to take place in three stages: the pollutants from the surrounding air mix with the air in the vicinity of the surface; the pollutants cross the surface boundary layer; they are finally captured by the surface. To study the aerodynamic and boundary layer transfers the authors have conducted wind tunnel tests using a model statue and various architectural columns. Matte black models are coated with a saturated naphthalene in acetone solution to produce a uniform white coating. In the air stream the coating sublimates and the black surface is exposed where deposition would take place. Various positions and arrangements of architectural columns were investigated. The technique is quite interesting and seems to be promising. More work is required before predictions of deposition rates can be made from wind tunnel tests. Also the technique needs to be compared with field observations.

55 Hudson, J.C. and Stanners, J.F. (1953) The effect of climate and atmospheric pollution on corrosion, (refer to section B).

56 Johansson, L.-G. (1984) The corrosion of steel in atmospheres containing small amounts of SO_2 and NO_2, (refer to section B).

57 Johansson, L.-G., Lindqvist, O. and Mangio, R.E. (1988) Corrosion of calcareous stones in humid air containing SO_2 and NO_2, (refer to section B).

58 Justo, M.J. and Ferreira, M.G.S. (1993) The corrosion of zinc in simulated SO_2-containing indoor atmospheres, (refer to section B).

59 Kalin, Z. (1992) The construction demolition waste stream: magnitude and potential, in *Proceedings, CIB 92 – World Building Congress 1992, Montreal, Canada*, Tome 1, pp. 163-5.

According to a report released in 1992 by Environment Canada, nine million tonnes of solid waste are created yearly only from construction and demolition. Those yearly figures for The Netherlands are reported to be 12 million tonnes. It is estimated that, before the recession, 12 % of Ontario's work force was employed in construction and demolition. The author discusses some of the problems encountered by municipalities in Ontario with the disposal of so much waste. Although recycling is part of the solution many problems are encountered and it remains only partially satisfactory.

60 King, G.A. (1988) A corrosivity survey on a grid of sites ranging from rural to moderately severe marine; part 1. steel, galvanised steel, and zincalume coated steel, (refer to section B).

61 King, G.A., Dougherty, G.J., Dalzell, K.W. and Dawson, P.A. (1988) Assessing atmospheric corrosivity in Antarctica, (refer to section B).

62 King, G.A. and Gibbs, P. (1986) Corrosivity mapping around a point source of pollution, (refer to section B).

63 Kucera, V. (1985) Influence of acid deposition on atmospheric corrosion of metals: a review, (refer to section B).

64 Kucera, V., Haagenrud, S., Atteraas, L. and Gullman, J. (1988) Corrosion of steel and zinc in Scandinavia with respect to the classification of the corrosivity of atmospheres, (refer to section B).

65 Kudder, R.J. and Hoigard, K.R. (1991) Vapour control and psychrometric monitoring in exterior walls, in *Water in Exterior Building Walls: Problems and Solutions*, (ed. T.A. Schwartz), ASTM STP 1107, American Society for Testing and Materials, Philadelphia, pp. 124-37.

A discussion of the measurement of temperature, relative humidity, and air pressure in exterior walls is presented. The discussion covers mostly the field work organisation and instrumentation to use for such a task. The purpose of conducting the measurements discussed is to detect the occurrence of condensation inside wall. Topics such as instrumentation locations, data sampling and recording equipment, duration of the monitoring programme and the interpretation of the of the data are covered. Examples are also given of recorded data to illustrate the response of the sensors to various conditions of condensation on surfaces or other phenomena related to moisture and air penetration in walls. An interesting table is provided summarising the wall response behaviour (in terms of temperature, air response and vapour pressure) for properly designed and built walls, walls with inadequate vapour control, and walls with excessive vapour control.

66 Lambert, P. and Wood, J.G.M. (1990) Improving durability by environmental control, in *Proceedings of the 5th International Conference on Durability of Building Materials and Components, Brighton, 1990*, (eds. J.M: Baker, P.J. Nixon, A.J. Majumdar and H. Davies), E & FN Spon, U.K., 7-9 November, pp. 445-52.

The paper examines the environmental factors which influence the service life of building materials and shows how they may be controlled by the use of techniques such as coating and cladding. The paper also examines potential risks associated with incorrect use of such techniques. For example, while sheltering concrete from the elements may be beneficial in limiting existing corrosion and slowing alkali aggregate reaction, it can also accelerate the rate of carbonation of the concrete cover leading to the destruction of the protective alkaline environment.

67 Latta, J.K. (1962) *Water and Building Materials*, (refer to section B).

68 Leduc, A. (1991) Monitoring urban precipitation chemistry in the ville de Montréal, *APT Bulletin, The Journal of Preservation Technology*, Vol. 23, No. 4, pp. 10-2.

A description of a research programme to monitor the acid precipitation in Montreal is presented. The author comments that research in Canada, so far , is based on sampling stations located outside major cities. The purpose of the programme is to evaluate the chemistry of Montreal's precipitation, as one aspect of the trans-boundary pollution question. Sampling of precipitation started in March 1990. The average precipitation pH to date was 4,26 and ranges typically between pH 3,8-4,6. The sulphate concentration was found to be approximately double of the nitrate concentration, which is believed to be typical of North American precipitation. Storm systems from the south west were found to be more acidic with an average pH of 4,07.

69 Lipfert, F.W. (1987) Effects of acidic deposition on the atmospheric deterioration of materials, (refer to section B).

70 Lipfert, F.W., Benarie, M. and Daum, M.L. (1986) Metallic corrosion damage functions for use in environmental assessments, in *Proceedings of the Symposia on Corrosion Effects of Acid Deposition and Corrosion of Electronic Materials*, (eds. F. Mansfeld et. al.), April, Vol. 86-6, pp. 108-54.

The authors present the results of a statistical analysis of atmospheric corrosion test results for zinc and galvanised steel, plain carbon steel, pure copper, and pure aluminium. The data base was assembled from published data from eight test programmes covering up to 72 test sites world wide. Various statistical models were fitted to the test data and the various models were compared for the goodness of fit using the correlation coefficient and the root mean square of the residuals. Based on a comparison of the various models investigated, a best fit model was proposed and compared with previous damage functions proposed by other authors. This paper presents in a fairly detailed fashion the work of Lipfert and collaborators. It is also a good source of reference for results of field exposure tests and some of the damage functions proposed by other researchers. The list of damage functions from other researchers is very short compared to what has been published in the literature.

71 Lipfert, F.W., Horst, R., Sherwood, S. and Lareau, T. (1986) The 1985 acid precipitation materials damage assessment, (refer to section B).

72 Macchi, G., Radogna, E.F. and Materazzi, A.L. (1989) Synergetic effects of environment actions and fatigue, in *Durability of Structures, IABSE Symposium, Lisbon*, 6-8 September, Vol. 57, pp. 493-8.

The effect of corrosion of the fatigue life of steel is discussed by the authors. It is well known that corrosion has several effects on the conventional S-N curve obtained in a non-corrosive environment. It removes the endurance limit and increases the slope of the S-N curve. The authors look at the effect of the slope of the S-N curve and the presence and absence of an endurance limit on the safety index. Not enough details have been given to make any sense of the numerical examples given in the paper. Nevertheless, as one might expect, the effect of the presence of an endurance limit on the safety index is more significant than the effect of the slope of the S-N curve. When an endurance limit is used, the effect of the slope of the S-N curve is relatively small compared to its effect when an endurance limit is not used. The examples shown by the authors indicate that, as the endurance limit increases, the safety index increases. This is also expected since the chances of failure in fatigue are smaller as the endurance limit increases. When the endurance limit is not used, the effect of the slope of the S-N curve is much more significant. This is again expected since the probability of fatigue failure is controlled only by the slope of the curve rather than by the level of the endurance limit.

73 Malhotra, S.K. (1989) Factors which influence durability of wooden structures, in *Durability of Structures, IABSE Symposium, Lisbon,* 6-8 September, Vol. 57/1, pp. 193-8.
Various factors affecting the serviceability and durability of wooden structures are briefly discussed. The factors discussed in the paper are: loading (man-made and natural loads); time (duration of load and creep and fatigue of wood); temperature, moisture content and cyclic environment (wood is dimensionally stable as long as its moisture content is above the fibre saturation point); weathering, insects, fungi and other organisms, and chemicals (generally hardwoods are more susceptible to chemical degradation than are coniferous species. Alkaline solutions, which dissolve some of the hemicelluloses and attack lignin in wood, have a greater effect on wood strength than do acids. Acids make wood more brittle and reduce its strength); fire (the basic effects of fire on wood members are the reduction of cross-section and the weakening of metallic fasteners in the member).

74 Mangat, P.S. and Molloy, B.T. (1992) Factors influencing chloride-induced corrosion of reinforcement in concrete, (refer to section B).

75 Mansfeld, F. (1982) Electrochemical methods for atmospheric corrosion studies, in *Atmospheric Corrosion*, (ed. W.H. Ailor), John Wiley and Sons, New York, pp. 139-60.
Past and present efforts to use electrochemical techniques for the evaluation of atmospheric corrosion phenomena are reviewed. Techniques for the determination of the time-of-wetness and the recent controversy concerning the definition of the time-of-wetness are discussed. Various approaches for measurements of atmospheric corrosion rates and the question of the efficiency of electrochemical sensors (”cell factor”) are evaluated. Recent results obtained in the author's laboratory concerning the reproducibility of electrochemical measurements in a statistically designed experiment under atmospheric corrosion conditions (15 sensors of one type; RH = 65, 80, and 95 %; 0,1 or 1,0 ppm SO_2) are presented.

76 Martin, K.G. and Price, R.E. (1982) Quantitative considerations of moisture as a climatic factor in weathering, *Durability of Building Materials*, Vol. 1, No. 2, pp. 127-40.
The authors discuss functions that relate to moisture-associated degradation of building materials. These have involved studies where some quantitative information has been given on both exposure and effect. Observations have shown that the corrosion of metals is greatly influenced by the time-of-wetness, which, in turn, can be related to the relative humidity. When UV irradiance exceeded a relatively low level, the relative humidity was found to be the most important climatic factor for chalking of paint, although no damage function could be derived. Yellowing of polyester resins was found to be influenced by the moisture content of the ambient air rather than the RH, and also independently by the temperature. Results of tests on timber showed that the temperature dependence of the loss of strength fitted an Arrhenius-type equation. Cycles of wet and dry are also influential on the deterioration of timber. This suggests that climatic factors used to characterise different sites should be in terms of mean annual air temperatures under dry and wet conditions. Climatic data for various locations in Australia are compared with similar data for Canada coastal and inland regions. Comparison of corrosion rate and time-of-wetness (expressed as RH>85 %) showed that temperature was also an important factor that should be considered.

77 Masuda, Y. (1987) Penetration mechanism of chloride ion into concrete, in *Proceedings of the 1st International Conference on Durability of Construction Materials, RILEM, Versailles, France*, 7-11 September, pp. 935-42.
The penetration mechanism of chloride ions into concrete structures standing in a seaside area has been analysed by applying the diffusion theory. Other researchers have solved the diffusion equation assuming the chloride ion concentration at the surface of the concrete to be constant. This assumption is believed to be valid for structures in direct contact with sea water. In order to simulate conditions where the chloride ion are brought to the concrete surface by the atmosphere, the author considers the case where the chloride ions concentration at the surface of the concrete varies with time. Chloride ion concentrations were measured in actual buildings located at various distances from the seashore. A good correlation between the measured Cl^- concentrations and the predicted values was found to exist.

78 Matsumoto, M. and Fujiwara, S. (1991) A study of annual moisture variation in an internally insulated building wall under a mild climate using a small-scale model and the similarity laws, *Energy and Buildings*, Vol. 16, Nos. 3-4, pp. 933-45.

The authors present an experimental method using a small-scale model of a building wall and similarity laws based on the governing equations of simultaneous heat and moisture transfer through porous media. The scale model tests are used to predict the behaviour of the real building wall using similarity laws. The technique is validated using the results of a numerical solution of the governing equations in which the values of the physical parameters are independently measured values.

79 Maurenbrecher, A.H.P. and Suter, G.T. (1989) A loadbearing clay brick masonry deterioration problem: monitoring of temperature and moisture, (refer to section D).

80 McGee, E.S. (1991) Influence of microclimate on the deterioration of historic marble buildings, (refer to section B).

81 Mikhailovsky Y.N. and Strekalov, P.V. (1982) Atmospheric corrosion tests in the USSR, (refer to section B).

82 Moresby, J.F., Reeves, F.M. and Spedding, D.J. (1982) Atmospheric corrosion testing in Australasia, (refer to section B).

83 Nagataki, S. and Ohga, H. (1992) Combined effect of carbonation and chloride on corrosion of reinforcement in fly ash concrete, (refer to section B).

84 Norberg, P., Sjöström, C., Kucera, V. and Rendahl, B. (1993) Microenvironment measurements and materials degradation at the Royal Palace in Stockholm, in *Proceedings of the 6th International Conference on Durability of Building Materials and Components, Omiya, Japan, 1993*, (eds. S. Nagataki, T. Nireki and F. Tomosawa), E & FN Spon, U.K., 26-29 October, Vol. 2, pp. 589-97.

As part of the research project EU 640 Eurocare WetDry-Dep, the deterioration building materials under micro environmental conditions existing on existing buildings. The project described in this paper is the Swedish part of five national measurement and exposure programmes. The corrosivity and variation of the micro environment around the Royal Palace is being monitored using C-steel, zinc, copper and limestone mounted at various locations around the Palace. Some of the test racks are sheltered while others are boldly exposed. The time-of-wetness, surface temperature, UV intensity, and concentration and deposition of SO_2 and NO_2 are being monitored. A standard exposure rack was place on the roof of the Palace for comparison with the samples mounted on the walls. At the time the paper was prepared the results of only six months of exposure were available. The sheltered specimens have shown lower deterioration than the boldly exposed specimens. The corrosion rates observed on the facade were considerably lower than those observed on the standard rack on the roof. SO_2 deposition was found to vary a lot around the building and also with time with significantly more deposition recorded during the winter. A strong correlation between deposition rates and concentrations was observed.

85 Oelsner, G. (1982) Atmospheric corrosion testing in the Federal Republic of Germany, (refer to section B).

86 Oldfield, J.W. and Todd, B. (1990) Ambient-temperature stress corrosion cracking of austenitic stainless steel in swimming pools, (refer to section B).

87 Ottar, B. (1985) Acidification of precipitation, in *Materials Degradation Caused by Acid Rain, 20th State-of-the-Art Symposium of the American Chemical Society*, (ed. R. Baboian), ACS Symposium Series 318, pp. 2-22.

Concerns about acidification of precipitation in Europe started to appear in the late 60's. Several international projects have been completed with 26 countries participating and Canada and the U.S.A. being observers. It has been found that the acidification of precipitation in central Europe has increased over the years. The main sources of acidification are sulphuric and nitric acids. Those acids form in the atmosphere by oxidation of SO_2 and NO_x through a photochemical process. The main sources of SO_2 and NO_x are the use of fossil fuels, with 50 % of the emissions coming from vehicular traffic. The distribution of SO_2 in North America presented by the author shows the eastern part of the North American continent as the main emission area. A map of North America with pH isopleths is presented and shows a similar distribution of acidity as that presented for the SO_2 distribution. In arid areas where pollution levels are significantly high, the SO_2 and NO_x emission stay in the atmosphere for longer periods of time. Because of the photochemical process the precipitation will be increasingly acid the longer the pollutants are allowed to stay in the atmosphere. As a consequence, precipitation with a pH of 2,5 have been reported in Iceland in the spring when the periods of sunshine are long and the amount of precipitation small. The author discusses the changes taking place in fresh water bodies and soils and the effects on vegetation and aquatic life resulting from acidification of precipitation.

88 Pfeifer, D.W., Perenchio, W.F. and Hime, W.G. (1992) A critique of the ACI 318 chloride limits, *PCI Journal*, Vol. 37, No. 2, March-April, pp. 68-71.

The paper traces the changes in the requirements for chloride ions and suggests new limits based on the results of recent studies. Current ACI specifications suggest maximum chloride ion content for corrosion protection in terms of water soluble chloride. However, corrosion specialists and specifying agencies have not yet agreed on a proper method to determine water-soluble chloride. The paper suggests, based on published test results, that the current recommended maximum chloride content for reinforced concrete that will be dry or protected from moisture in service should be decreased from 1,0 % water soluble to 0,2 % acid soluble. Corrosion test data show that prestressing steel has a much greater resistance to corrosion (the time to corrosion and the chloride ion corrosion threshold are both greater) than for deformed reinforcing steel. No difference in corrosion activity is found to exist between stressed and unstressed strand. The authors suggests that the current water soluble chloride ion content for prestressed concrete is too conservative at 0,06 % by weight of cement and they suggest the use of an acid soluble chloride ion content of 0,10 % by weight of cement.

89 Reddy, M.M. and Youngdahl, A. (1987) Acid rain and weathering damage to carbonate building stone, (refer to section B).

90 Roper, H. and Baweja, D. (1991) Carbonation-Chloride Interactions and Their Influence on Corrosion Rates of Steel in Concrete, (refer to section B).

91 Saricimen, H., Al-Tayyib, A.J., Maslehuddin, M. and Shamim, M. (1991) Concrete deterioration in high chloride and sulphate environment and repair strategies, (refer to section B).

92 Semonin, R.G. (1986) Wet deposition chemistry, in *Materials Degradation Caused by Acid Rain, 20th State-of-the-Art Symposium of the American Chemical Society*, (ed. R. Baboian), ACS Symposium Series 318, pp. 23-41.

The various chemical elements encountered in rain and which are potentially damaging to materials are discussed. Those elements are calcium, ammonium, chloride, nitrate, and sulphate. The pH of the precipitation is also discussed. Concentration of those elements in rain have been measured on a U.S. network. The results of those measurements have been mapped and are presented in the paper. The dry deposition, although quite significant, has not been presented in this paper. The author comments that dry deposits must be measured before the biochemical and geochemical cycles can be fully understood. The maps presented in the report indicate that the greatest deposition is directly associated with the largest emission regions. The data collected showed that there is not a rapidly declining quality of precipitation.

93 Sereda, P.J. (1986) *Assessment of the State of Knowledge on the Long-Range Transport of Air Pollutants and Acid Deposition – Part 6: Effects on Man-Made Structures*, (refer to section B).

94 Sereda, P.J. (1974) Weather factors affecting corrosion of Metals, in *Corrosion in Natural Environments*, ASTM STP 558, American Society for Testing and Materials, pp. 7-22.

The author discusses the implication of the definition of the time-of-wetness and its importance on the prediction of the corrosion of metals in the atmosphere. Although there is still doubt regarding the level of humidity that should be taken in determining percentage time-of-wetness and whether it is different for each metal, it is clear that the corrosion process is definitely related to it and that prediction of relative corrosivity at a given site can be improved if time-of-wetness can be predicted. The author presents, in the form of a literature review the effect of time-of-wetness, level of sulphur dioxide, chlorides, corrosion products and temperature on atmospheric corrosion.

95 Sereda, P.J., Croll, S.G. and Slade, H.F. (1982) Measurement of the time-of-wetness by moisture sensors and their calibration, in *Atmospheric Corrosion of Metals*, (eds. S.W. Dean, Jr., and E.C. Rhea), ASTM STP 767, American Society for Testing and Materials, pp. 267-85.

A one-year programme involving several laboratories located in different climatic zones has afforded an opportunity to evaluate the response of miniature moisture sensors to surface moisture on panels exposed to the atmosphere. The authors present the results of a round-robin evaluation of the moisture sensor developed at NRCC. It was shown that when these moisture sensors are placed on the surface of metal or plastic panels they respond to moisture conditions at the sensor surface and that such moisture conditions result from interaction of the total environment with the material as well as with the ambient relative humidity conditions.

96 Sereda, P.J. and Hechler, J.-J. (1989) *Assessment of the State of Knowledge on the Long Range Transport or Air Pollutants and Acid Deposition Update 1989 – Part 7 : Effects on Man-Made Structures*, (refer to section B).

97 Sherwood, S.I. and Dolske, D.A. (1991) Acidic deposition and marble monuments at Gettysburg National Military Park, (refer to section B).

98 Skerry, B.S., Johnson, J.B. and Wood, G.C. (1988) Corrosion in smoke, hydrocarbon and SO_2 polluted atmospheres – I. General behaviour of iron, (refer to section B).

99 Skerry, B.S., Wood, J.C., Johnson, J.B. and Wood, G.C. (1988) Corrosion in smoke, hydrocarbon and SO_2 polluted atmospheres – II. Mechanistic implications for iron from surface analytical and allied techniques, (refer to section B).

100 Skerry, B.S., Johnson, J.B. and Wood, G.C. (1988) Corrosion in smoke, hydrocarbon and SO_2 polluted atmospheres – III. The general behaviour of zinc, (refer to section B).

101 Southwell, C.R. and Bultman, J.D. (1982) Atmospheric corrosion testing in the Tropics, (refer to section B).

102 Sussex, G.A.M., Scantlebury, J.D. and Johnson, J.B. (1984) Atmospheric corrosion and its accelerated testing, (refer to section B).

103 Tomiita, T. (1992) Solar UV, wetness and thermal degradation maps of Japan, *Construction & Building Materials*, Vol. 6, No. 4, pp. 195-200.

The author presents contour maps of deterioration factors obtained from meteorological data collected at approximately 150 locations in Japan. The yearly amount of solar UV energy (measured for part of experimentation with a photodiode) was calculated from the solar altitude and the hourly range of solar energy measured with a pyranometer. The values obtained can serve as a guide for the severity of the climatic conditions on polymeric building materials. The wetness time was estimated as the time during which the relative humidity was greater than 80 % while the temperature was above freezing (it may also have been determined from dew point calculations: the paper is not clear in this respect). The number of wet-dry cycles per year was also obtained. The black panel temperature, which is an index of the severest thermal degradation conditions, was measured at 66 locations in Japan and the results were used to calculate a daily equivalent black panel temperature. The maximum daily temperature range was calculated using minimum and maximum measured temperatures corrected for the effect of wind and cloud cover.

104 Weaver, M.E. (1991) Acid rain and air pollution vs. the buildings and outdoor sculptures of Montréal, *APT Bulletin, The Journal of Preservation Technology*, Vol. 23, No. 4, pp. 13-9.

A brief description of the deterioration process of various materials (metals, stones and masonry, glass) exposed to acid rain is given. Various deterioration problems are outlined and concrete examples of such occurrences in Montreal are given. It is stressed that deterioration caused by acidic precipitation and air pollution is the result of extremely complex interactions involving chemical, physical and physico-chemical processes. The time-of-wetness is said to be the most important factor in the atmospheric corrosion of metals. It is also an important factor with other materials. Rough or highly textured surfaces have relatively large surface areas in comparison with smooth or polished surfaces, are wetted more easily and remain wet longer. Examples of pollution induced deterioration in Montreal are given for stone, metals, and stained glass. Remedial and protective measures against the effects of pollution are proposed. Finally, the author states that the assessment of risks of pollutant-accelerated deterioration need to be factored into cost-benefit ratios. The initial investment in stainless steel flashings and roofs is offset by avoiding the risk of interior damage from leaks and the cost of maintenance.

105 Williams, M.F. and Williams, B.L. (1991) Water intrusion in barrier and cavity /rain screen walls, in *Water in Exterior Building Walls: Problems and Solutions*, (ed. T.A. Schwartz), ASTM STP 1107, American Society for Testing and Materials, pp. 1-10.

The authors discuss two exterior wall concepts - barrier and cavity/rain screen - as they relate to water penetration. Typically, a barrier wall is designed to prevent water intrusion. On the other hand the cavity and rain screen walls allow water penetration into the wall assembly (to the air cavity) but are designed to manage the water infiltration by redirecting it to the outside. The rain screen relies on pressure equalisation between the outdoor and the air cavity to minimise water infiltration. Therefore, the cavity in a rain screen is usually smaller than that in a cavity wall in order to allow for fast equalisation. The authors discuss various deficiencies of two wall systems encountered when workmanship is not adequate.

106 Yamasaki, R.S., Slade, H.F. and Sereda, P.J. (1983) Determination of time-of-wetness due to condensed moisture, *Durability of Building Materials*, Vol. 1, No. 4, pp. 353-61.

The authors present the results of an experimental evaluation of the time-of-wetness on a metal surface caused by condensation of moisture as opposed to wetting from precipitation. Time-of-wetness resulting from condensation of moisture on an exposed metal surface is believed to be more detrimental than that resulting from precipitation. This is due to the fact that SO_2 will be present in higher concentration in moisture than in rain. Furthermore, rain washes the surface and makes the environment cleaner than for moisture condensation. In order to separate the two sources of wetting of the surface of a metal, the authors used two stainless steel plates, one slightly heated and the other at ambient temperature of the exposure site. The heated plate was used to measure the time-of-wetness due to rain precipitation while the other plate measured the time-of-wetness resulting from both precipitation and moisture condensation. Measurements over a period of one year showed that the time-of-wetness resulting from moisture condensation alone is about twice as long as the time-of-wetness resulting from precipitation alone. The time-of-wetness was measured using a surface moisture sensor developed by the third author.

107 Zak, T. and Chojnacka-Kalinowska, G. (1982) Evaluation of corrosivity of various atmospheres, in *Atmospheric Corrosion*, (ed. W.H. Ailor), John Wiley and Sons, New York, pp. 217-26.

The atmospheric corrosion of low-carbon steel in various climatic conditions (at 14 weather stations located in different regions of Poland) was examined. Corrosion losses were determined at monthly intervals for two years. The results obtained were correlated with meteorological parameters and SO_2 concentrations. Suitability of one and multi-parameter equations was statistically determined. The best results were obtained when correlating the corrosion rate (K) and the time when the specimen is wet. The wet time was calculated from the difference of air and dew point temperatures. The rate of corrosion was found to be best predicted using the equation $K = at_B + b$ where a and b are the regression coefficients and t_B is the time-of-wetness. This equation was found most suitable for predicting the corrosion losses in Poland.