Durability Design of Concrete Structures

EDITED BY

A. Sarja and E. Vesikari

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Report of TC 116-PCD. Edited by H.K.Hilsdorf and J.Kropp

13 Ice and Construction
Report of TC 118-IC. Edited by L.Makkonen

14 Durability Design of Concrete Structures
Report of TC 130-CSL. Edited by A.Sarja and E.Vesikari

Durability Design of Concrete Structures

Report of RILEM Technical Committee 130-CSL

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(The International Union of Testing and Research Laboratories for Materials and Structures)
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Contents

Preface

Control of the durability of concrete structures has gained increasing importance at the design and maintenance stages of these structures. Most of the current durability problems of concrete structures in, for example, bridges, dams and façades of buildings could have been avoided with systematic durability design. Such design, however, requires both an overall methodology and detailed calculation models of actual deterioration processes. Safety requirements and margins must be defined for service life. This report represents the first systematic attempt to introduce into structural design both a general theory of structural reliability and existing calculation models for the commonest degradation processes.

In fact, the introduction of systematic durability design will mean better utilization of the existing research results for systematized control of the service life of concrete structures during design. These pioneering attempts at systematic durability design will help to formulate new ideas and identify the needs for further research of degradation processes and their calculation models. Durability calculations enable not only priority ranking of materials and structural factors, but also produce numerical values of factors for the intended service life.

The idea of this report is to present the methodology of durability design in such a way that it can be combined with traditional mechanical design. This procedure utilizes current results with regard to safety and performance under static, fatigue and impact loading. The greatest change is the highly improved design regarding durability. Through durability design, all durability parameters such as the concrete cover, durability properties of materials, amount of reinforcement and dimensions of the structures are calculated, taking into account in each case actual degradation processes and, where necessary, their interaction.

In order to understand the design methodology and calculation methods, a fairly extensive introduction is given to the statistical background. The reliability theory of structures is applied to durability safety principles. A number of new definitions are also presented.

The planning and writing of this report were done in close cooperation by the RILEM TC 130-CSL working team comprising the following persons:
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Important ideas and comments were contributed by several members of RILEM TC 130-CSL and related Technical Committees in RILEM and CEB, especially Dr Christer Sjöström, supervisor of the working commission, Prof. H.K. Hilsdorf, Dr C.D.Pomeroy and Prof. Peter Schieβl. I should like to express my gratitude to them, to all the members of TC 130-CSL and to all those who contributed to our work.

Asko Sarja
Chairman of RILEM TC 130-CSL
Espoo, August 1994

1
Introduction

1.1
BACKGROUND

Traditionally the durability design of concrete structures is based on implicit rules for materials, material compositions, working conditions, structural dimensions, etc. Examples of such 'deem-to-satisfy' rules are the requirements for minimum concrete cover, maximum water/cement ratio, minimum cement content, crack limitation, air content, cement type and coatings on concrete. These rules are sometimes related to the type of environmental exposure such as indoor climate, wet exposure, presence of frost and deicing salts, sea water and so on. The purpose of all these rules has been to secure robustness for structures, although no clear definition for service life has been presented.

Modern building codes will increasingly be based on the performance of buildings. It must be ensured that this performance exists throughout the service life of the building. With deem-to-satisfy rules it is not possible to give an explicit relationship between performance and service life. For concrete, but also for other building materials, these relationships are not yet available as design tools. They have to be developed.

Especially in the 1990s, clients and owners of buildings have shown increasing interest in setting requirements for the service life of structures. This has been a natural consequence of a greater awareness of quality and costs of buildings. Understanding durability as an essential part of the quality of a building has become self-evident. Also increasingly emphasized is the fact that total costs comprise not only immediate construction costs, but also costs of maintenance and repair. As a result, durability and service life aspects have been stressed in contract briefs. New methods for more accurate durability design of structures have been demanded.

On the other hand, the vast research of the 1970s to the 1990s on concrete durability has produced reliable information on deterioration processes. Based on this knowledge it is now possible to incorporate durability even in the mechanical design of concrete structures.

A structural designer must be able to prove the fulfilment of a service life requirement. For him a simple durability model showing the performance over time or the service life as a function of appropriate design parameters is a valuable tool. With the aid of durability models a designer can make decisions on the required dimensions and material specifications for structures with a service life requirement. Even relatively rough durability models will give a rational basis for this kind of durability design.

Many environmental factors affect the degradation mechanisms of materials, but their precise influence is difficult to predict as they vary greatly locally. Also the properties of the building materials themselves may vary substantially. Because of these and other factors which cause increased scatter, the performance and service life of a structure should preferably be treated stochastically. This means that not only average values, but also distributions, are considered. Stochastic treatment of design problems takes into account the real nature of structural performance, making a reliable design of structures possible.

1.2
AIM OF THE WORK

The aim of the RILEM commission TC 130-CSL has been to work out the theoretical background and the design procedure of the durability design of concrete structures and to present a selection of durability models which are suited for such a design. Additionally, the aim was to present some examples of practical durability design. The report is aimed at assisting designers at the design stage of concrete structures for both normal outdoor conditions and aggressive environments. The purpose is to show how the results of materials research can be transferred to the design of structures.

To this end the work of TC 130-CSL included the following tasks:

1. examination of methods for incorporating degradation of materials into the structural design of structures;

2. study and selection of durability models for the commonest degradation factors of concrete structures;
3. study of stochastic methods of durability design;
4. laying down design procedures for concrete structures with elucidating examples.

It is hoped that this report will also serve as a link towards the development of new design codes of concrete structures. It is evident that this first attempt cannot be complete with all relevant degradations, environmental conditions and types of concrete structures, but by exposing the needs it will pave the way for further durability research and development of design.

   The work of RILEM TC 130 has been linked to that of CEB Commission V, Working Group 1 'Calculation models' and Working Group 2 'Environments'. The role of the CEB work is more formal and directed towards new model codes. The character of the RILEM work can be described as freer, less formal, demonstrative and prenormative.

2
Definitions and explanation of durability concepts

2.1
DEFINITIONS

Degradation	Gradual decrease in performance of a material or a structure.
Degradation factor	Any of the group of external factors, including weathering, biological, stress, incompatibility and use, that adversely affect the performance of building materials and components (Masters and Brandt, 1989).
Degradation mechanism	The sequence of chemical, mechanical or physical changes that lead to detrimental changes in one or more properties of building materials or components when exposed to one or a combination of degradation factors (Masters and Brandt, 1989).
Degradation model	Mathematical function showing degradation with time (or age).
Degradation process	Time-dependent mechanism leading to degradation.
Design service life	Service life used in the design of structures to take into account the timerelated scatter, and to provide a required safety against falling below the target service life.
Deterioration	The process of becoming impaired in quality or value.
Durability	The capability of a building, assembly, component, structure or product to maintain minimum performance over at least a specified time under the influence of degradation factors.
Durability design parameter	A property of a material, a dimension of a structure or its detail or the location of reinforcement influencing the durability of a structure.
Durability model	Degradation, performance or service life model of calculation.
Durability failure	Exceeding the maximum degradation or falling below the minimum performance.
Failure	Exceeding or falling below the limit state (serviceability limit or ultimate limit).
Failure probability	The probability of failure occurring.
Lifetime	Period of time from installation to the moment of examination.
Lifetime safety factor	Coefficient by which the target service life is multiplied to obtain the corresponding design service life.
Limit state of durability	Minimum acceptable state of performance or maximum acceptable state of degradation. The limit state may be set with regard to the ultimate limit or the serviceability limit.
Mechanical design parameter	A property of a material, a dimension of a structure or its detail, or the location of reinforcement influencing the mechanical resistance of a structure.
Performance	Measure to which the structure fulfils a certain function.
Performance model	Mathematical function showing performance with time (or age).
Risk	Multiplication of the probability of failure by the amount of damage.

Service life	Period of time after manufacturing or installation during which all essential properties meet or exceed minimum acceptable values, when routinely maintained (Masters and Brandt, 1989).
Service life model	Mathematical function for evaluating the service life.
Serviceability	Capacity of a building, assembly, component, product or structure to perform the service function(s) for which it is designed and used.
Serviceability limit state	State which corresponds to conditions beyond which specified service requirements for a structure or structural element are no longer met (CEN, 1994).
Target service life	Required service life imposed by general rules, the client or the owner of the structure.
Ultimate limit state	State associated with collapse, or other similar forms of structural failure (CEN, 1994).

2.2
EXPLANATION OF SOME DURABILITY CONCEPTS

2.2.1
Performance and degradation

Performance is generally understood as behaviour related to use. In principle the performance can be related to bearing capacity, stability, safety in use, tightness, hygrothermal properties, acoustic properties, visual appearance etc. (RILEM, 1986). In the context of this report, we assume that performance is a quantifiable property.

Performance is always a function of time, which is why expressions like 'over time' or 'with time' are commonly added to the concept of performance. When time is considered in the evaluation of performance various external factors, called degradation factors, take on great significance. In this way performance is linked to the concept of durability. Durability is the property expressing the ability to maintain the required performance.

Degradation is by definition the gradual decrease in performance over time. Often degradation can be understood as the opposite, or inverse, of performance. Degradation serves an optional way to treat performance problems.

The concepts of performance and degradation over time can be applied at different levels: (1) buildings, (2) structural components, and (3) materials. It is important to identify the level used. However, there may be interactions between levels. If, for instance, the load-bearing capacity of a concrete column or a beam is studied, the problem is actually treated at the structural component level. However, in the long term the load-bearing capacity will be dependent on the degradation of concrete and steel. Thus the performance over time at structural component level must be evaluated by first analysing the rate of change in performance at material level.

The minimum acceptable values for performance (or maximum acceptable values for degradation) are called durability limit states. The limit state is a performance requirement critical to the service life, which can be set with regard to either the ultimate limit or the serviceability limit.

2.2.2
Service life

Service life is the period of time after manufacture during which the performance requirements are fulfilled. As with performance, service life can be treated at different levels. The necessary actions taken at the end of service life depend on the level applied. At building level the end of service life would normally entail complete renovation, reconstruction or rejection of the building. At structural component or material level it would mean replacement or major repair of those components or materials.

On the other hand, the problem of service life can be approached from at least three different aspects:

1. technical
2. functional
3. economic.

Depending on the aspect, one talks about technical, functional or economic service life. Different aspects give rise to different requirements for the object at hand.

Technical requirements are demands for technical performance. Depending on the level of treatment, they include requirements for the structural integrity of buildings, the load-bearing capacity of structures, and/or the strength of materials. Most of these requirements are set in the codes and standards.

Functional requirements are set in relation to the normal use of buildings or structures. For instance, the width and height of a bridge must meet the requirements of traffic both on and under the bridge. In this case the length of service life is not primarily dependent on the technical condition of the structure, but rather on its development in traffic. From the economic point of view a building, structural component or material is treated as an investment and requirements are set on the basis of profitability.

In the context of this report the aspect of service life problems is mainly technical. The technical point of view covers the following sub-aspects:

1. mechanical and other structural performance
2. serviceability and convenience in use
3. aesthetics.

The primary focus is on the mechanical and other structural functioning of concrete structures. The load-bearing capacity of structures can be violated by the degradation of concrete and reinforcement. Structures must be designed so that the minimum safety level is secured during the intended service life despite degradation and ageing of materials.

Defects in materials may also lead to poor serviceability or inconvenience in the use of a structure. For instance, disintegration of concrete in a pavement may cause inconvenient vibration and impacts in vehicles.

Aesthetic aspects are included in the technical requirements if the aesthetic defects of structures are due to degradation or ageing of materials. Then the question of aesthetics can be treated in technical terms.

The exact definition of service life is obscured by the fact that maintenance routines are performed during the service life of a structure. Maintenance can influence the length of service life and should therefore be included in the definition. Thus the addition 'when routinely maintained' as stated in the definition of TC 71-PSL is recommended (Masters and Brandt, 1989).

In the design of structures, the requirement of service life imposed by the client or owner of the building is called the target service life.

2.2.3
Probability of failure

In stochastic durability design, not only target service life but also the definition of maximum allowable probability of not reaching the target service life is necessary. It is called the probability of failure. On the other hand, probability of failure can be defined as the probability of exceeding or falling below a certain limit state, which may be an ultimate limit state or a serviceability limit state.

When failure is caused by degradation of materials, the term 'durability failure' is used as distinct from 'mechanical failure', which is caused by mecahnical loads. However, durability failure may be a partial reason for mechanical failure.

The required probability of failure depends on how the event of failure is defined, and what the consequences of such a failure would be. If the failure is expected to bring serious consequences, the maximum allowable failure probability must, of course, be small. For evaluating the consequences of failure the concept of risk may be valuable. The risk is defined as the multiplication of the probability of failure by the amount of damage (Kraker, de Tichler and Vrouwenvelder, 1982).

In general, when evaluating the required probability of failure, social, economic and environmental criteria should be considered. Social criteria should include:

1. the social importance of a building or structure;
2. the consequences of failure (the number of human lives at stake, etc.);
3. difficulty in the evaluation of the risk level.

Economic criteria may be important. Any interruption in the production of a factory due to a failure may cause considerable economic losses in comparison to the construction cost of the structures.

Environmental and ecological criteria must also be considered. In some cases, structural damage may lead to a serious environmental accident. In normal buildings a long and secure service life is also best in line with ecological principles whereby the consumption of building materials is small and the emissions and energy consumption during the manufacture of materials are minimized.

In the design phase the uncertainties with respect to the final quality of the building are greater than for existing buildings. This implies that for existing buildings the safety margins can be smaller than for buildings in the design phase. For both, the

probability of failure is in principle the same. For the same reason it can be stated that, in assessing the effect of degradations on structural behaviour, the same theory can be applied to existing structures as to structures in the design phase. The only difference will be that in the end, the safety margins for existing structures will be lower than for structures in the design phase.

3
Methods of durability design

3.1
GENERAL

The theory of durability design is in principle based on the theory of safety (or structural reliability) traditionally used in structural design. In this context safety denotes the capacity of a structure to resist, with a sufficient degree of certainty, the occurrence of failure in consequence of various potential hazards to which the structure is exposed. The theory, however, has hitherto been confined more particularly to problems in which time plays only a subordinate part. Now the use of this technique is increasingly advocated for dealing also with durability and service life problems (Siemes, Vrouwenvelder and van den Beukel, 1985).

Although traditionally the methodology of safety has been almost exclusively applied to studies of structural mechanics, the method is by no means restricted to such design problems. The methods presented in the following can in principle be applied to any performance problems dealing with structures or materials.

A new feature in the theory of safety is the incorporation of time into the design problems. It allows the possibility of treating degradation of materials as an essential part of the problem. Safety against failure (falling below the performance requirement) is a function of time. Designing a structure with the required safety now includes a requirement of time during which the safety requirement must be fulfilled. In other words a requirement for the service life must be imposed.

In the design of structures the required service life is called the target service life. The level of safety is expressed as the maximum allowable failure probability.

3.2
THEORY OF FAILURE PROBABILITY AND SERVICE LIFE

The simplest mathematical model for describing the event 'failure' comprises a load variable S and a resistance variable R. In principle the variables S and R can be any quantities and expressed in any units. The only requirement is that they are commensurable.

If R and S are independent of time, the event 'failure' can be expressed as follows: (Kraker, de Tichler and Vrouwenvelder, 1982):

$$\{\text{failure}\} = \{R < S\} \tag{3.1}$$

In other words, the failure occurs if the resistance is smaller than the load.

The failure probability P_f is now defined as the probability of that 'failure':

$$P_f = P\{R < S\} \tag{3.2}$$

Either the resistance R or the load S or both can be timedependent quantities. Thus the failure probability is also a timedependent quantity. Considering $R(\tau)$ and $S(\tau)$ are instantaneous physical values of the resistance and the load at the moment τ the failure probability in a lifetime t could be defined as

$$P_f(t) = P\{R(\tau) < S(\tau)\} \quad \text{for all } \tau \leq t \tag{3.3a}$$

The determination of the function $P_f(t)$ according to equation (3.3a) is mathematically difficult. Usually the resistance and the load cannot be treated as instantaneous physical values. That is why R and S are considered to be stochastic quantities with time-dependent or constant density distributions. By this means the failure probability can usually be defined as

$$P_f(t) = P\{R(t) < S(t)\} \tag{3.3b}$$

According to definition (3.3b) the failure probability increases continuously with time as schematically presented in Figure 3.1. At the moment $t=0$ the density distributions of the load and the resistance are far apart and the failure probability
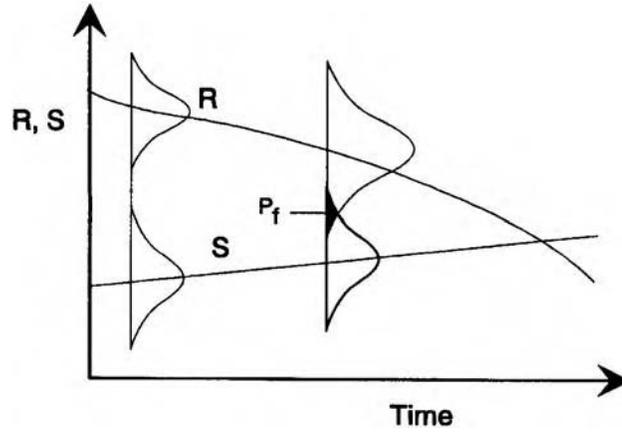
Figure 3.1 The increase of failure probability. Illustrative presentation.

is small at first. With time the distributions approach each other, forming an overlapping area of increasing size. The overlapping areaillustrates the failure probability.

The function $P_f(t)$ has the character of a distribution function. If the service life is defined so that the event '$t_L < t$' is identical with the event 'failure in lifetime t' the distribution function of service life can be defined as

$$F_L(t) = P\{t_L < t\} = P_f(t) \tag{3.4}$$

where F_L is the cumulative distribution of service life.

The probability density function can be determined as the derivative of the distribution function:

$$f_L(t) = \frac{d}{dt} F_L(t) \tag{3.5}$$

At a certain moment of time the probability of failure can be determined as the sum of products of two probabilities: (1) the probability that $R < S$, at $S = s$, and (2) the probability that $S = s$, extended for the whole range of S:

$$P_f = \sum_{\forall s} P\{R < S | S = s\} P\{S = s\} \tag{3.6a}$$

Considering continuous distributions the failure probability P_f at a certain moment of time can be determined using the convolution integral:

$$P_f = \int_{-\infty}^{\infty} F_R(s) f_S(s) ds \tag{3.6b}$$

where $F_R(s)$=the distribution function of R,
  $f_s(s)$=the probability density function of S, and
  s=the common quantity or measure of R and S.

The general solution of the convolution integral with timedependent distributions of R and S may be troublesome. The straightforward solution of the integral is only available in a few cases, i.e. when the distributions of R and S are normal. However, the integral can be solved by approximative numerical methods. The distribution of service life can be obtained by calculating the failure probability values at different moments of time, e.g. t=0, 10, 20, 30, etc. years.

3.3
BASIC FORMULATION OF DURABILITY DESIGN

3.3.1
Principles and methods of design

The basic formulae of durability design can be written according to two optional principles:

  1. performance principle
  2. service life principle.

When the performance principle is used, the basic design formula is written by setting the load S into a relationship with the performance R. The performance, evaluated by a performance model, must be greater than the required load. The performance can be related to the mechanics, physics, chemistry, and functions of the use, or to aesthetics. The load can be either mechanical or environmental.

When the service life principle is used, the service life t_L, evaluated by a service life model, must be greater than the required target service life t_g.

Normally both principles lead to the same results. Which principle is selected depends much on how the design problem is set up, what kind of durability models are available (cf. Chapter 7) and what is the easiest way to treat the problem mathematically.

The method of durability design can be (1) deterministic, (2) stochastic or (3) lifetime safety factor. The principles of these methods are explained below.

3.3.2
Deterministic design method

In deterministic durability design, the load, resistance and service life are used as deterministic quantities. The distributions of these functions are not considered. According to the performance principle the design formula is written as

$$R(t_g) - S(t_g) > 0 \tag{3.7}$$

where t_g is the target service life.

The load S or resistance R, or both, are time-related functions. Degradation and performance models (cf. Chapters 7 and 8) are used as these functions. Design parameters such as structural dimensions and material specifications and environmental coefficients are incorporated into these functions.

The corresponding design formula is written according to the service life principle as follows:

$$t_L - t_g > 0 \tag{3.8}$$

where t_L is the service life function. Service life models including design parameters (cf. Chapters 7 and 8) are used as this function.

The design of structures is performed by selecting an appropriate combination of values for design parameters in such a way that the conditions of equations (3.7) and (3.8) are fulfilled.

When using the service life principle it is important to note that the performance requirement is incorporated into the formula for service life. This is why the performance concept and service life concept lead to a similar design formulation, i.e. the results of the design should be the same irrespective of the method used.

3.3.3
Stochastic design method

In stochastic durability design, the distributions of load, response and service life are also taken into account. The condition is expressed as the probability that the design formula is not true. The design formula can be written according to the performance principle or the service life principle in basically the same way as in deterministic design. However, a requirement for the maximum allowable failure probability is added to the final condition.

According to the performance principle the following requirement must be fulfilled: the probability of the resistance of the structure being smaller than the load within the service period is smaller than a certain allowable failure probability.

Mathematically the requirement is expressed as

$$P\{\text{failure}\}_{t_g} = P\{R - S < 0\}_{t_g} < P_{\text{fmax}} \tag{3.9}$$

where $P\{\text{failure}\}t_g$=the probability of failure of the structure within t_g, and

P_{fmax}=the maximum allowable failure probability.

The problem can be solved when the distributions of the load and the resistance are known. In section 4.2 a solution is presented for cases where the load and resistance are normally distributed.

When the service life principle is applied the requirement is set as follows: the probability that the service life of a structure is shorter than the target life is smaller than a certain allowable failure probability. Mathematically the condition is written as

$$P\{\text{failure}\}_{t_g} = P\{t_L < t_g\} < P_{\text{fmax}} \tag{3.10}$$

The problem can be solved if the distribution of service life is known. If the form of distribution is not known, it must be assumed to follow some known distribution type. A solution for cases where the distribution of service life is assumed to be lognormal is presented in section 4.3.

3.3.4
Lifetime safety factor method

When the formulae for load, resistance and service life are complex and many degradation factors affect the performance of structures, application of the above stochastic design method may be difficult. In such cases it may be reasonable to apply the lifetime safety factor method. Although the method is based on the theory of safety and reliability, formulation of the design procedure returns to the deterministic form. This is possible by changing the requirement of target service life to the corresponding requirement of design service life. In practice the design service life is determined by multiplying the target service life by a lifetime safety factor:

$$t_d = \gamma_t t_g \tag{3.11}$$

where t_d=the design service life,
  γ_t=the lifetime safety factor and
  t_g=target service life.
  The design formulae can then be written by applying either the performance principle or the service life principle as follows:

$$R(t_d) - S(t_d) \geq 0 \tag{3.12}$$
$$t_L - t_d > 0 \tag{3.13}$$

The value of the lifetime safety factor depends on the maximum allowable failure probability. The lifetime safety factor must be calibrated with the results of stochastic design methods as presented in Chapter 5.

The lifetime safety factor method is used in structural durability design as discussed in Chapter 6. The application of the method is especially justified in structural design because the stochasticity of loads and strengths is also taken into account by safety factors, the method in this way being in principle familiar to mechanical structural design.

4
Examples of durability design by stochastic methods

4.1
GENERAL

In order to use stochastic design methods, some assumptions must be made concerning the form of distributions. Distribution types that can be used for the evaluation of service life or performance of structures include:

1. normal (Gaussian) distribution
2. log-normal distribution
3. exponential distribution
4. Weibull distribution
5. gamma distribution.

When the performance principle is applied, the commonest assumption is that either the load or the resistance, or both, are normally distributed. When the service life principle is used, the distribution of service life is often assumed to be log-normal, i.e. normal on a logarithmic time scale.

4.2
DESIGN WITH PERFORMANCE PRINCIPLE WHEN R AND S ARE NORMALLY DISTRIBUTED

4.2.1
Theory

When the performance principle is used in durability design and the resistance, $R,$ and the load, $S,$ are normally distributed quantities, the failure probability can be determined using the test index β:

$$\beta(t) = \frac{\mu[R, t] - \mu[S, t]}{(\sigma^2[R, t] - \sigma^2[S, t])^{1/2}} \tag{4.1}$$

where μ denotes the mean and σ the standard deviation.

The test index β is (0,1)-normally distributed. The failure probabilities corresponding to β are available as tables or as functions in up-to-date spreadsheet applications. In structural design the index β is also referred to as the safety index or the reliability index.

Very often either R or S is constant. Then the relationship in equation (4.1) is reduced to the forms

$$\beta(t) = \frac{r - \mu[S, t]}{\sigma[S, t]} \tag{4.2}$$

or

$$\beta(t) = \frac{\mu[R, t] - s}{\sigma[R, t]} \tag{4.3}$$

where r and s are constants.

When R is constant and S is a time-related function approximated by a degradation model, the problem is called a degradation problem. Likewise when S is constant and R is a time-related function approximated by a performance model, theproblem is called a performance problem.
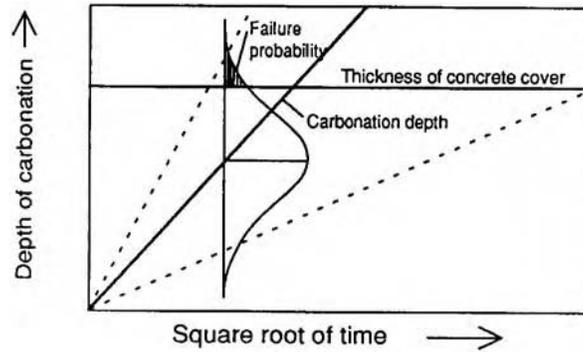
Figure 4.1 Schematic representation of the process of carbonation in a concrete cover.

As the means and standard deviations are dependent on time, so is index *β*. To obtain the distribution of service life the failure probabilities must be solved with several values of *t* (e.g. *t*=0, 10, 20, etc. years).

4.2.2
Example

The process of carbonation is studied near the surface of a reinforced concrete structure. Failure is assumed to occur when carbonation depth exceeds the depth of reinforcement (Kasami *et al.,* 1986; Sentler, 1984). (As a result the reinforcement is depassivated and corrosion is initiated.)

In this case we have a degradation problem with a timerelated carbonation process *(S)* and a constant concrete cover *(R)*. Carbonation is assumed to proceed as related to the square root of time:

$$\mu(D) = K_{c}t^{1/2} \tag{4.4}$$

where $\mu(D)$=the mean of the depth of carbonation (mm),

K_c=the carbonation rate factor (mm/year$^{1/2}$), and

t=time (or age in years).

The carbonation rate factor depends on the strength and composition of the concrete (cf. section 8.5.3):

$$K_{c} = c_{\text{env}}c_{\text{air}}a(f_{\text{ck}} + 8)^{b} \tag{4.5}$$

where C_{env}=the environmental coefficient,

C_{air}=the coefficient of air content, concrete (MPa) and

f_{ck}=the characteristic cubic compressive strength of concrete (MPa) and

a, b=constants (depending on the binding agent).

Values of parameters and coefficients are listed in Tables 8.3–8.5. The depth of carbonation is assumed to be normally distributed. The coefficient of variation (ratio of standard deviation to mean) is assumed to be constant.

Figure 4.1 shows the process of carbonation in the concrete cover. When the mean of the carbonation depth increases with time, the standard deviation also increases, keeping the ratio of variation constant. That part of the distribution of carbonation depth that exceeds the thickness of the concrete cover shows the failure probability.

(a)
Task 1

Find the distribution function and probability density function of service life when:

1. the structure is sheltered from rain;
2. the concrete is made of Portland cement, no air entrainment;
3. the characteristic compressive strength of concrete is 30 MPa:
4. the thickness of concrete cover is 25 mm.

In this case the thickness of concrete cover is also assumed to be a stochastic quantity. For the carbonation depth the coefficient of variance is 0.6 and the coefficient of variance of the concrete cover is 0.2.

The test index *β* is calculated as follows (cf. equation (4.1):
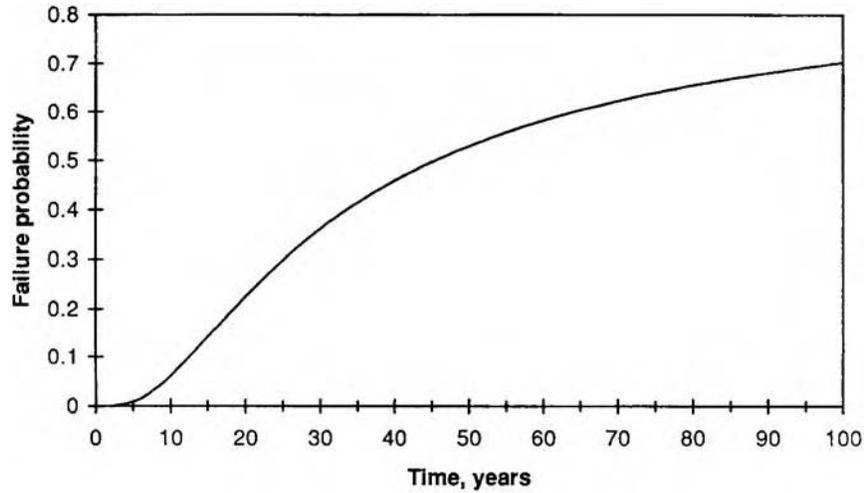
Figure 4.2 Probability distribution function of service life.

$$\beta = \frac{25 \text{ mm} - 1800(30+8)^{-1.7}t^{1/2} \text{ mm}}{[(0.6 \times 1800(30+8)^{-1.7}t^{1/2})^2 + (0.2 \times 25)^2]^{1/2} \text{ mm}}$$

(4.6)

The probability distribution function and probability density function are presented in Figures 4.2 and 4.3. Note that the probability density function is not normally distributed. The probability density peaks rapidly, then slowly decreases with increasing lifetime.

(b)
Task 2

What is the required concrete cover if the target service life is 60 years with a 10% fractile (probability of failure)? The calculation is to be performed for all binding agents in Table 8.5 and for the characteristic compressive strengths of 30, 50 and 70 MPa.

In the case of normal distribution, a fractile of 10% corresponds to a β value of 1.28. Thus the problem can be formulated as follows: what thickness of concrete cover, X, will produce a β value of 1.28? For example, for fly ash concrete and compressive strength 50 MPa the value of β would be

$$\beta = \frac{X - 360(50+8)^{-1.2} \times 60^{1/2} \text{ mm}}{[0.6 \times 360(50+8)^{-1.2} \times 60^{1/2})^2 + (0.2X)^2]^{1/2} \text{ mm}}$$

4.7
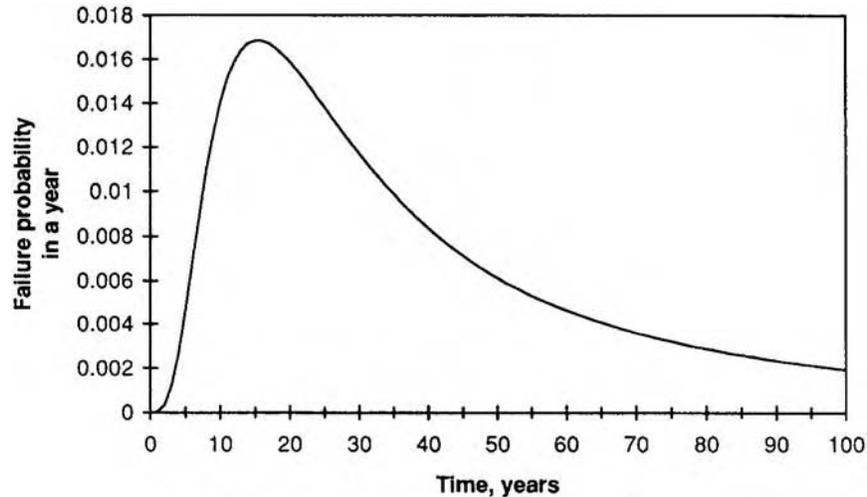
Figure 4.3 Probability density function of service life.

Solution of X is not too easy as this parameter is found in both the numerator and denominator. However, the problem can be solved unequivocally. The results are presented in Table 4.1.

Table 4.1 Required thickness of concrete cover (mm)

Binder	Cover (mm) with compressive strength (MPa) of		
30	50	70	
Portland cement	55	27	16
Portland cement+fly ash 28%	68	41	29
Portland cement+silica fume 9%	75	45	32
Portland cement+blast furnace slag	68	41	29

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4.3

DURABILITY DESIGN WITH LOG-NORMAL SERVICE LIFE DISTRIBUTION

4.3.1

General

</div>

As the example above shows, although the degradation (depth of carbonation) was normally distributed around the mean, the service life distribution showed a strong decline towards short service lives. The distribution of service life in fact most often follows this pattern. The probability density peaks rapidly before decreasing gently towards zero when approaching an infinitely long service life.

For the purpose of durability design it may often be useful to make an assumption for the type of distribution of service life. It should be selected from among those with an inclined appearance such as the log-normal distribution (Figure 4.4). A log-normal distribution means that the service life is distributed normally on a logarithmic time scale.

In a normal distribution (with linear time scale) a diminishing but still finite probability continues in both directions. This would allow negative service lives, which of course cannot be approved. The fact that negative values are impossible on a logarithmic time scale makes the assumption of a log-normal distribution for service life both suitable and plausible.

The theory of log-normal service life was first elaborated for durability problems in concrete structures by the Dutch authors Siemes, Vrouwenvelder and van den Beukel (1985). The theory also contains the necessary methods for evaluation of the standard deviation of service life.

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4.3.2

Standard deviation of service life

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The standard deviation of service life is determined using the model formula for mean service life, and the standard deviations of parameters in the formula. The model formula can generally be written as
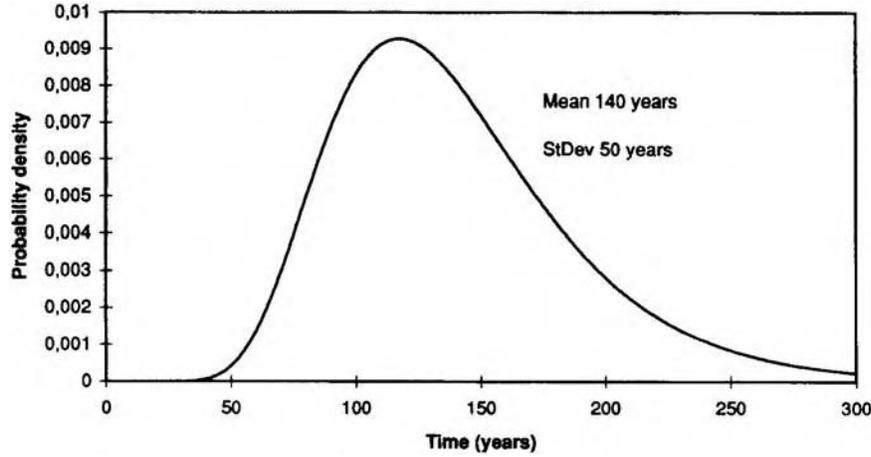
Figure 4.4 Log-normal distribution of service life.

$$t_L = t_L(x_1, x_2, \ldots x_n) \tag{4.8}$$

where t_L=the service life,

x_i=a parameter of the material or condition, or a structural dimension ($i=1\ldots n$).

Equation (4.8) is expected to yield the arithmetic mean of service life. $\mu(t_L)$, when the mean values of parameters $x_1, x_2, \ldots, x_n$ are inserted into it.

The standard deviation is evaluated using the following formula:

$$\sigma^2(t_L) = \sum_{i=1}^{n} \left(\frac{\partial t_L}{\partial x_i} \sigma(x_i) \right)^2 \tag{4.9}$$

where $\sigma(t_L)$=the standard deviation of service life,

$\underline{\partial t_L}$=the standard deviation of parameter x_i,

∂x_i=the partial derivative of the service life with respect to parameter x^i, and

n=the number of variables.

4.3.3
Probabilities of log-normal service life distribution

If the service life distribution is log-normal with a mean value $\mu(t_L)$ and standard deviation $\sigma(t_L)$, function $\gamma = \ln(t_L)$ is normally distributed and its mean value and standard deviation are obtained from the following formulae:

$$\sigma^2(Y) = \ln \left[1 + \left(\frac{\sigma(t_L)}{\mu(t_1)} \right)^2 \right] \tag{4.10}$$

$$(4.11) \qquad\qquad \mu(Y) = \ln\mu(t_L)^{-1/2}\sigma^2(Y)$$

The probability of the service life being shorter than a certain time t is as follows:

$$P\{t_L < t\} = P\{\ln t_L < \ln t\} = P\{Y < \ln t\} = \phi(-\beta) \tag{4.12}$$

where

$$\beta = \frac{\mu(Y) - \ln t}{\sigma(Y)} \tag{4.13}$$

and ø is the cumulative density function of the standard normal distribution ($\mu=0$, $\sigma=1$).

4.3.4
Example

Equation (4.14) is used to calculate the mean service life with regard to corrosion of the reinforcement. The first term in the equation is the initiation time of corrosion (cf. section 8.5.3). The second term is the cracking time of corrosion (cf. section 8.5.4).

$$\mu(t_L) = \frac{C^2}{(c_{env}c_{air}a(f_{ck}+8)^b)^2} + \frac{80C}{Dr} \tag{4.14}$$

where c_{env}=the environmental coefficient,
  c_{air}=the coefficient of air content,
  C=the thickness of the concrete cover (mm),
  D=the diameter of the rebar (mm),
  f_{ck}=the characteristic cubic strength of concrete (MPa),
  r=the rate of corrosion in rebars, and
  a, b=constants.
  See the values of c_{env}, c_{air}, a and b in Tables 8.3–8.5.
  In the formula for standard deviation (equation (4.9)), the standard deviations or the coefficients of variation of all parameters are needed. In Table 4.2 a choice has been made for the mean values and coefficients of variation for the problem variables. If the coefficients of variation are not known they must be evaluated.
  To be able to use equation (4.9), partial differentiation with respect to every variable must be performed. If the model formula for the mean is not too complicated it can be performed by means of elementary algebra.

Table 4.2 Problem variables and their means and coefficients of variation

Variable	Dimension	Mean (μ)	Coefficient of variation (σ/μ)
C_{env}	–	0.5 1	0.5
C_{air}	–	1	0.4
C	mm	15 20 25 30	0.3
		35	
D	mm	25	0
f_{ck}	MPa	30	0.2
a	–	1800	0
b	–	−1.7	0
r	μm/year	16 1	0.5

The mean and the standard deviation of service life can then be calculated by inserting the mean values of the variables into the formulae. For instance, in our example, if the structures are sheltered from rain (c_{env}=1, r=1 μm/year) and the concrete cover is 25 mm we get

$$\mu(t_L) = 125 \text{ years}$$
$$\sigma(t_L) = 82 \text{ years}$$
$$\sigma(Y) = 0.595$$

and

$$\mu(Y) = 4.65$$

Using the tables of standard normal distribution and β values for t=10, 20, 30 etc. years, the corresponding failure probabilities can then be determined.
  Figures 4.5 and 4.6 show the cumulative probability functions calculated for different thicknesses of the concrete cover. The figures show the probability of the service life being shorter than a given target service life, t_g. The required thickness of the concrete cover can be read from the figures as a function of the target service life and the accepted failure probability.
  When using the log-normal method some requirements must be set for the model formula. One restriction is that the service life model cannot be too complicated, as it must be differentiable with respect to every parameter in it. Another requirement is that the parameters must not be related to each other; if they are, some bias is expected in the evaluation of standard deviation.
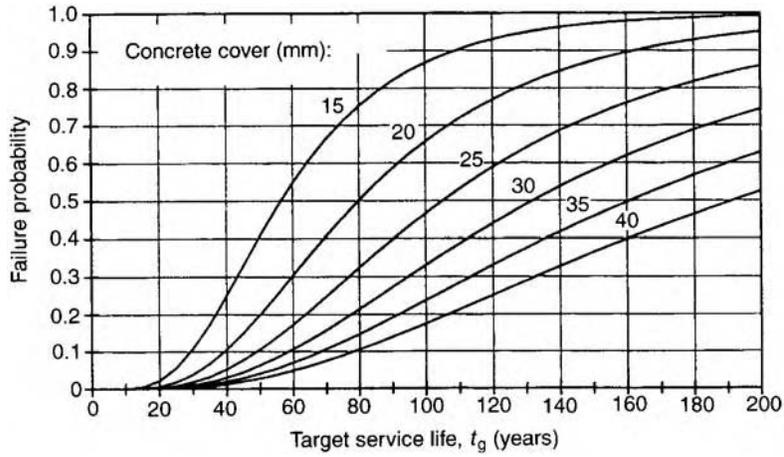
Figure 4.5 Probability of service life being shorter than the target service life when the structures are sheltered from rain. The characteristic cubic strength of concrete is 30 MPa and the diameter of the rebar is 25 mm.
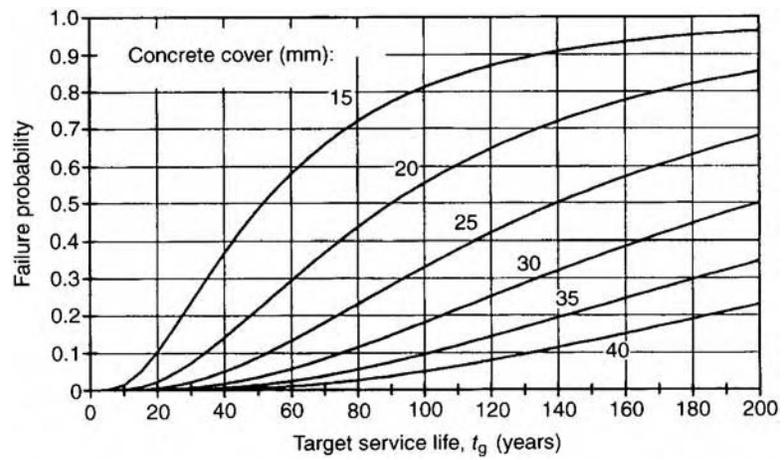
Figure 4.6 Probability of service life being shorter than the target service life when the structures are exposed to rain. The characteristic cubic strength of concrete is 30 MPa and the diameter of the rebar is 25 mm.

5
Determination of lifetime safety factors

5.1
THE MEANING OF LIFETIME SAFETY FACTOR

The lifetime safety factor method in the design of structures is based on the same safety principles as those applied in the stochastic methods above. With the aid of the lifetime safety factor the design problem returns to the form of deterministic design.

Figure 5.1 shows a distribution of service life and the relationships between the target service life, failure probability and mean service life. The lifetime safety factor is the relation of mean service life to the target service life.

$$\gamma_t = \frac{\mu(t_L)}{t_g} \tag{5.1}$$

where γ_t=the central lifetime safety factor,
   $\mu(t_L)$=the mean service life and
   t_g=the target service life.
   Using the lifetime safety factor, the requirement of target service life (corresponding to a maximum allowable failure probability) is converted to the requirement of mean service life. The reason for this is that the durability models available to designers show only the mean performance, or the mean degradation, or the mean service life. As designers operate with mean functions, every requirement of target service life must first be interpreted in terms of the corresponding mean service life.
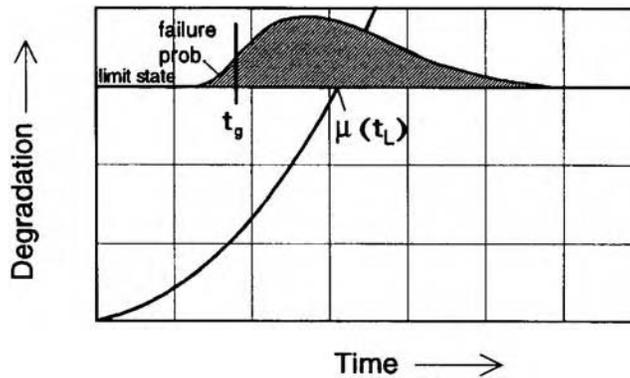
Figure 5.1 Relationship between mean service life and target service life.

In the durability design of structures, the mean service life is often approximated by service life models which show the crossing-point of the degradation curve with the limit state of durability (Figure 5.1). The mean service life evaluated by the service life model must be greater than or equal to the **design service life,** which is the product of the lifetime safety factor and target service life.

$$\mu(t_L) \geq t_d \tag{5.2}$$
$$t_d = \gamma_t t_g \tag{5.3}$$

where t_d is the design service life.
   Note that the mean service life is not necessarily the same as the service life corresponding to 50% failure probability, which is the median service life.
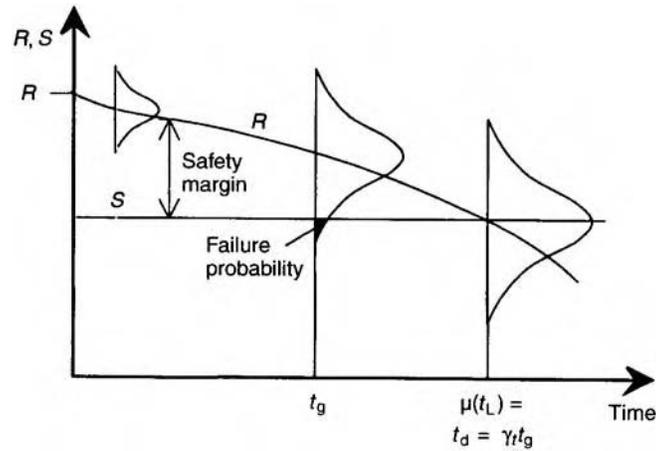
Figure 5.2 The meaning of lifetime safety factor in a performance problem.
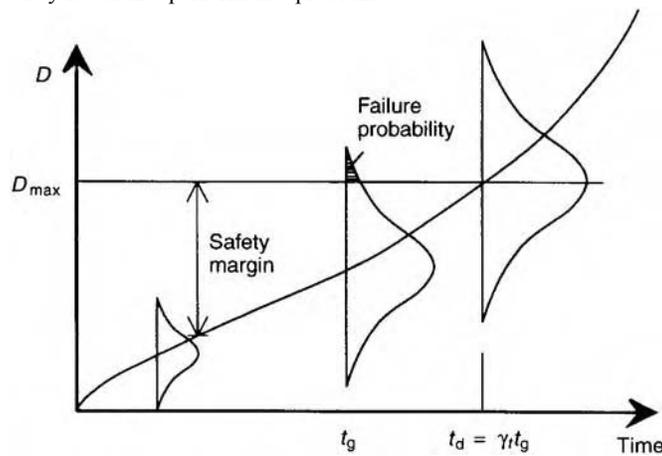
Figure 5.3 The meaning of lifetime safety factor in a degradation problem.

The lifetime safety factor depends on the maximum allowable failure probability, the smaller the maximum allowable failure probability, the greater is the lifetime safety factor. The lifetime safety factor also depends on the form of service life distribution.

Figure 5.2 illustrates the meaning of lifetime safety factor when the design is carried out according to the performance principle. The curves in the figure correspond to a situation common to the design problem of load-bearing capacity. S could be a constant load effect on the structure, and $R(t)$ a structural capacity which must be greater than S to avoid failure. The function $R(t)-S$ is called the safety margin.

The structural performance decreases with time due to degradation of materials. The crossing-point of the $R(t)$ curve with the minimum load effect S gives the mean service life which equals the design service life. If the target service life were the same as the design service life, roughly half of all structures would fall below the load requirement. To have a smaller failure probability at the target service life, the design service life must be longer than the target service life.

In general, $R(t)$ is the performance capacity of the structure. S, which in the case of structural performance is the load effect, is in many other applications of durability design replaced by the minimum performance capacity of the structure, R_{min}.

Performance behaviour can always be translated into degradation behaviour. By definition, degradation is a decrease in performance. The transformation is performed by the following substitutions:

(5.4)
$$R_0 - R(t) = D(t)$$

(5.5a)
$$R_0 - S = D_{max}$$

or

(5.5b)
$$R_0 - R_{min} = D_{max}$$

Figure 5.3 shows the principle of design in a degradation problem. $D(t)$ is the degrading effect of environmental loading on the performance of the structure. The degradation curve crosses the maximum degradation at the design service life, which must be longer than the target service life. The range $D_{max}-D(t)$ is the safety margin.
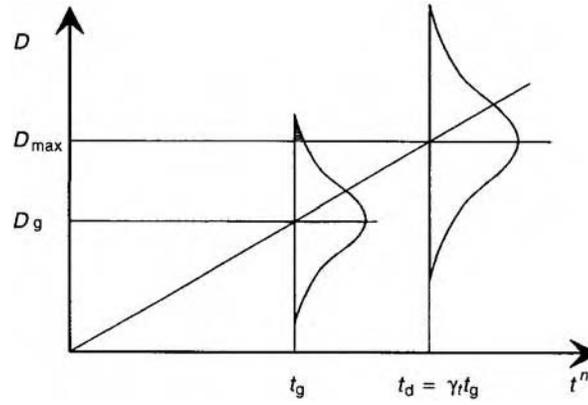
Figure 5.4 Deterioration of lifetime safety factors with normally distributed D.

A prerequisite for using the lifetime safety factor method is, of course, that the correct values for lifetime safety factors are known. The rest of this chapter focuses on the determination of lifetime safety factors by different methods. Practical applications of the lifetime safety factor method are presented in Chapter 6 (Structural durability design).

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5.2

LIFETIME SAFETY FACTORS AS DETERMINED BY STOCHASTIC METHODS

5.2.1

Determination of lifetime safety factor with a normally distributed degradation function

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Let us consider that the degradation function is of the following form:

$$\mu(D(t)) = at^n \tag{5.6}$$

where $\mu(D(t))$=the mean of degradation,
    a=the constant coefficient,
    t=time, and
    n=the exponent.
    The exponent n may in principle vary between $-\infty$ and $+\infty$.
    The coefficient a is fixed when the mean service life is known:

$$a = \frac{D_{\max}}{\mu(t_L)^n} \tag{5.7}$$

Degradation is assumed to be normally distributed around the mean. It is also assumed that the standard deviation of D is proportional to the mean degradation, the coefficient of variation being constant, v_D.

Figure 5.4 shows the degradation as a function of t^n. The value of γ_t can be determined as follows (Vesikari, 1995). The index β of standard normal distribution at t_g is

$$\beta = \frac{D_{\max} - D_g}{v_D D_g} = \frac{1}{v_D}\left(\frac{D_{\max}}{D_g} - 1\right) \tag{5.8}$$

where $D_{\max}$=the maximum allowable degradation,
    D_g=the mean degradation at t_g, and
    v_D=the coefficient of variation of degradation.
    From Figure 5.4 we get

$$\frac{D_{\max}}{D_g} = \frac{(\gamma_t t_g)^n}{t_g^n} = \gamma_t^n \tag{5.9}$$

By assigning this to equation (5.8) we obtain

$$\gamma_t = (\beta v_D + 1)^{1/n} \tag{5.10}$$

The lifetime safety factor depends on β (respective to the maximum allowable failure probability at t_g), the coefficient of variation of D and the exponent n. Note that the lifetime safety factor is not directly dependent on t_g.

Table 5.1 Values of lifetime safety factor. Degradation curve is linear (n=1)

Failure probability (%)	β	γ_t with v_D of	
		0.5	1.0
1	2.33	2.16	3.33
5	1.64	1.82	2.64
10	1.28	1.64	2.28
50	0	1.00	1.00

Table 5.2 Values of lifetime safety factor. Degradation is proportional to the square root of time ($n=\frac{1}{2}$); retarding degradation

Failure probability (%)	β	γ_t with v_D of	
		0.5	1.0
1	2.33	4.64	11.06
5	1.64	3.32	7.00
10	1.28	2.69	5.21
50	0	1.00	1.00

Table 5.3 Values of lifetime safety factor. Degradation is proportional to the square of time (n=2); accelerating degradation

Failure probability (%)	β	γ_t with v_D of	
		0.5	1.0
1	2.33	1.47	1.82
5	1.64	1.35	1.63
10	1.28	1.28	1.51
50	0	1.00	1.00

In the following we study the values of γ_t as a function of β, v_D and n. For n we assume

$$n = 1 \text{ (linear degradation)}$$
$$n = 0.5 \text{ (retarding degradation)}$$
$$n = 2 \text{ (accelerating degradation)}$$

Tables 5.1–5.3 show the values of lifetime safety factor for β values corresponding to failure probabilities of 1, 5, 10, 20 and 50%.

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5.2.2
Determination of lifetime safety factors with a lognormally distributed service life function

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In the following we assume that the distribution function of service life is log-normal. This distribution has already been introduced in section 4.3.

By inserting , $t = t_g, \mu(t_L) = \gamma_t t_g$ and $v_L = \sigma(t_L)/\mu(t_L)$ i into equations (4.10), (4.11) and (4.13) the following formula is obtained for β:

$$\beta = \frac{\ln(\gamma_t t_g) - \frac{1}{2}\ln(1 + v_L^2) - \ln(t_g)}{[\ln(1 + v_L^2)]^{1/2}} \tag{5-11}$$

where γ_t=the lifetime safety factor,
β=the index of standard normal distribution, and
v_L=the coefficient of variation of service life.
For the lifetime safety factor we obtain

$$\gamma_t = \exp\left\{\beta[\ln(1 + v_L^2)]^{1/2} + \frac{1}{2}\ln(1 + v_L^2)\right\} \tag{5.12}$$

Thus the lifetime safety factor depends on the index β and the coefficient of variation of service life but not directly on the length of service life. Table 5.4 shows the lifetime safety factors calculated with varying β values.

Table 5.4 Lifetime safety factors determined by the log-normal method

Failure probability (%)	Lifetime safety factor, γ_t with coefficient of variation, v_L, of	
0.5	1.0	
1	3.36	9.81
5	2.43	5.56
10	2.05	4.11
50	1.12	1.41

5.3
DETERMINATION OF LIFETIME SAFETY FACTORS FOR STRUCTURAL DURABILITY DESIGN

5.3.1
Common principles

The basis of a conventional design procedure for concrete structures can be expressed as

$$R_d - S_d > 0 \tag{5.13}$$

or

$$\Theta_d > 0 \tag{5.14}$$

where R_d=the design value of the load-bearing capacity,

S_d=the design value of the load and

Θ_d=the safety margin calculated with the design values of load-bearing capacity and the load ($=R_d-S_d$).

The design value of the load-bearing capacity is determined by dividing the characteristic strength of concrete by a material safety factor defined for concrete, and the characteristic strength of reinforcing steel by the safety factor for steel, and then using normal design formulae for calculating the capacity. The design value of the load is obtained by multiplying the characteristic loads by their corresponding safety factors.

As the values of partial safety factors are calibrated according to the requirement of mechanical safety, the state of R_d being equal to S_d corresponds to the required **safety level**. 'Failure' of R_d being smaller than S_d does not mean a collapse or other mechanical failure, but running under the safety level requirement. Safety in terms of not falling below the safety level is called the durability safety, and is distinct from the mechanical safety. Respectively the term 'durability failure' is used distinct from the mechanical failure.

The structural durability design is very similar to normal structural design. Here the load-bearing capacity is, however, a time-dependent quantity because of the time-related material losses in concrete and steel. Possibly also the load is time-dependent. Thus the basic design formula (equation (5.14)) is rewritten as

$$\Theta_d(t_d) > 0 \tag{5.15}$$

where t_d is the design service life.

Figure 5.5 shows the reduction of the design safety margin with time. The 0-line in the figure shows the minimum value of Θ_d which fulfils the safety level requirement. The initial Θ_d must be higher than this value to be adequate at the end of the service life. Examination of the design safety margin with time shows how the probability of falling below the safety level is increased with time due to reduction of the mean of Θ_d and the increase in scatter.

Figure 5.5 also shows why it is necessary to use 'design service life' instead of 'target service life' in the design equation (5. 15). If the structure were designed using the target service life, t_g, some 50% of structures would not meet the safety level requirement at the age of target service life. To have a smaller fractile of non-qualified structures at the target service life, a longer design service life must be used in the design formulae.

The necessary difference between the design service life and the target service life is specified by the lifetime safety factor. The method closely resembles that for dealing with the stochasticity of strengths and loads in the design formula. The target service life is now multiplied by the lifetime safety factor to obtain the design service life.

The lifetime safety factor must be calibrated in such a way that the probability of falling short of the safety level requirement because of degradation (probability of durability failure) is smaller than that required. The smaller the required probability of durability failure, the greater is the lifetime safety factor.

There are two principles for determination of the required probability of durability failure and the corresponding lifetime safety factor:

1. separated safety principle
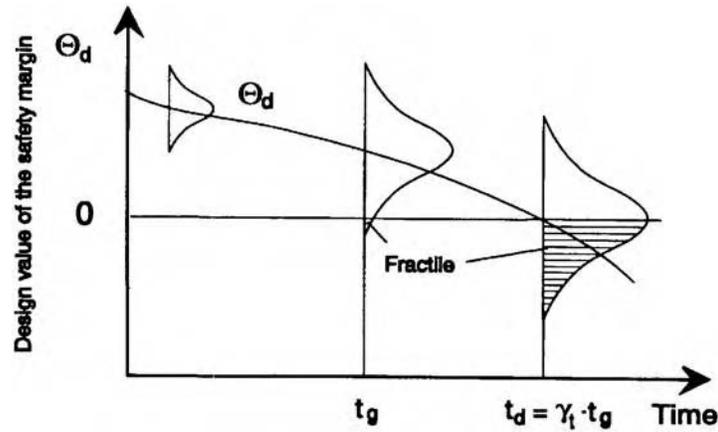
Figure 5.5 Decrease of the design value of the safety margin, and effect of the lifetime safety factor calculated with design values of loads and load-bearing capacities.

2. combined safety principle.

According to the separated safety principle the requirement of mechanical safety and the requirement of durability safety are imposed separately.

In the principle of combined safety, the durability safety is understood as being one of several safety elements that produce the total mechanical safety. Thus the required probability of durability failure is linked to the requirement of mechanical safety.

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5.3.2
Determination of lifetime safety factors by the separated safety principle

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The separated safety principle means a separate specification of the requirements for durability safety and mechanical safety. However, in this context it has been decided to apply the requirements of mechanical safety according to Eurocode 1 also to the durability safety. By this requirement the reliability against durability failure at the end of the target service life is the same as the short-term reliability against a mechanical failure in the ordinary structural design. Some grounds for this decision are presented in section 5.3.3(d).

The required safety indexes for ordinary design defined by Eurocode 1 are as follows:

$$\text{Ultimate limit state: } \beta = 3.8 \quad (P_f = 7.2 \times 10^{-5})$$

$$\text{Serviceability limit state: } \beta = 1.5 \quad (P_f = 6.7 \times 10^{-2})$$

The ultimate limit state is associated with collapse, or with other similar forms of mechanical failure. It generally corresponds to the maximum load-carrying resistance of a structure or structural part. Serviceability limit states correspond to conditions beyond which specified service requirements for a structure or structural element are no longer met. The serviceability requirements concern (CEN, 1994)

1. functioning of the construction or parts of them;
2. the comfort of people;
3. the appearance.

If a durability failure in an ultimate state design does not lead to serious consequences, a slightly smaller value for β_t than that specified in Eurocode 1 for mechanical safety can be allowed. On the other hand, if the consequences of a durability failure in serviceability state design are noticeable and the repair costs are high, a slightly higher value for the safety index than that stated in Eurocode 1 is required. Thus the final durability safety requirements are as follows:

1. ultimate limit state:

$$\beta_t = 3.8 \quad \text{(serious consequences of a durability failure)}$$

$$\beta_t = 3.1 \quad \text{(no serious consequences of a durability failure)}$$

2. serviceability limit state:

$$\beta_t = 2.5 \quad \text{(consequences of a durability failure are noticeable}$$
$$\text{and the repair costs are high)}$$
$$\beta_t = 1.5 \quad \text{(no noticeable consequences of a durability failure)}$$

The corresponding lifetime safety factors are determined using equation (5.10). The linear mode of the degradation process is applied (n=1). In Table 5.5 the lifetime safety factors are determined for different values of the coefficient of variation (v_D).

5.3.3
Determination of lifetime safety factors by the combined safety principle

(a)
Principle

According to the combined safety principle the requirement of durability safety is determined on the basis of its influence on the mechanical safety. The required lifetime safety factors are determined by an analysis performed with the **characteristic values** of loads and load-bearing capacities, as the requirements of the total mechanical safety are defined with respect to characteristic values. In addition, the extra scatter due to degradation (Figure 5.5) is totalled with the ordinary scatter of the safety margin (ordinary scatter of loads and load-bearing capacity).

Table 5.5 Values of the lifetime safety factor

Limit state	Safety class (consequence of durability failure)	Probability of durability failure (after t_g), P_f	Safety index (after t_g), β_t	Lifetime safety factor, γ_t, with v_D of		
				0.4	0.6	0.8
Ultimate limit	1. (serious)	7.2×10^{-5}	3.8	2.52	3.28	4.04
	2. (non-serious)	9.7×10^{-4}	3.1	2.24	2.86	3.48
Service ability limit	1. (noticeable)	6.2×10^{-3}	2.5	2.00	2.50	3.00
	2. (not noticeable)	6.7×10^{-2}	1.5	1.60	1.90	2.20

(b)
Requirements for ordinary design

The Eurocode requirements for the mechanical safety (cf. section 5.3.2) refer to the safety margins calculated with characteristic values of material properties and loads. This allows the graphical presentation of the safety requirements in the ultimate limit state and in the serviceability limit state as shown in Figure 5.6. In Figure 5.6, Θ_k is the mean of the 'characteristic' safety margin calculated as

$$\Theta_k = R_k - S_k \tag{5.16}$$

where R_k=the characteristic load-bearing capacity (ultimate limit state) or other structural performance (serviceability limit state) and

S_k=the characteristic load.

σ is the standard deviation of the safety margin:

$$\sigma = \sigma(\Theta_k) = \frac{\Theta_k}{\beta} \tag{5.17}$$

(c)
Derivation of requirements for durability design

In durability design the mechanical safety index after the service life must be at least β_m, which as a rule can be the same as that defined for the ultimate limit state in ordinary design. However, in cases where no serious consequences of a structural failure are expected, the safety index β_m could be a little smaller than that specified for an ordinary designed structure. The reason is that in ordinary (traditional) design the safety is only specified at the start of service life. As a result of degradation the actual safety is bound to fall under the specified level towards end of service life. Thus the dimensions of a durability designed
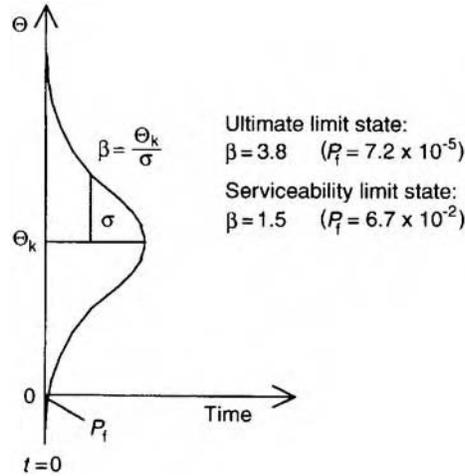
Figure 5.6 Safety requirements for ordinary design according to Eurocode 1.

structure with a lowered safety index β_m are probably more consistent with those of a traditionally designed structure. However, the safety of a durability designed structure is under control throughout the service life.

 For the above reason, the same safety classification presented in section 5.3.2 is now applied to mechanical safety:

1. ultimate limit state:

$$\beta_m = 3.8 \quad \text{(serious consequences of a durability failure)}$$
$$\beta_m = 3.1 \quad \text{(no serious consequences of a durability failure)}$$

2. serviceability limit state:

$$\beta_m = 2.5 \quad \text{(consequences of a durability failure are noticeable}$$
$$\text{and the repair costs are high)}$$
$$\beta_m = 1.5 \quad \text{(no noticeable consequences of a durability failure)}$$

In the following analysis it is assumed that the loads and material strengths remain constant throughout the service life of the structure, and their effects on the safety margin are determined in basically the same way as in ordinary design. The degradation of materials is considered to affect the cross-sectional dimensions of materials (cf. Chapter 6) by both reducing the means and increasing the standard deviations, with corresponding effects on the safety margin.

 The time (or degradation)–related extra scatter is taken into account by the lifetime safety factor, $\gamma_t (\gamma_t > 1)$, in such a way that the determinative time in the design is the design time, t_d, which is obtained by multiplying the target service life, t_g, by the lifetime safety factor. This means that, when expressing the basic formulae of durability design for the time $t=t_d$, the time-related extra scatter is not taken into account as the effect of the scatter is included in the safety factor, γ_t. The safety factor is determined in such a way that the safety index, β, at the time $t=t_d$, when only the reduction of the mean is taken into account, is the same as that for the time $t=t_d$, when the extra scatter due to degradation is also taken into account (Vesikari, 1995).

 In the following analysis the characteristic safety margin $\Theta(t)$, defined as in equation (5.16), is expressed as

$$\Theta(t) = \Theta_o - \Delta\Theta(t) \tag{5.18}$$

where Θ_o=the mean of the characteristic safety margin at t=0 and
 $\Delta\Theta$=the decrease in the characteristic safety margin due to degradation.
 σ_o is the standard deviation of Θ_o (constant) and σ_t is the standard deviation of $\Delta\Theta$ (due to degradation, corresponds to the scatter in Figure 5.5). We then write

$$\sigma_{tot} = (\sigma_o^2 + \sigma_t^2)^{1/2} \tag{5.19}$$

$$\alpha_t = \frac{\sigma_t}{\sigma_{tot}} \tag{5.20}$$

where σ_{tot}=the total standard deviation Θ and
 α_t=the ratio of standard deviation due to degradation to the total standard deviation.
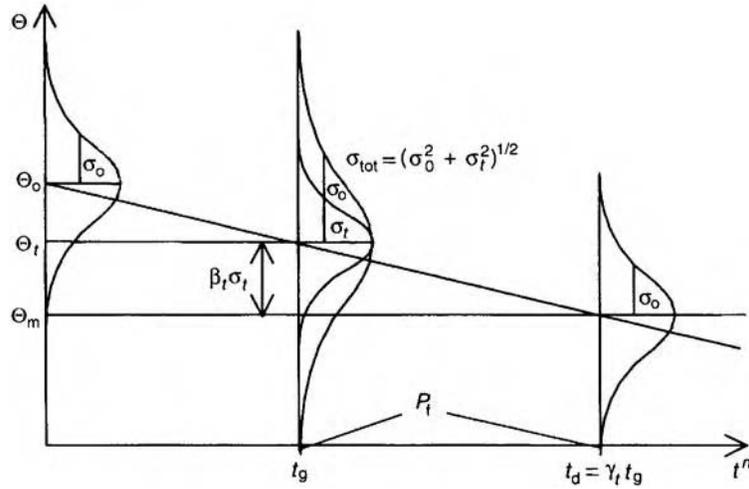
Figure 5.7 Decrease of the safety margin with time.

We also denote:

Θ_m=the minimum value of Θ corresponding to the safety index β_m (corresponds also to the 0-line in Figure 5.5).

β_t=the partial safety index with regard to degradation (safety index of durability failure).

In the following examination the lower index t refers to the time $t=t_g$. The mean of $\Delta\Theta$ is assumed to increase proportionally to the function t^n, where n is the exponent of time and σ_t is directly proportional to $\Delta\Theta$.

Figure 5.7 shows the diminishing with time of safety margin Θ The mechanical failure probability is described by that part of the distributions which has fallen below the limit Θ=0. The failure probability is influenced both by the decrease in the mean of Θ and the increase of standard deviation with time.

According to Figure 5.7 the safety index at $t=t_d$, when the time-related scatter is not taken into account, is

$$\beta_d = \frac{\Theta_m}{\sigma_o} \tag{5.21}$$

According to the principle mentioned above, the safety index β_d must be equal to the safety index β_g at $t=t_g$ when the timerelated scatter is also included. At $t=t_g$ the mean of Θ deviates from Θ_m by the amount $\beta_t\sigma_t$.

$$\beta_g = \frac{\Theta_m + \beta_t\sigma_t}{\sigma_{tot}} \tag{5.22}$$

By setting β_d equal to β_g and applying equation (5.20) we obtain

$$\frac{\Theta_m}{\sigma_o} = \frac{\Theta_m(1 - \alpha_t^2)^{1/2}}{\sigma_o} + \beta_t\alpha_t$$

Further applying the requirement $\beta_d=\Theta_m/\sigma_o=\beta_m$ gives

$$\beta_m = \beta_t \frac{\alpha_t}{1 - (1 - \alpha_t^2)^{1/2}} \tag{5.23}$$

Equation (5.23) describes how the relation of the total mechanical safety requirement (β_m) to the durability safety requirement (β_t) depends on the ratio α_t.

a_t can be determined as follows. According to Figure 5.7 we get for Θ_t:

$$\Theta_t = \Theta_o(1 - kt^n) \tag{5.24}$$

where k is the reduction coefficient of Θ. Thus we get

$$\Delta\Theta = \Theta_o kt^n \tag{5.25}$$

and

$$\sigma_t = \nu_D\Theta_o kt^n \tag{5.26}$$

where ν_D is the coefficient of variation of the decrease of Θ. σ_o can be expressed as

$$\sigma_o = \Theta_m/\beta_m \tag{5.27}$$

For k we can denote

$$k = \frac{\Theta_o - \Theta_m}{\Theta_o t_d^n} = \frac{m}{(\gamma_t t_g)^n} \tag{5.28}$$

where the quantity m is the relative reduction of Θ during the interval $0 \to t_d$:

$$m = \frac{\Theta_o - \Theta_m}{\Theta_o} = 1 - \frac{\Theta_m}{\Theta_o} \tag{5.29}$$

From equations (5.19) and (5.20) we can then derive for α_t at $t=t_g$:

$$\alpha_t = \frac{m\nu_D \Theta_o / \gamma_t^n}{[(\Theta_m/\beta_m)^2 + (m\nu_D\Theta_o/\gamma_t^n)^2]^{1/2}} \tag{5.30}$$

By dividing both the numerator and the denominator by Θ_m/β_m and taking into account that $\Theta_o/\beta_m = 1/(1-m)$ we get finally

$$\alpha_t = \frac{r_t}{(1 + r_t^2)^{1/2}} \tag{5.31}$$

where

$$r_t = \frac{\nu_D m \beta_m}{(1 - m)\gamma_t^n} \tag{5.32}$$

Let us then examine the safety index β_t. From Figure 5.7 we obtain

$$\beta_t \sigma_t = \Theta_t - \Theta_m$$

from which we can derive

$$\beta_t = \frac{\Theta_t - \Theta_m}{\nu_D(\Theta_o - \Theta_t)} = \frac{1}{\nu_D}\left(\frac{\Theta_o - \Theta_m}{\Theta_o - \Theta_t} - 1\right)$$

On the other hand we can see from Figure 5.7:

$$\frac{\Theta_o - \Theta_m}{\Theta_o - \Theta_t} = \frac{(\gamma_t t_g)^n}{t_g^n} = \gamma_t^n$$

Assigning this to the formula for β_t gives

$$\beta_t = \frac{(\gamma_t^n - 1)}{\nu_D} \tag{5.33}$$

The result is the same as equation (5.10).

By assigning equations (5.31) and (5.33) to equation (5.23) the values of γ_t can be determined as a function of ν_D, n and m. The corresponding values for β_t and α_t can then be obtained from equations (5.33) and (5.31).

(d)
Results and application

In Table 5.6 the values of γ_t, β_t and α_t have been determined for varying values of m. The safety index β_m is 3.8. For the other parameters we assume

Table 5.6 Values of γt, β_t and α_t; β_m=3.8

m	γt	β_t	α_t
0.001	1.00	0.00	0.00
0.1	1.23	0.39	0.20
0.2	1.44	0.73	0.37
0.3	1.63	1.05	0.51
0.4	1.83	1.38	0.64
0.5	2.02	1.71	0.75
0.6	2.23	2.06	0.84
0.7	2.46	2.43	0.91
0.8	2.70	2.84	0.96
0.9	2.97	3.29	0.99
0.999	3.28	3.79	1.00

$$\nu_D = 0.6$$
$$n = 1$$

The corresponding values for β_m=3.1 are given in Table 5.7 (other parameters as above).

Tables 5.8 and 5.9 apply to serviceability limit state design. Values of γ_t, β_t and a_t are given for βm=2.5 and 1.5. ν_D and n are the same as above.

The required value for β_t and γ_t depends on m (the relative decrease of Θ during the time interval $0 \rightarrow t_d$). In principle γt could be any value in Tables 5.6–5.9, if it is consistent with the real m. The real m depends on the amount of degradation during the service life and should be checked by calculations after the design. The real m must not be greater than that respective to γ_t which was applied in the design.

In practical design a standard assumption for m is 0.7 corresponding to the values of γ_t of 2.46 for βm=3.8 and 2.15 for βm=3.1 respectively. After design of the structure the real m is checked to ensure that it does not exceed 0.7.

Table 5.7 Values of γ_t, β_t and a_t·>β_m=3.1

m	γ_t	β_t	a_t
0.001	1.00	0.00	0.00
0.1	1.16	0.27	0.17
0.2	1.32	0.53	0.33
0.3	1.47	0.79	0.48
0.4	1.63	1.05	0.61
0.5	1.79	1.32	0.72
0.6	1.97	1.61	0.82
0.7	2.15	1.92	0.90
0.8	2.36	2.27	0.95
0.9	2.59	2.66	0.99
0.999	2.86	3.10	1.00

Table 5.8 Values of γ_t, β_t and a_t·β_m=2.5

m	γ_t	β_t	a_t
0.001	1.00	0.00	0.00
0.1	1.11	0.19	0.15
0.2	1.22	0.37	0.29
0.3	1.34	0.57	0.43
0.4	1.46	0.77	0.56
0.5	1.59	0.99	0.69
0.6	1.74	1.23	0.79
0.7	1.89	1.49	0.88
0.8	2.07	1.78	0.95
0.9	2.27	2.11	0.99
0.999	2.50	2.50	1.00

Table 5.9 Values of γ_t, β_t and a_t·β_m=1.5

m	γt	β_t	α_t
0.001	1.00	0.00	0.00
0.1	1.04	0.07	0.10
0.2	1.09	0.15	0.20
0.3	1.15	0.25	0.32
0.4	1.21	0.35	0.44
0.5	1.28	0.47	0.57
0.6	1.37	0.62	0.70

m	γt	β_t	α_t
0.7	1.47	0.78	0.82
0.8	1.59	0.98	0.92
0.9	1.73	1.21	0.98
0.999	1.90	1.50	1.00

The value 0.7 is based on the assumption that the safety margin decreases roughly linearly in this range (corresponding approximately to 0.5 in the relative reduction of load-bearing capacity R, cf. section 6.1.2). During the time interval $0 \rightarrow t_g$ the relative decrease of Θ is of course much less. Also the difference in capacities of an ordinarily designed and a durability designed structure would be too great if m were greater than 0.7.

If m turns out to be greater than 0.7 the designer should check whether there is any way to reduce it by changing the dimensions or material specifications or by reducing the rates of degradation. If not, the required target service life may not be realistic in the environment concerned and should be shortened. Then the designer should propose a new requirement for the target service life.

From Tables 5.6–5.9 it is seen that the values of β_t and γ_t respective to the maximum value of m ($m=1$) are equal to the requirements of durability safety in section 5.3.2. From this one can conclude that when applying the separated safety principle with the requirements presented in Table 5.5 no check for m is required after the design.

5.3.4
Partial safety factors of loads and materials in durability design

If β_m is 3.8 the partial safety factors defined by Eur ocode 1 (CEN, 1994) and CEB/FIP model code (CEB, 1988) are used in durability design.

Loads:
$$\gamma_g = 1.35 \quad \text{(dead loads)}$$
$$\gamma_p = 1.5 \quad \text{(variable loads)}$$

Materials:
$$\gamma_c = 1.5 \quad \text{(concrete)}$$
$$\gamma_s = 1.15 \quad \text{(steel)}$$

If β_m is 3.1 the following partial safety factors for loads and materials are used:

Loads:
$$\gamma_g = 1.30 \quad \text{(dead loads)}$$
$$\gamma_p = 1.38 \quad \text{(variable loads)}$$

Materials:
$$\gamma_c = 1.40 \quad \text{(concrete)}$$
$$\gamma_s = 1.13 \quad \text{(steel)}$$

The partial safety factors for $\beta_m=3.1$ have been determined according to Eurocode 1, Annex A (CEN, 1994) using equations (5.34)–(5.36). For dead loads the normal distribution and for variable loads the Gumbel distribution has been assumed. The safety factors of materials have been determined assuming lognormal distribution.

Normal distribution:
$$\gamma = (1 + \alpha\beta V_x)\xi \tag{5.34}$$

Gumbel distribution:
$$\gamma - \left[1 + V_x(0.577 + \ln\{-\ln\phi(\alpha\beta)\})\frac{\sqrt{6}}{\pi}\right]\xi \tag{5.35}$$

Log-normal distribution:
$$\gamma = \exp(\alpha\beta V_x) \tag{5.36}$$

In equations (5.34)–(5.36)
   a=the sensitivity coefficient,
   β=safety index,

V_x=coefficient of variation,
ξ=extra safety coefficient,
$\o$=the distribution function normal distribution.
The following values have been used in the calculations:

$$\xi = 1.05$$

$$\alpha = 0.7 \text{ for loads}$$

$$\alpha = 0.8 \text{ for material strengths}$$

V_x has been determined backwards by applying β=3.8 and the corresponding values of γ. The new values of γ have then been determined by applying the calculated values for V_x and β=3.1.

If β_m is smaller than 3.8 (the principal safety requirement according to Eurocode 1) it is necessary to check that the safety margin Θ_o and the bearing capacity R_o are not smaller than those obtained by normal design. If they are, the dimensions obtained by normal design are determinative. This check is to ensure that the final safety of the structure is never less than that of a normally designed structure.

6
Structural durability design

6.1
FORMULATION OF LOAD-BEARING CAPACITY WITH TIME

6.1.1
Principles

The following degradation factors may have long-term effects on the load-bearing capacity of concrete structures:

1. corrosion due to chloride penetration
2. corrosion due to carbonation
3. mechanical abrasion
4. salt weathering
5. surface deterioration
6. frost attack.

Additionally there exist some internal degradation processes, such as alkaline-aggregate reaction, that are not primarily caused by environmental stresses. Such degradation problems are not treated here as they can be solved by a proper selection of raw materials and an appropriate design of concrete mix.

Degradation factors affect either the concrete or the steel or both. Usually degradation takes place on the surface zone of concrete or steel, gradually destroying the material. The main structural effects of degradation in concrete and steel are the following (Andrade *et al.,* 1989):

1. loss of concrete leading to reduced cross-sectional area of the concrete;
2. corrosion of reinforcement leading to reduced cross-sectional area of steel bars. Corrosion may occur:
    (a) at cracks;
    (b) at all steel surfaces, assuming that the corrosion products are able to leach out through the pores of the concrete (general corrosion in wet conditions);
3. splitting and spalling of the concrete cover due to general corrosion of reinforcement, leading to a reduced crosssectional area of the concrete and a reduced bond between concrete and reinforcement.

The capacity of structures can in principle be studied at three different levels (Andrade *et al.,* 1989). At the first level only the cross-section of a structure is studied against different action effects such as bending moment, shear force, axial force, etc. The second level introduces a deterioration model of isolated structural elements such as simply supported beams or columns, taking into account deformations, sliding and buckling of reinforcement, etc. Finally, at the third level the whole structure is considered, taking into account the possible redistribution of action effects provided this is allowed by the remaining ductility of materials.

In the following only a first-level study is introduced. The load-bearing capacity of structures is studied by the design formulae for cross-sections of structures. The design formulae are supplied with time-dependent degradation models of concrete and steel. This is demonstrated below using several examples.
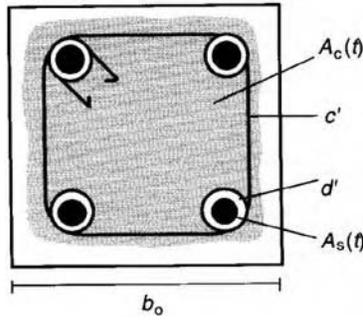
Figure 6.1 Cross-section of a column.

6.1.2
Examples of the calculation of degradation processes

The purpose is to show how the degradation of materials is taken into account in the formulation of load-bearing capacity, and to provide insight into the way in which the load-bearing capacity is reduced with time.

Cross-sections of a simple axially loaded column and a bent beam are studied. Degradation in both concrete and steel is assumed, leading to a reduction in the concrete and in steel cross-sections. No splitting or spalling around the corroded steel bars is assumed.

(a)
Column

The load-bearing capacity of an axially loaded square concrete cross-section reinforced with rebars at its corners is (Figure 6.1)

$$R_d = A_c(t)\frac{f_{ck}}{\gamma_c} + A_s(t)\frac{f_y}{\gamma_s} \tag{6.1}$$

where
   R_d=the compressive capacity of the cross-section,
   A_c=the cross-sectional area of the concrete,
   A_s=the cross-sectional area of the steel,
   f_{ck}=the characteristic compressive strength of concrete,
   f_y=the nominal yield strength of steel,
   γ_c=the partial safety factor of concrete, and
   γ_s=the partial safety factor of steel.
   The cross-sectional areas of concrete and steel are

$$A_c(t) = (b_o - 2c'(t))^2 \tag{6.2}$$

$$A_s(t) = \frac{4\pi(D_o - 2d'(t))^2}{4} \tag{6.3}$$

where
   b_o=the original width of the column,
   D_o=the original diameter of steel bars,
   c'=a degradation model of concrete expressing the depth of deterioration of concrete, and
   d'=a degradation model of steel expressing the depth of corrosion in reinforcement.
   If c' is small compared to b_o and d' is small compared to D_o, the following approximation of equation (6.1) can be written:

$$R_d \approx R_{do} - 4\left[b_o c'(t)\frac{f_{ck}}{\gamma_c}\right] + D_o d'(t)\frac{f_y}{\gamma_s} \tag{6.4}$$

Equation (6.4) shows that at least at the start of service life (when c' and d' are relatively small), the degradation of Rd follows the type of degradation in concrete and steel. If c' and d' are linear with respect to time, the capacity of the structure will also decrease linearly with time. If c' and d' are accelerating or retarding, the reduction in R_d will show the same tendency.

Figure 6.2 shows the reduction in A_c, A_s and R_d expressed in per cent (according to equations (6.1)–(6.3)) for a typical axially loaded column, when c' and d' are assumed to be linear with time. The calculations have been done for the following dimensions and material specifications:
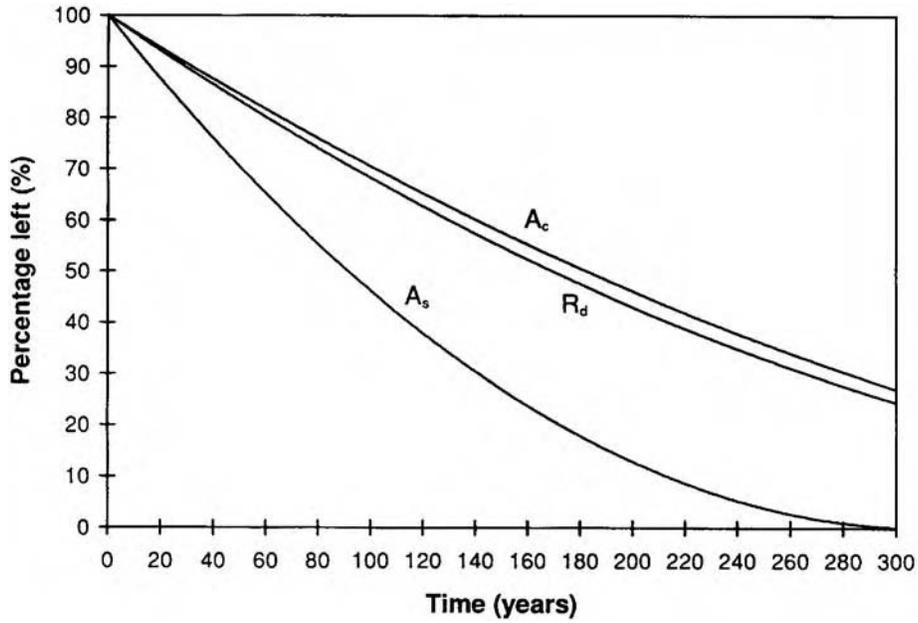
Figure 6.2 Reductions in material cross-sections and compressive capacity of a column.
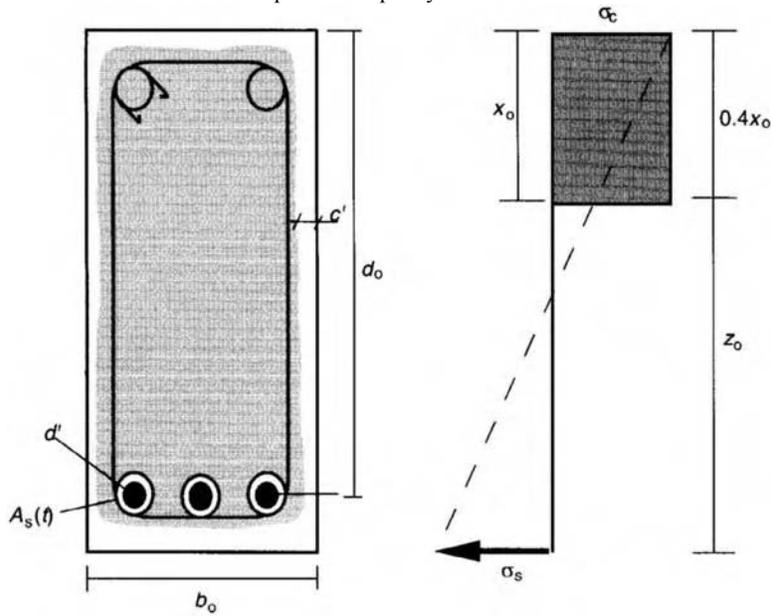
Figure 6.3 Cross-section of a beam.

$$b_o = 500 \text{ mm}$$
$$D_o = 25 \text{ mm}$$
$$f_{ck} = 40 \text{ MPa}$$
$$f_{yk} = 400 \text{ MPa}$$
$$\gamma_c = 1.5$$
$$\gamma_s = 1.15$$
$$c'(t) = 0.4 \text{ mm/year}$$
$$d'(t) = 0.04 \text{ mm/year}$$

The degradation of R_d seems to be roughly linear with time and closely follows the reduction of A_c. The corrosion of steel has only a small effect on the total capacity of the column. Even if the entire steel area were to disappear, the beam would retain some load-bearing capacity.

(b)
Beam

The load-bearing capacity of a beam is determined by the following formulae (Figure 6.3):

$$R_{ds} = A_s(t)z(t)\frac{f_y}{\gamma_s} \quad \text{(the stress of steel is decisive)} \tag{6.5}$$

$$R_{dc} = b(t)x(t)z(t)\frac{f_c}{2\gamma_c} \quad \text{(the stress of concrete is decisive)} \tag{6.6}$$

$$x(t) = d(t)\mu(t)n\left[-1 + \left(1 + \frac{2}{(\mu(t)n)}\right)^{1/2}\right] \tag{6.7}$$

$$z(t) = d(t) - \frac{0.8x(t)}{2} \tag{6.8}$$

$$n = \frac{E_s}{E_c} \tag{6.9}$$

$$\mu(t) = \frac{A_s(t)}{b(t)d(t)} = \frac{N_s\pi(D_o - d'(t))^2/4}{(b_o - 2c'(t))(d_o - c'(t))} \tag{6.10}$$

where R_{ds}=the bending capacity of the beam when the tensile stress of steel is decisive,
   R_{dc}=the bending capacity of the beam when the compressive strength of concrete is decisive,
   A_s=the cross-sectional area of the steel,
   d=the effective height of the beam,
   x=the distance of the neutral axis from the top surface of the structure,
   z=the internal lever arm of the moment,
   E_s=the modulus of elasticity of steel,
   E_c=the modulus of elasticity of concrete,
   μ=the geometric area of steel,
   c'= a degradation model expressing the depth of deterioration in concrete,
   d'= a degradation model expressing the depth of corrosion in reinforcement and
   N_s=the number of steel bars.

   The reduction in A_s, z, R_{ds} and R_{dc} expressed in per cent for a typical beam with constant degradation rate c' and d' is shown in Figure 6.4. The following dimensions and material specifications have been used in the calculations:

$$b_o = 400 \text{ mm}$$
$$d_o = 700 \text{ mm}$$
$$D_o = 25 \text{ mm}$$
$$N_s = 3$$
$$f_c = 40 \text{ MPa}$$
$$f_y = 400 \text{ MPa}$$
$$E_s = 200\,000 \text{ N/mm}^2$$
$$E_c = 9500(f_{ck} + 8)^{1/3} \text{ N/mm}^2$$
$$\gamma_c = 1.5$$
$$\gamma_s = 1.15$$

The reduction in R_{ds} seems to follow closely the reduction in cross-sectional area of the steel. This is because the reduction of z with time seems very small and is relatively smaller than that of d, as d is diminishing as well.
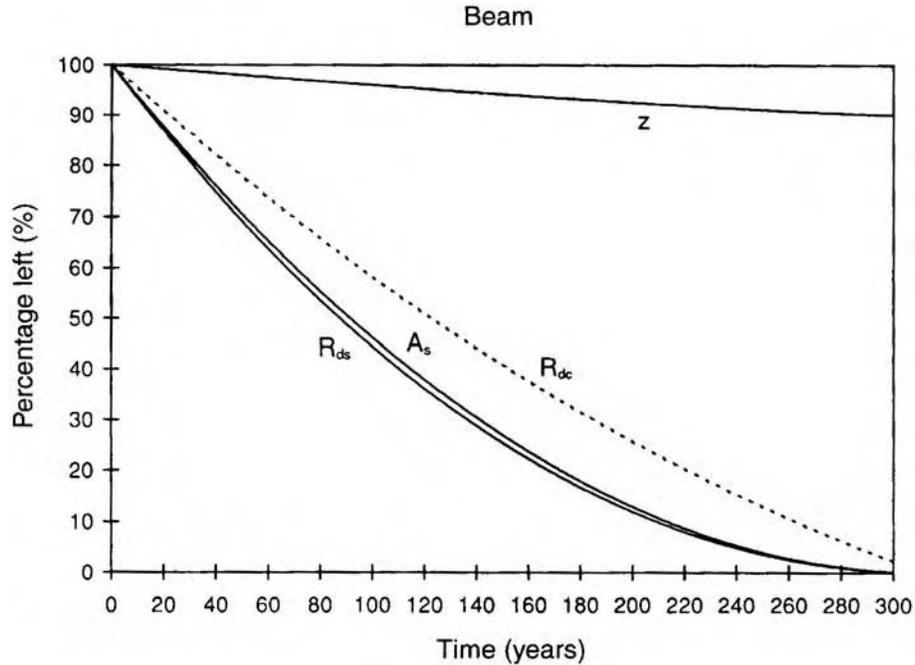
Beam

Figure 6.4 Reduction in the bending capacity of a beam.

The absolute value of R_{ds} is smaller than that of R_{dc}. Thus R_{ds} is decisive in the present study. However, the R_{dc} curve is also presented in Figure 6.4, showing an almost linear reduction tendency.

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6.2

PROPOSED PROCEDURES FOR STRUCTURAL DURABILITY DESIGN

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The proposed design procedure includes the following phases:

1. ordinary mechanical design
2. durability design
3. final design.

Two methods are proposed for phase 3 (final design):

1. separated design method comprising a simple integration of the results of phases 1 and 2;
2. combined design method where the mechanical design is newly performed as $R_d(t_d) > S_d(t_d)$, applying the results of phase 2. In this case phase 1 serves mainly as a reference but may be determinative in some cases.

A flow chart of the design procedure is presented in Figure 6.5.

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6.2.1

Ordinary mechanical design

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Ordinary mechanical design is performed using conventional design methods. Its purpose is to determine the preliminary dimensions for the structure.

Table 6.1 Classification of the target service life (design working life) (source: CEN, *ENV 1991–1. Euro 1,* published by CEN, 1994)

Class	Target service life (years)	Example
1	[1–5]	Temporary structures
2	[25]	Replaceable structural parts, e.g. gantry girders, bearings
3	[50]	Building structures and other common structures
4	[100]	Monumental building structures, bridges and other civil engineering structures
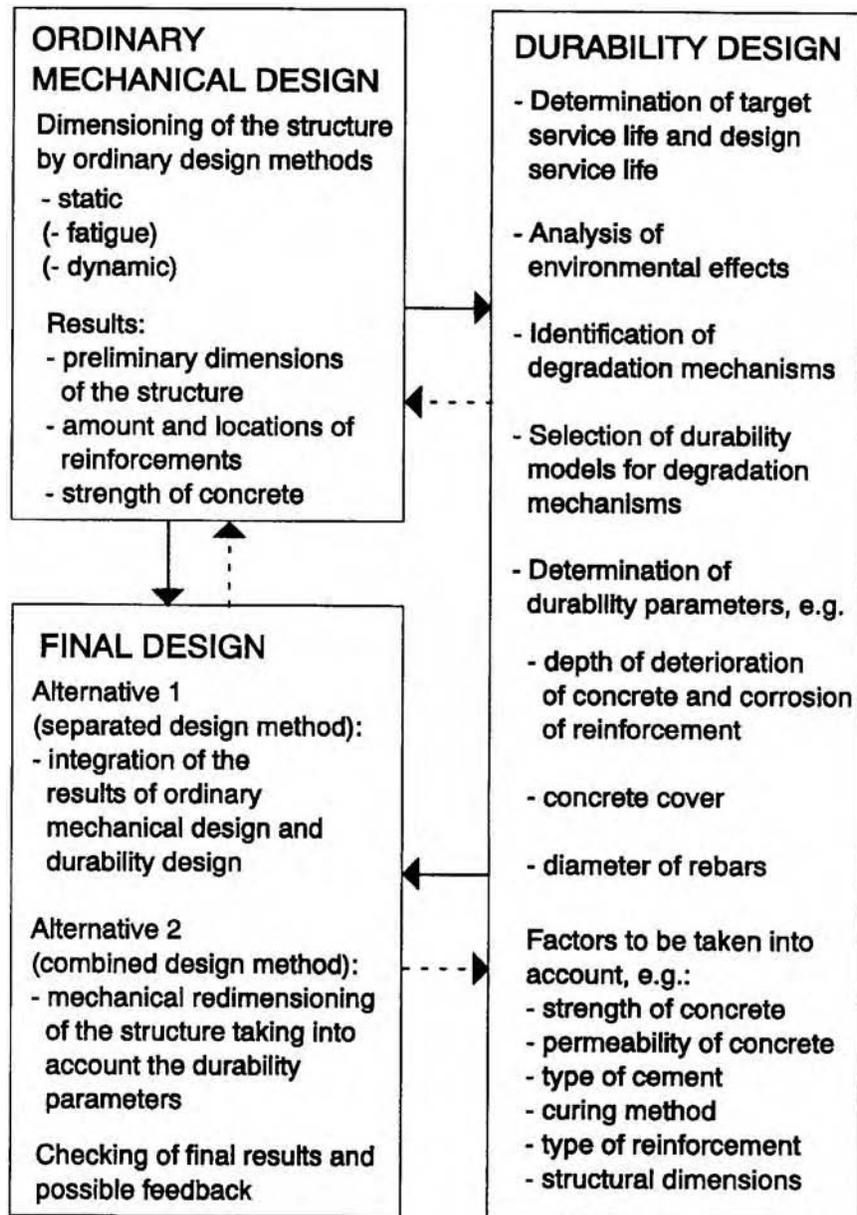
Figure 6.5 Flow chart of the durability design procedure.

6.2.2
Durability design

(a)
Durability design procedure

The aim of durability design is to improve the control of durability over the required service life. The durability design procedure is the following:

1. specification of the target service life and design service life;
2. analysis of environmental effects;
3. identification of durability factors and degradation mechanisms;
4. selection of a durability calculation model for each degradation mechanism;
5. calculation of durability parameters using available calculation models;
6. possible updating of the calculations of the ordinary mechanical design (e.g. own weight of structures);

7. transfer of the durability parameters into the final design.

(b)
Specification of target service life and design service life

The target service life is defined corresponding to the requirements given in common regulations, codes and standards in addition to possible special requirements of the client.

Typical classes of service life are 10, 25, 50, 75, 100, etc. years. Eurocode 1 (CEN, 1994) presents the classification for the target service life given in Table 6.1 (Eurocode 1 uses here the term 'design working life' which can, however, be interpreted as the minimum requirement for service life). Another classification is presented in the standard BS 7543 (BSI, 1992).

The proposed safety classification of durability design is presented in Table 6.2. The required lifetime, load and material safety factors are also given in the table.

The design service life is determined by equation (6.11):

$$t_d = \gamma_t t_g \tag{6.11}$$

where
t_d=the design service life,
γ_t=the lifetime safety factor, and
t_g=the target service life.

(c)
Analysis of environmental effects

The analysis of environmental effects includes identification of the climatic conditions such as temperature and moisture variations, rain, condensation of moisture, freezing, solar radiation and aerial pollution, and the identification of geological conditions such as the location of ground water, possible contact with sea water, contamination of the soil by aggressive agents like sulphates and chlorides. Man-made actions such as salting of roads, abrasion by traffic, etc. must also be identified.

(d)
Identification of degradation factors and degradation mechanisms

Based on the environmental effect analysis the designer identifies the degradation factors to which the structure will most likely be subjected. The list of degradation factors in Table 7.1 may be used at this stage.

If the structures are not fully protected from the environment, some kind of degradation process is usually assumed to take place in both the concrete and the reinforcement.

(e)
Selection of durability models for each degradation mechanism

A designer must determine which degradation factors are decisive for service life. Preliminary evaluations of rates of degradation for different factors may be necessary. The models presented in Chapter 8 may be applied in these evaluations. The same design procedure and principles can be applied for all types of deterioration.

In concrete structures exposed to normal outdoor conditions the effects of degradation mechanisms can be classified into the following structural deterioration mechanisms.

1. Corrosion of reinforcement at cracks in the concrete, causing a reduction in the cross-sectional area of steel bars. The models and corrosion rate expectations presented in section 8.5 can be used in the calculations.
2. Surface deterioration or frost attack, causing a reduction in the cross-sectional area of concrete. The models presented in section 8.3 for surface deterioration or section 8.2 for frost attack can be used in the calculations.

Service life models for general corrosion presented in section 8.5 are used in the evaluation of the cracking time of concrete covers.

(f)
Calculation of durability parameters through calculation models

The depth of corrosion is determined using the design service life, td/ as time. The diameter of hoops (stirrups) and other possible non-load-bearing rebars is then designed. The minimum initial diameter of rebars, D_{omin}, is twice the depth of corrosion added to the minimum final (after the service life) diameter of the rebar. The minimum final diameter is specified by the designer ($\geq$0). The initial diameter must then be chosen to be consistent with the standards ($D_o \geq D_{omin}$).

The depth of deterioration of concrete is evaluated using the

Table 6.2 Safety classification of durability design and the corresponding safety factors

Limit state	Safety class of durability design	Separated design		Combined design[a]	
		Lifetime safety factor γ_t[b]	Load and material safety factors	Lifetime safety factor γ_t[c]	Load and material safety factors
Ultimate limit state	1. Serious social, economic or ecological consequences of a mechanical failure	3.3	Normal[d]	2.5	Normal[d]
					γ_g=1.3[e]
					γ_P=1.38
	2. Consequences of a mechanical failure are not serious	2.9	Normal[d]	2.2	γ_c=1.4
				γ_s=1.13	
Serviceability limit state	1. Noticeable consequences and considerable repair costs	2.5	–	1.9	–
	2. Non-noticeable consequences and repair costs	1.9	–	1.5	–

[a] Check for m (relative reduction of the safety margin during 0→t_d) ≤0.7 is required.
[b] Cf. Table 5.5.
[c] Cf. Tables 5.6–5.9.
[d] Load and material safety factors specified for ordinary mechanical design are used.
[e] Reduced values of load and material safety factors may be used in the durability design (cf. section 5.3.4). However, the safety at the start of service life t=0) must be at least the same as that required in ordinary design.

design service life, t_d, as time. The minimum thickness of the concrete cover, C_{omin}, is dimensioned by adding the depth of deterioration of concrete to the minimum final cover thickness (after the service life). The minimum final thickness of concrete cover is specified by the designer ($\geq$0). The initial thickness of concrete cover, C_o, is then selected by possibly rounding off upwards taking into account the requirements of the codes.

In addition, a check for general corrosion must be performed. The purpose is to ensure that no cracking or spalling of the concrete cover that would violate the reinforcement bond can take place during the service life. This can be done by applying the service life models for general corrosion with the earlier dimensioned concrete cover and diameter of steel bars as parameters. The determinative rebars with respect to cracking and spalling are normally the hoops.

(g)
Possible updating of calculations in ordinary mechanical design

Some durability parameters may influence the mechanical design. Such an interactive influence would be an increase in concrete dimensions, which increases the dead load of horizontal structures, thus increasing the load effects on both the horizontal and vertical structures. This phase is not necessary when the combined method is used.

(h)
Transfer of durability parameters to the final design

The parameters of the durability design are listed and transferred to the final design phase for use in the final dimensioning of the structure.

6.2.3
Final design

(a)
Separated design method

In the separated design method the mechanical design and the durability design are separated. The ordinary structural design (phase 1) produces the mechanical safety and serviceability parameters whereas the durability design (phase 2) produces the durability parameters. Both of these groups of parameters are then combined in the final design of the structure.

The possible interaction between mechanical design parameters and durability design parameters is not normally taken into account. In some cases there may exist some interaction and need for updating the mechanical design parameters as discussed in section 6.2.2(g). A fairly common case might be an increase in strength of concrete due to durability, which again influences the load-bearing capacity and reduces the dimensions especially in the case of compressed structures.

The dimensions of ordinary design are assumed to be those existing at the end of service life. Thus these dimensions must be increased by the amounts corresponding to the depth of deterioration and corrosion during service life.

The depth of deterioration of concrete is added to the structural dimensions obtained in ordinary design. If two opposite sides of the structure are exposed to degradation the dimension in ordinary design is increased by twice the depth of deterioration. The final dimension may then be rounded off upwards.

The diameter of rebars in the ordinary design of the structure is increased by twice the depth of corrosion. The final diameter must then be rounded off upwards to be consistent with standards.

No check for m (relative reduction of the safety margin during $0 \rightarrow t_d$) is required.

(b)
Combined design method

In the combined design method the mechanical design is newly performed, taking into account the results of the durability design and the required safety at the end of service life.

The combined method is especially suited to degradation mechanisms which directly affect the bearing capacity or the mechanical serviceability of structures. The method also, in some cases, allows use of smaller lifetime safety factors than in the separated method, as the value of m is checked after the design. In safety class 2 (ultimate state) the required safety index after service life may be slightly smaller than in ordinary design.

The structures are provided with dimensions and material specifications relevant to fulfilling the following condition:

$$R_d(t_d) - S_d(t_d) \geq 0 \tag{6.12}$$

where

$R_d(t_d)$=the design capacity of the structure at the end of the design service life, and

$S_d(t_d)$=the design load of the structure at the end of the design service life.

The values for material and load safety factors depend on the safety class. In safety class 1 (β=3.8) the material and load safety factors are the same as in ordinary design. In safety class 2 (β=3.1) the following safety factors are used:

$$\gamma_g = 1.3$$
$$\gamma_p = 1.38$$
$$\gamma_c = 1.4$$
$$\gamma_s = 1.13$$

The final dimensions are readily obtained using spreadsheet applications with 'Goal seek' or 'Solver' tools.

After solution of the dimensions and material specifications for a structure the following control measures are performed. The condition for m (relative reduction of the safety margin during $0 \rightarrow t_d$ is

$$m = \frac{\Theta_o - \Theta_m}{\Theta_o} \leq 0.7 \tag{6.13}$$

In equation (6.13) Θ is the safety margin $(=R—S)$ determined using characteristic values of loads and material properties. The index o refers to the initial state of the structure and m to the final state after the design service life t_d. Θm is determined from the final mechanical design solution by setting the load and material safety factors equal to 1. Θ_o is obtaine d by also setting γ_t equal to 0. The calculations are easy when using a spreadsheet application (cf. examples in Section 6.3).

If m is greater than 0.7 the required safety is probably not fulfilled, as the lifetime safety factor used in the calculations is too small for the real m value. Also the durability designed structure is probably too robust compared with the respective ordinarily designed structure.

To keep m smaller than required it may be necessary to return to phase 2 to make some modifications in the durability design. The designer may have to consider whether there are any means to reduce the rate of degradation or otherwise to change the design assumptions. If such is not possible, expectations of the service life of the structure may not be realistic in the intended environment and a reduced target service life may have to be considered.

If reduced values for material and load safety factors are used in the final design, a check must be performed that safety margin Θ_o and the bearing capacity R_o (at the start of service life) are not smaller than those obtained by ordinary mechanical design. If they are, the dimensions obtained by ordinary mechanical design are determinative. The final safety of the structure must never be less than that of an ordinarily designed structure.

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6.3

EXAMPLES OF THE DESIGN PROCEDURE

6.3.2
Column

(a)
Setting up the design problem

</div>

The column is to be dimensioned for the following loads:
$$F_\mathrm{g} = 1000 \ \mathrm{kN} \quad \text{(dead load)}$$
$$F_\mathrm{p} = 3000 \ \mathrm{kN} \quad \text{(variable load)}$$

In a square cross-section (side length b) there are four steel bars ($\o=D$, one in each corner. The yield strength of steel is 400 MPa ($=f_\mathrm{y}$).

The column is supposed to be maintenance free so that corrosion of steel bars in the assumed cracks or deterioration of the concrete cover do not hinder use of the column during its service life. The hoops (stirrups) must not be completely broken at cracks after the service life. The concrete cover must be at least 20 mm after the service life with no spalling due to general corrosion of hoops.

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(b)
Ordinary mechanical design

</div>

The ordinary design of the column is performed using the following formulae:
$$R_\mathrm{d} \geq S_\mathrm{d} \tag{6.14}$$
where
$$S_\mathrm{d} = \gamma_\mathrm{g} F_\mathrm{g} + \gamma_\mathrm{p} F_\mathrm{p} \tag{6.15}$$
and
$$R_\mathrm{d} = \frac{A_\mathrm{c} f_\mathrm{c}}{\gamma_\mathrm{c}} + \frac{A_\mathrm{s} f_\mathrm{y}}{\gamma_\mathrm{s}} \tag{6.16}$$

The cross-sectional areas of concrete and steel, A_c and A_s, are
$$A_\mathrm{c} = b^2 \tag{6.17}$$

$$\tag{6.18} \qquad\qquad\qquad\qquad A_\mathrm{s} = \frac{4\pi D^2}{4}$$

Taking $D=15$ mm we obtain from equation (6.18)
$$A_\mathrm{s} = 707 \ \mathrm{mm}^2$$

Column						
	Ordinary mechanical design				Durability and	
		Design values	Charact. values			final design
	γ_a	1.35	1		γ_t	3.3
	γ_p	1.5	1		t_g	50
	γ_c	1.5	1		t_d	165
	γ_s	1.15	1		C_{min}	20
	F_g	1350000	1000000		$\delta L/\delta t$	0.135
	F_p	4500000	3000000		$b'(t_d)$	22.3
	f_c	27	40		C_{omin}	42
	f_y	348	400		C_o(chosen)	45
	D	15.0			b_{omin}	503
	A_s	707			b_o(chosen)	503
	b	458			D_{min}	15
	A_c	210155			$\delta r/\delta t$	0.03
	S	5850000			$d'(t_d)$	5.0
	R	5850000			D_{omin}	24.9
	R-S	-1.9E-09			D_o(chosen)	25
					Check of the cracking of concrete cover	
					C	45
					D_{hmin}	10
					D_h(chosen)	10
					C_h	35
					C_{env}	1
					C_{al}	0.7
					f_{ck}	40
					a	1800
					b	-1.7
					r	12
					t_o	401
					t_l	30
					t_l	431
					t_d	165

Figure 6.6 Spreadsheet design of a column. Separated design method.

As a result of the condition (6.14) we get

$$A_c = 210\ 155\ \text{mm}^2$$

From equation (6.17) we get

$$b = 458\ \text{mm}$$

These are the preliminary dimensions for steel bars and concrete.

The calculations are also presented in Figure 6.6, which shows an excerpt from an Excel spreadsheet. The solution has been calculated by Solver by setting $R-S$ equal to 0 and changing the value of b.

(c)
Durability design

The target service life of a column is 50 years. The lifetime safety factor is 3.3 for the separated design method and 2.5 for the combined design service life. Thus the design service life is

$$t_d = 3.3 \times 50 = 165\ \text{years} \quad \text{(for the separated method)}$$
$$t_d = 2.5 \times 50 = 125\ \text{years} \quad \text{(for the combined method)}$$

The column is assumed to be partly immersed in river water that freezes in winter. Thus the depth of deterioration in concrete, c' is evaluated using the following formula (cf. section 8.2):

$$c' = c_{env}c_{cur}c_{age}a^{-0.7}(f_{ck}+8)^{-1.4}t \tag{6.19}$$

The following values are chosen for the parameters in this formula:

$$c_{env} \text{ (environmental factor)} = 80$$
$$c_{cur} \text{ (curing factor)} = 1$$
$$c_{age} \text{ (ageing factor)} = 1$$
$$a \text{ (air content in \%)} = 4$$
$$f_{ck} \text{ (characteristic compressive strength MPa)} = 40$$

By inserting these values into equation (6.19) we get for the depth of deterioration of concrete

$$c' = 0.135t \text{ (mm)} \tag{6.20}$$

The depth of corrosion in steel bars at cracks is evaluated as follows (cf. section 8.5):

$$d' = 0.03t \text{ (mm)} \tag{6.21}$$

Depending on the design service life we get the following durability design parameters.

Separated design method

(t_d=165 years)
   The depth of deterioration of concrete is

$$c' = 0.135 \times 165 = 22.3 \text{ mm}$$

The minimum thickness of concrete cover is

$$C_{min} = 20 + 22.3 = 43 \text{ mm}$$

We choose C=45 mm.
   The depth of corrosion of steel bars at cracks is

$$d' = 0.03 \times 165 = 5.0 \text{ mm}$$

The hoops must not be broken due to corrosion. Thus the minimum diameter of the hoops is

$$D_{hmin} = 2 \times 5.0 \text{ mm} = 10 \text{ mm}$$

We choose D_h=10 mm.

Combined design method

(t_d=125 years)

$$c' = 0.135 \times 125 = 16.9 \text{ mm}$$

$$C_{min} = 20 + 16.9 = 37 \text{ mm}$$

We choose C=40 mm.

$$d' = 0.03 \times 125 = 3.8 \text{ mm}$$

$$D_{hmin} = 2 \times 3.8 \text{ mm} = 8 \text{ mm}$$

We choose D_h=10 mm.
   A check for general corrosion with the chosen concrete cover and diameter of steel bars must still be performed. The chloride content of water is assumed to be so low that corrosion cannot be induced by chlorides. However, steel bars in the upper end of the columns may be corroded due to carbonation.
   The determinative reinforcement with respect to general corrosion is the hoops. Thus equations (8.20) and (8.29) (cf. section 8.5) are applied to hoops (the diameter of the hoops is subtracted from the concrete cover of the main reinforcement):

$$\mu(t_L) = \frac{C_h^2}{\left(c_{env}c_{air}a(f_{ck}+8)^b\right)^2} + \frac{80C_h}{D_h r} \tag{6.22}$$

where
   C_h=the concrete cover with respect to hoops (=$C-D_h$),
   D_h= the diameter of hoops,
   C_{env}=the environmental coefficient (=1),

C_{air}=the coefficient of air content (=0.7), and

r=the rate of corrosion of steel before cracking (~12 μm/year).

As a result we get $\mu(t_L)$=431 years for the separated design method, which is longer than the design service life (165 years). For the combined design method we get 322 years which is also longer than the design service life (125 years). Therefore the chosen concrete covers are acceptable.

<div align="center">

(d)
Final design

</div>

<div align="center">

Separated design method

</div>

The width of the column at the start of service life is determined by adding twice the depth of deterioration of concrete to the width obtained in the ordinary design (phase 1):

$$b_o = b + 2c' = 458 \text{ mm} + 2 \times 22.3 \text{ mm} = 503 \text{ mm}$$

The diameter of steel bars of the main reinforcement is determined in the same way:

$$D_{omin} = 15 \text{ mm} + 2 \times 5.0 \text{ mm} = 25.0 \text{ mm}$$

As the diameter matches the standard we choose 25 mm. The calculations are presented in Figure 6.6.

Note: The required thickness of the concrete cover (C_{min}) and the width of the column (b_o) could be reduced by increasing the compressive strength of the concrete. For f_{ck}=50 MPa the minimum thickness of concrete cover would be 37 mm and the width of the column 444 mm.

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Combined design method

</div>

When the combined method is used, dimensioning of the column is performed by applying the ordinary design formulae at $t=t_d$:

$$R_d(t_d) \geq S_d \tag{6.23}$$

where

$$S_d = \gamma_g F_g + \gamma_p F_p \tag{6.24}$$

and

$$R_d(t_d) = A_c(t_d)\frac{f_c}{\gamma_c} + A_s(t_d)\frac{f_y}{\gamma_s} \tag{6.25}$$

The cross-sectional areas of concrete and steel, A_c and A_s, are

$$A_c(t_d) = (b_o - 2c'(t_d))^2 \tag{6.26}$$

$$A_s(t_d) = \frac{4\pi(D_o - 2d'(t_d))^2}{4} \tag{6.27}$$

For D_o=25 mm we get from equation (6.27)

$$A_s(t_d) = 962 \text{ mm}^2$$

As a result of the condition from equation (6.23) we get

$$A_c(t_d) = 206\,826 \text{ mm}^2$$

From equation (6.26) we obtain

$$(b_o - 2 \times 16.9)^2 = 206\,826 \text{ mm}^2$$

which yields

$$b_o = 489 \text{ mm}$$

The calculations are presented in Figure 6.7. The solution has been sought by Solver by setting $R-S$ equal to 0 and changing the value of b_o. In the next column the safety margin $\Theta_m=R_k-S_k$ corresponding to the solution obtained has been determined by inserting $\gamma_g=\gamma_p=\gamma_c=\gamma_s=1$;

$$\Theta_m = 4658 \text{ kN}$$

By also inserting γ_t=0 we get Θ_o representing the safety margin at the start of service life (t=0):

Column

Ordinary mechanical design			Durability and final design			
	Design values $t=0$	Charact. values $t=0$		Design values $t=t_d$	Charact. values $t=t_d$	Charact. values $t=0$
γ_g	1.35	1	γ_g	1.35	1	1
γ_p	1.5	1	γ_p	1.5	1	1
γ_c	1.5	1	γ_c	1.5	1	1
γ_v	1.15	1	γ_v	1.15	1	1
F_g	1350000	1000000	γ_c	2.5	2.5	0
F_p	4500000	3000000	F_g	1350000	1000000	1000000
f_c	27	40	F_p	4500000	3000000	3000000
f_v	348	400	f_c	27	40	40
D	15.0	15.0	f_v	348	400	400
A_s	707	707	C_{min}	20	20	20
b	458	458	D_{min}	15	15	15
A_c	210155	210155	t_o	50	50	50
S	5850000	4000000	t_d	125	125	0
R	5850000	8688948	$\delta L/\delta t$	0.135	0.135	0.135
$R\text{-}S$	0.1322364	4688948	$b'(t_d)$	16.9	16.9	0.0
			C_{emin}	37	37	20
			C_e(chosen)	40	40	40
Check of the cracking			$\delta r/\delta t$	0.03	0.03	0.03
of concrete cover			$d'(t_d)$	3.8	3.8	0.0
C	40		D_{emin}	22.5	22.5	15.0
D_{hmin}	8		D_e(chosen)	25	25	25
D_h(chosen)	10		$D(t_d)$	17.5	17.5	25.0
C_h	30		$A(t_d)$	962	962	1963
C_{env}	1		b_o	489	489	489
C_{gr}	0.7		$A_c(t_d)$	206826	206826	238662
f_{ck}	40		S	5850000	4000000	4000000
a	1800		R	5850000	8657873	10331897
b	-1.7		$R\text{-}S$	-0.043762	4657873	6331897
r	12					
t_o	295		**Check of structural degradation**			
t_1	27		Rel. reduction of R	0.162		
t_L	322		Rel. reduction of R-S	0.264		
t_d	125					

Figure 6.7 Spreadsheet design of a column. Combined design method.

$$\Theta_o = 6332 \text{ kN}$$

The relative reduction of $R\text{–}S(=m)$ is

$$m = (\Theta_o - \Theta_m)/\Theta_o = 0.264$$

which is smaller than 0.7.

   The width of the column obtained by the combined method is a little smaller than that obtained by the separated design method.

6.3.3
Beam

(a)
Setting up the design problem

The beam is to be dimensioned for the following loads:

$$M_g = 10 + 0.1d \text{ kNm} \quad (d \text{ in mm})$$
$$M_p = 50 \text{ kNm}$$

The cross-section of the beam is assumed to be rectangular with width b ($\approx$ 300 mm) and efficient height d. At the lower edge of the beam are three steel bars. The yield strength of steel is 400 MPa. The characteristic compressive strength is 40 MPa, the air content is 2% (not air-entrained), and the binding agent is Portland cement.

The beam is supposed to be maintenance free so that corrosion of steel bars in the assumed cracks or degradation of the concrete cover will not hinder use of the column during its service life. The cross-section of hoops (stirrups) must not be fully corroded at cracks. The concrete cover must be at least 20 mm after the service life with no spalling due to general corrosion.

(b)
Ordinary mechanical design

The ordinary mechanical design of the beam is performed using traditional design principles:

$$R_d \geq S_d \qquad (6.28)$$

$$(6.29) \qquad S_d = \gamma_g M_g + \gamma_p M_p$$

$$R_{ds} = A_s Z \frac{f_y}{\gamma_s} \quad \text{(the stress of steel is decisive)} \qquad (6.30)$$

$$R_{dc} = bxz \frac{f_c}{2\gamma_c} \quad \text{(the stress of concrete is decisive)} \qquad (6.31)$$

$$x = d\mu n \left[-1 + \left(1 + \frac{2}{\mu n} \right)^{1/2} \right] \qquad (6.32)$$

$$z = d - 0.4x \qquad (6.33)$$

$$n = \frac{E_s}{E_c}$$

$$\qquad (6.34)$$

$$\mu = \frac{A_s}{bd}$$

A_s is the cross-sectional area of steel bars:

$$A_s = 3\pi \frac{D^2}{4} \qquad (6.35)$$

Taking D=15 mm we get

$$A_s = 530 \text{ mm}^2$$

By setting R_{ds} equal to S_d we get

$$d = 2083 \text{ mm}$$

However, increasing the diameter of the steel bars quickly reduces the efficient height. By changing D to 20 mm we get

$$A_s = 942 \text{ mm}^2$$
$$d = 543 \text{ mm}$$

The calculations are also seen in Figure 6.8.

(c)
Durability design

The target service life is 50 years. The lifetime safety factor is 3.3 for the separated design method and 2.5 for the combined design method. Thus the design service life, t_d, is 165 years for the separated method and 125 years for the combined method.

All sides of the beam are assumed to be exposed to frost action. The environmental factor for frost attack, c_{env}, is 40 and the anticipated curing time is 3 days.

The curing factor is (cf. equation (8.4), section 8.2.2)

$$c_{cur} = \frac{1}{0.85 + 0.17 \log_{10}(3)} = 1.074 \qquad (6.36)$$

As the concrete is made of Portland cement we conclude:

Beam						
	Ordinary mechanical design				**Durability and**	
		Design values	Charact. values		final design	
	γ_a	1.35	1		γ_t	3.3
	γ_b	1.5	1		t_a	50
	γ_c	1.5	1		t_d	165
	γ_s	1.15	1		C_{min}	20
	M_a	86813146	64306034		$\delta L/\delta t$	0.117
	M_p	75000000	50000000		$d'(t_a)$	19.3
	f_c	27	40		$b'(t_a)$	19.3
	f_y	348	400		C_{omin}	39.3
	D	20.0			C_o(chosen)	40
	A_s	942			b_{omin}	339
	b	300			b_o(chosen)	339
	d	543			d_{omin}	562
	n	5.80			d_o(chosen)	562
	m	0.0058			D_{min}	20.0
	x	124			$\delta r/\delta t$	0.03
	z	494			$d'(t_d)$	5.0
	S	161813146			D_{omin}	29.9
	R_c	244109121			D_o(chosen)	30
	R_s	161813146				
	R	161813146				
	R-S	2.98E-08			**Check of the cracking**	
					of concrete cover	
					C	40
					D_{hmin}	10
					D_h(chosen)	10
					C_h	30
					c_{env}	1
					c_{at}	1
					f_{ck}	40
					a	1800
					b	-1.7
					r	12
					t_o	145
					t_i	20
					t_l	165
					t_d	165

Figure 6.8 Spreadsheet design of a beam. Separated design method.

$$c_{\text{age}} = 1$$

Inserting these values into equation (6.19) we get

$$c' = 0.117t \text{ (mm)} \tag{6.37}$$

At the same time corrosion is occurring in steel bars at cracks. The rate of corrosion is evaluated as 0.03 mm/year:

$$d' = 0.03t \text{ (mm)} \tag{6.38}$$

The durability design parameters are as follows (depending on the design service life).

Separated design method

(t_d=165 years)
   The depth of deterioration is

$$c' = 0.117 \times 165 = 19.3 \text{ mm}$$

The required concrete cover is

$$C_{\text{min}} = 20 + 19.3 \text{ mm} = 39.3 \text{ mm}$$

We choose C=40 mm.
   The depth of corrosion at cracks is

$$d' = 0.03 \times 165 = 5.0 \text{ mm}$$

The diameter of hoops must be at least

$$D_{hmin} = 2 \times 5.0 \, mm = 10.0 \, mm$$

We choose D_h=10 mm.

Combined method

(td=125 years)

$$c' = 0.117 \times 125 = 14.6 \, mm$$
$$C_{min} = 20 + 14.6 \, mm = 34.6 \, mm$$

We choose C=35 mm.

$$d' = 0.03 \times 125 = 3.75 \, mm$$
$$D_{hmin} = 2 \times 3.75 \, mm = 7.5 \, mm$$

We choose D_h=10 mm.

The cracking time of the concrete cover is then checked by applying equation (6.22). The following values of parameters are inserted into the formula:

$$C = 40 \, mm \quad \text{(separated design method)}$$
$$\text{or } 35 \text{ (combined design method)}$$
$$C_h = C - D_h = C - 10 \, mm$$
$$f_{ck} = 40 \, MPa$$
$$c_{env} = 1$$
$$c_{air} = 1$$
$$D_h = 10 \, mm$$
$$r = 12 \, \mu m$$

With the separated method we get $\mu(t_L)$=165 years, which equals the design service life (165 years). Thus the concrete cover of 40 mm is adequate. With the combined method we get $\mu(t_c)$= 117 years, which is less than the design service life (125 years). Therefore the concrete cover is increased from 35 to 40 mm. Then the calculated service life is 165 years, which fulfils the requirement.

(d)
Final design

Separated design method

The width of the beam at the start of service life is twice the deterioration depth of concrete added to the width obtained in ordinary design:

$$b_o = b + 2b' = 300 + 2 \times 19.3 = 339 \, mm$$

The effective height of the beam is increased by the depth of deterioration:

$$d_o = d + b' = 543 + 19.3 \, mm = 562 \, mm$$

The minimum diameter of the steel bars is

$$D_{omin} = 20 + 2 \times 5.0 \, mm = 30.0 \, mm$$

We choose D_o=30 mm.

The calculations are presented in Figure 6.8.

Combined design method

Dimensioning is performed using the following formulae:

$$R_d(t_d) \geq S_d \tag{6.39}$$

where

$$S_d = \gamma_g M_g + \gamma_p M_p \tag{6.40}$$

and R_d is the smaller of the following quantities:

$$R_{ds} = A_s(t_d)z(t_d)\frac{f_y}{\gamma_s} \quad \text{(the stress of steel is decisive)} \tag{6.41}$$

$$R_{dc} = b(t_d)x(t_d)z(t_d)\frac{f_c}{2\gamma_c} \quad \text{(the stress of concrete is decisive)} \tag{6.42}$$

where

$$x(t_d) = d(t_d)\mu(t_d)n\left(-1 + \left(1 + \frac{2}{\mu(t_d)n}\right)^{1/2}\right) \tag{6.43}$$

$$(6.44) \qquad\qquad\qquad z(t_d) = d(t_d) - 0.4x(t_d)$$

$$n = \frac{E_s}{E_c}$$

$$\mu(t_d) = \frac{A_s(t_d)}{b(t_d)d(t_d)} = \frac{N_s\pi(D_o - d'(t_d))^2/4}{(b_o - 2c'(t_d))(d_o - c'(t_d))} \tag{6.45}$$

A_s is the cross-sectional area of steel:

$$A_s(t_d) = \frac{3\pi(D_o - 2d'(t_d))^2}{4} \tag{6.46}$$

With D_o=28 and $d'(t_d)$ we get

$$A_s(t_d) = 990 \text{ mm}^2$$

The effective height of the beam, d_o, is now dimensioned by setting R_{ds} or R_{dc}, whichever is smaller, equal to Sd at t=. This gives (cf. Figure 6.9)

$$R_{ds} = S_d => d_o = 531 \text{ mm}$$

The width of the beam is 300 mm and the concrete cover is 40 mm with respect to the main reinforcement.

   The effective height is slightly smaller than that determined by the separated design method. The relative reduction of safety margin (m=0.693) fulfils the maximum requirement of 0.7.

Combined design method in safety class 2

In safety class 2 (ultimate state) the structural durability design is carried out by applying the value 2.2 for γ^t and reduced values for material and load safety factors (cf. section 5.3.4). The calculations are presented in Figure 6.10. The effective height of the beam is 414 mm which is clearly smaller than in safety class 1 (531 mm); 35 mm is adequate for the cover. The relative reduction of Θ is 0.681, which is smaller than 0.7.

   When reduced values of load and material safety factors are used it is essential to check also that the structural capacity and the safety margin determined by the final design are not smaller than those of the ordinary design.

   Durability design:

$$R_m = 265 \text{ kNm}$$

and

$$\Theta_m = 163 \text{ kNm}$$

Ordinary design:

$$R_o = 186 \text{ kNm}$$

and

$$\Theta_o = 72 \text{ kNm}$$

In this case the values of R_m and Θ_m of the durability design are both greater than those of the ordinary design. Thus the dimensions of the final (durability) design are determinative.

Figure 6.9 Spreadsheet calculations for a beam. Combined design method.

Beam

Ordinary mechanical design

	Design values	Charact. values
	t = 0	t = 0
γ_a	1.35	1
γ_b	1.5	1
γ_c	1.5	1
γ_s	1.15	1
M_a	86813137	64306027
M_p	75000000	50000000
f_c	27	40
f_y	348	400
D	20.0	20.0
A_s	942	942
b	300	300
d	543	543
n	5.80	5.80
m	0.0058	0.0058
x	124	124
z	494	494
S	161813137	114306027
R_c	244109073	366163609
R_s	161813125	186085094
R	161813125	186085094
R-S	-11.76266	71779067

Check of the cracking of concrete cover

C	40
D_{hmin}	8
D_h(chosen)	10
C_h	30
C_{env}	1
C_{at}	1
f_{ck}	40
a	1800
b	-1.7
r	12
t_e	145
t_i	20
t_k	165
t_d	125

Durability and final design

	Design values	Charact. values	Charact. values
	t = t_d	t = t_d	t = 0
γ_a	1.35	1	1
γ_b	1.5	1	1
γ_c	1.5	1	1
γ_s	1.15	1	1
γ_t	2.5	2.5	0
M_a	85195765	63107974	63107974
M_p	75000000	50000000	50000000
f_c	27	40	40
f_y	348	400	400
C_{min}	20	20	20
D_{min}	20	20	20
t_a	50	50	50
t_d	125	125	0
$\delta L/\delta t$	0.117	0.117	0.117
$d'(t_d)$	14.6	14.6	0.0
$b'(t_d)$	14.6	14.6	0.0
C_{cmin}	34.6	34.6	20.0
C_c(chosen)	40	40	40
$\delta r/\delta t$	0.03	0.03	0.03
$d'(t_d)$	3.8	3.8	0.0
D_{cmin}	27.5	27.5	20.0
D_c(chosen)	28	28	28
$D(t_d)$	20.5	20.5	28.0
$A_s(t_d)$	990	990	1847
b_e	300	300	300
d_e	531	531	531
$b(t_d)$	271	271	300
$d(t_d)$	516	516	531
n	5.80	5.80	5.80
$m(t_d)$	0.0071	0.0071	0.0116
$x(t_d)$	128	128	162
$z(t_d)$	465	465	466
S	160195765	113107974	113107974
R_c	215468243	323202365	453953369
R_s	160195764	184225129	344445782
R	160195764	184225129	344445782
R-S	-1.0721515	71117155	231337807

Check of structural degradation

Relative reduction of R	0.465
Relative reduction of R-S	0.693

Figure 6.10 Spreadsheet calculations for a beam designed in safety class 2 by the combined design method.

Beam							
Ordinary mechanical design				Durability and final design			
	Design values	Charact. values			Design values	Charact. values	Charact. values
	$t=0$	$t=0$			$t=t_d$	$t=t_d$	$t=0$
γ_a	1.35	1		γ_a	1.3	1	1
γ_b	1.5	1		γ_n	1.38	1	1
γ_c	1.5	1		γ_c	1.4	1	1
γ_s	1.15	1		γ_s	1.13	1	1
				γ_c	2.2	2.2	0
M_a	86813137	64306027		M_a	66866942	51436110	51436110
M_p	75000000	50000000		M_p	69000000	50000000	50000000
f_c	27	40		f_c	29	40	40
f_y	348	400		f_y	354	400	400
D	20.0	20.0		C_{min}	20	20	20
A_s	942	942		D_{min}	20	20	20
b	300	300		t_a	50	50	50
d	543	543		t_d	110	110	0
n	5.80	5.80		$\delta L/\delta t$	0.117	0.117	0.117
m	0.0058	0.0058		$d'(t_d)$	12.9	12.9	0.0
x	124	124		$b'(t_d)$	12.9	12.9	0.0
z	494	494		C_{omin}	32.9	32.9	20.0
S	161813137	114306027		C_o(chosen)	35	35	35
R_c	244109073	366163609		$\delta r/\delta t$	0.03	0.03	0.03
R_s	161813125	186085094		$d'(t_d)$	3.3	3.3	0.0
R	161813125	186085094		D_{omin}	26.6	26.6	20.0
$R-S$	-11.76266	71779067		D_o(chosen)	28	28	28
				$D(t_d)$	21.4	21.4	28.0
				$A_s(t_d)$	1079	1079	1847
				b_o	300	300	300
Check of the cracking				d_o	414	414	414
of concrete cover				$b(t_d)$	274	274	300
C	35			$d(t_d)$	401	401	414
D_{hmin}	7			n	5.80	5.80	5.80
D_h(chosen)	10			$m(t_d)$	0.0098	0.0098	0.0149
C_h	25			$x(t_d)$	114	114	140
C_{env}	1			$z(t_d)$	356	356	358
C_a	1			S	135866942	101436110	101436110
f_{ck}	40			R_c	159515798	223322118	301012942
a	1800			R_s	135866939	153529642	264795633
b	-1.7			R	135866939	153529642	264795633
r	12			$R-S$	-2.9321909	52093532	163359523
t_o	100						
t_i	17			Check of structural degradation			
t_k	117			Relative reduction of R			0.420
t_d	110			Relative reduction of R-S			0.681

7
Durability models

7.1.1
Degradation, performance and service life models

For structural durability design a designer needs durability models with which he can evaluate the time-related changes in materials and structures. These models include design parameters such as structural dimensions, material properties, environmental parameters, etc. The durability model concept covers different types of models that are specified more closely in the following.

Mathematical presentations that show an increase in degradation with time (or age) and with appropriate design parameters are called degradation models. They are used in durability design when the limit state is expressed as maximum degradation.

Degradation can alternatively be presented as a decrease in performance. Mathematical presentations that show decreased performance as a function of time and appropriate design parameters are called performance models. Performance models are used in durability design when the limit state is expressed as the minimum performance.

Mathematical presentations that show the service life of a structure as a function of different design parameters are called service life models. They can often be derived from degradation
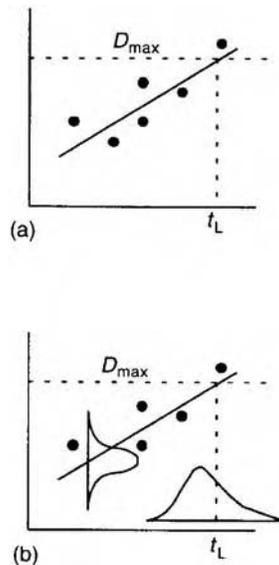
Figure 7.1 (a) Deterministic and (b) stochastic models.

or performance models when the limit states of maximum degradation or minimum performance are known. Service life models are used whenever the design problem can be formulated according to the service life principle (cf. section 3.3).

There may be durability models for different levels such as materials, structural elements or buildings. All of these can be used in durability design. Structural level models such as those for bearing capacity are used in structural design. They are usually created by incorporating degradation models of materials in the basic design formulae of structural design.

7.1.2
Deterministic and stochastic durability models

Durability models can also be divided into deterministic or stochastic models (Figure 7.1). Deterministic durability models are used in deterministic durability design where the scatter of degradation (or performance or service life) is not taken into account. With known values of parameters the models yield only one value (of degradation or performance or service life) which is often the mean value. In some cases, deterministic models are formulated to give an upper or lower fractile value instead of the mean.

In many cases the information yielded by deterministic models is insufficient to evaluate the risk of not reaching the target service life. Especially in the mechanical design of structures, stochastic design methods are considered essential as the scatter due to degradation is normally wide and the degree of risk may be great.

When using stochastic durability models, the structures are designed by ensuring a certain minimum reliability with respect to target service life. The target service life is not an absolute value which must be met at all costs. Rather the requirement is that the probability of the service life falling short of the target service life is smaller than the allowed failure probability. In this way the real nature of service life is better considered and the design corresponds more closely to the expected result.

Stochastic design methods also provide a designer with the possibility of evaluating the sensitivity of different parameters affecting service life. Thus the main attention can be directed to these parameters. This possibility may be valuable even at the developmental stage of modelling.

7.1.3
Durability models for different purposes

(a)
Identification of needs for modelling

Durability models are produced for different purposes. The premises and aims for creating durability models are different depending on the purpose. There are various types and levels of durability data on which the models are based. Sometimes the prediction of service life can be based on the history of, and experience with, the structures, while in other cases tests must be performed to obtain necessary data for such predictions.

There are also differences in the parameters of models depending on the theoretical backgrounds of the models and the requirements of the user. For instance, parameters for design and quality control are different from those for surveillance and repair.

At least the following needs for durability models can be identified:

1. technical material development;
2. ecological evaluation of materials;
3. network level management systems for the maintenance, repair and rehabilitation of structures;
4. planning of project level repairs;
5. risk analysis of important structures;
6. design of a material mix and quality assurance at the construction site;
7. structural durability design.

There is no expectation concerning the weathering resistance of new materials. To prove the resistance of new materials, accelerated ageing tests are performed. To evaluate the service life of a new material an assumption is made that the number of cycles in an accelerated ageing test bears some kind of relationship to the lifetime of that material in actual conditions. By comparing the rate of change of material performance in an accelerated ageing test with that observed in a long-term ageing test under in-use conditions, the service life of a new material can be evaluated (Masters and Brandt, 1989). Comparative tests with some known materials may also serve as a basis for service life prediction (Pihlajavaara, 1984).

For ecological evaluation of materials the prediction of service life may also be based on accelerated ageing tests. The difference is, however, that the range of prediction may be much longer. For example, for the final repository conditions of nuclear waste the required service life of materials may be thousands of years.

Durability models are also needed for planning the maintenance policies and strategies of existing structures with cost analysis. Network level management systems are created for purposes of maintenance, repair and rehabilitation of bridges and other structures. Durability models for such systems are often based on inspection data collected by the agency for which the system is created. The parameters of durability models are then dependent on the data items with which the inspection data can be differentiated in the data base system.

Other kinds of models may be used in planning the repair work on existing single structures. By knowing the age and present condition of the structure, the prediction of future performance and remaining service life is usually possible using an extrapolation model.

Risk analyses are performed for structures associated with great social, economic and ecological risks. Such structures include nuclear power plants, oil platforms, dams, bridges, etc. For risk analysis it is essential to combine different plausible failure modes to determine the critical paths that would violate the security of the structure. All kinds of models, even theoretical, can be used in risk analysis.

Durability models are also needed in the design of concrete mixes. The parameters of such models would be the amounts and proportions of mix ingredients and properties that can be measured from a fresh mix. These models can also be used at the construction site. With the aid of durability models a supervisor of concrete construction work can evaluate the quality of concrete before allowing it to be placed in moulds.

(b)
Models for structural durability design

Parameters of models to be used in structural durability design must be appropriate for a structural designer. Instead of parameters relating to a fresh concrete mix, parameters that are measurable from hardened concrete such as strength, porosity, etc. are preferred. They should also be included in the quality control system of the designed structures.

The strength properties of materials strongly influence the load-bearing capacity of structures and indirectly also the dimensions, span lengths, deformations, etc. As the strength properties already belong to the design parameters of structural design they are also very suitable as parameters of durability models.

If strength properties are not used in the durability models of concrete structures, it is highly probable that other parameters

Table 7.1 Degradation factors and processes

Degradation factor	Process	Degradation
Mechanical		
Static loading	Deformation	Deflection, cracking, failure
Cyclic loading	Fatigue, deformation	Deflection, cracking, failure
Impact loading	Fatigue	Vibration, deflection, cracking, failure
Biological		
Micro-organisms	Acid production	Leaching
Chemical		
Soft water	Leaching	Disintegration of concrete
Acid	Leaching	Disintegration of concrete
Acid	Neutralization	Steel depassivation[a]
Acidifying gases	Neutralization	Steel depassivation[a]
Carbon dioxide	Carbonation	
Sulphur dioxide		
Nitrogen dioxide		
Chlorides	Penetration, destruction of passive film	Steel depassivation[a]
Steel depassivation, oxygen, water	Corrosion	Expansion of steel, loss of diameter in rebars, loss of bond
Stress/chlorides	Stress corrosion	Failure in prestressing tendons
Sulphates	Crystal pressure	Disintegration of concrete
Silicate aggregate, alkalis	Silicate reaction	Expansion, disintegration
Carbonate aggregate	Carbonate reaction	Expansion, disintegration

Degradation factor	Process	Degradation
Physical		
Temperature change	Expansion	Shortening, lengthening, restricted deformation
RH change	Shrinkage, swelling	Shortening, lengthening, restricted deformation
Low temperature, water	Ice formation	Disintegration of concrete
Deicing salt, frost	Heat transfer	Scaling of concrete

Degradation factor	Process	Degradation
Floating ice	Abrasion	Cracking, scaling
Traffic	Abrasion	Rutting, wearing, tearing
Running water	Erosion	Surface damage
Turbulent water	Cavitation	Caves

ª Indicates 'intermediate state'.

with a relationship to strength parameters are used instead. This leads to a risk of contradictory requirements of design parameters. For this reason, the water-cement ratio of concrete, which is often used as a design parameter of concrete proportioning, is not considered suitable for structural design. Another reason for rejecting it as a parameter of structural design is that cement replacements such as blast furnace slag, fly ash and silica fume have been increasingly incorporated into concrete mixes, obscuring the whole concept of the water-cement ratio. Also, the control of the water-cement ratio for hardened concrete is difficult especially if cement replacements are used.

The degree of hydration of cement is another parameter often used for characterizing the quality of concrete (together with the water-cement ratio). Because of cement replacements the concept of degree of hydration has also become obscured, its value as a practical design parameter being reduced. For a structural designer the length of curing time is more meaningful and precise enough for design purposes.

7.2
DEVELOPMENT OF DURABILITY MODELS FOR DURABILITY DESIGN

7.2.1
Qualitative analysis of degradation

The first step in the process of producing durability models is the analysis of degradation factors and processes. All possible degradation factors, processes and effects are listed systematically in a table, as exemplified in Table 7.1. For convenience the degradation factors are subdivided into:

1. mechanical
2. biological
3. chemical
4. physical
5. use.

It is not always possible in this way to make clear distinctions between degradation factors. An example of this problem is biogenic sulphur attack, in which the origin is biological but the degradation is chemical. In such cases the type of degradation factor is based on the type of origin. The main purpose of classifying degradation factors is to present them in a clear overview.

A degradation table, such as Table 7.1 (originally used for risk analysis, in which it is called failure mode and effect analysis— FMEA) can be used by the designer to select the relevant degradation factors. The selection can be done in two ways:

1. selection based on degradation factors expected at the location of the future structure;
2. selection based on risk; only degradation factors with a relatively high risk need be considered. In this respect risk is the probability of failure multiplied by the amount of damage caused by degradation.

Mechanisms of great complexity, in which several mechanisms play a part and/or the effect of a mechanism in turn constitutes a degradation factor for another, can be represented with the aid of a tree diagram (Figure 7.2).

Every influence to which the structure is subjected will produce a particular response, depending on the nature of the influence concerned. For every degradation factor there is a response factor in the structure that tends to resist the degradation. For mechanical loading there is bearing capacity, for frost attack there is frost resistance, etc. The internal response factors are related mostly to material properties and geometrical shapes and dimensions. Generally the responses can be classified similarly to the degradation factors as follows:
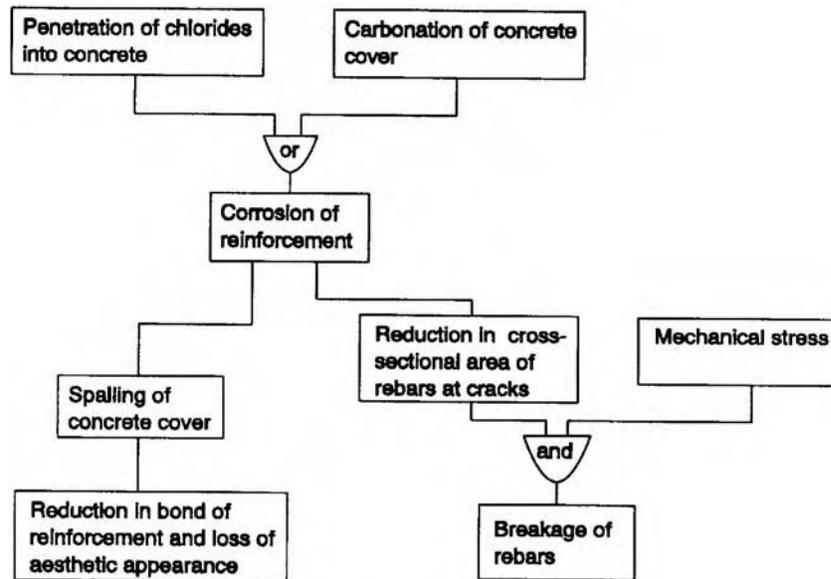
Figure 7.2 Fault tree for the corrosion of reinforcement.

1. mechanical
2. mechanical-physical
3. physical
4. mechanical-chemical-physical
5. chemical
6. geometrical.

When formulating durability models, human error and other uncertainties should also be considered. This category of influence includes the following groups (Kraker, de Tichler and Vrouwenvelder, 1982):

1. uncertainties of design;
2. errors of communication;
3. uncertainties of manufacture and execution;
4. errors in mathematical and statistical modelling.

If not otherwise treated, uncertainties of design, manufacture and execution may be taken into account as extra scatter in durability models. Omitting some parameters in durability models may also lead to greater scatter. However, gross errors cannot be dealt with in the scatter.

7.2.2
Quantification of degradation, performance and service life

The final step in the process of producing durability models is quantification and formulation. Statistical methods and theoretical reasoning are the tools used for these tasks. Simplifications, omitting irrelevant factors and limitation of relevant factors are often necessary actions.

Durability models can be based on empirical or analytical grounds. Empirical models are based on experience and test results. They are developed from results of field surveys and laboratory tests by applying correlative and other statistical methods.

Analytical models are based on laws of nature and fundamental reasoning. They are created as a thorough analysis of degradation mechanisms and kinetics. Before models can be applied, tests are usually required for determining values for some material properties.

Very often, empirical models represent the viewpoint of engineers, and analytical models that of material scientists. A drawback of empirical models is that mechanisms of influence are poorly understood in models in general. Consequently, any deviation from the limits of the model may not be possible without entailing risk. Analytical methods are based on a deeper understanding of the characteristic features of damage, but their practical importance may be small if the parameters in the model are not measurable or the models cannot otherwise be brought to a level of practical utilization.
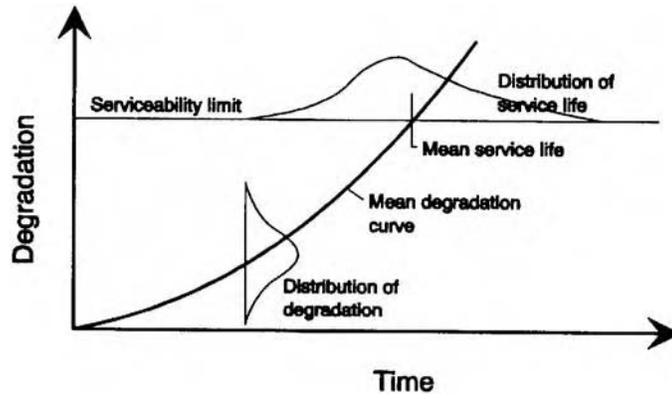
Figure 7.3 Mean degradation and mean service life.

Both the empirical and analytical viewpoints should be considered when developing durability models. Models can be considered good when based on an analysis of mechanisms and factors leading to degradation, yet subjected also to laboratory and field tests. A practical requirement for applying models to structural calculation is that they must be reasonably simple and understandable.

When developing an empirical degradation (or performance) model a field investigation is normally necessary. The aim of the research is to find out the effects of the main parameters, especially environmental ones, on degradation over time. Degradation (or performance) is measured in any appropriate quantities or units. Regression analysis and other statistical methods are used for determining the mean degradation curve (Figure 7.3).

The model can then be extended by laboratory tests. To do that it is essential to ascertain that the laboratory test is valid with respect to the degradation factor studied. The effect of different parameters on the rate of degradation is then studied using test series with varying material properties, dimensions, etc. The information gained from these tests, together with the results of field tests, is often sufficient to build up a degradation or performance model with several parameters as follows:

$$\mu(D) = D(x_1, x_2, \ldots x_n, t) \tag{7.1}$$

where

$\mu(D)$=the mean degradation,

x_1, x_2, x_n=are material, structural, and environmental

parameters, and

t=the age of the structure.

Performance models can often easily be derived from degradation models, as what they represent in fact is the opposite of degradation. If, for instance, a degradation model shows the depth of carbonation with time, the corresponding performance model would show the non-carbonated part of the concrete cover with time. The non-carbonated portion of concrete cover would in this case be the 'capacity' of the structure, i.e. its performance.

The general form of a performance model resembles that of a degradation model:

$$\mu(P) = (x_1, x_2, \ldots x_n, t) \tag{7.2}$$

where

$\mu(P)$=the mean performance and

x_1, x_2, x_n=are material, structural, and environmental parameters, and

t=the age of the structure.

At structural level the performance is not usually a linear function of degradation in materials, and more than one degradation factor may be incorporated in a performance model. For example, in a performance model for the bearing capacity of a concrete column, the bearing capacity is related to the cross-sectional areas of concrete and steel. Thus the degradation models of concrete and steel are written into the mathematical functions of cross-sectional areas in which the width of the column and the diameter of the steel are raised to a second power.

The mean service life is usually approximated as the period of time over which the mean degradation reaches the maximum allowable degradation, or over which the mean performance reaches the minimum allowable performance. When the maximum degradation, D_{max}, or the minimum performance, P_{min}, is known the corresponding mean service life can usually be worked out from the degradation or performance models:

$$\mu(t_L) = t_L(x_1, x_2, \ldots x_n, D_{max}) \tag{7.3}$$

or

$$\mu(t_L) = t_L(x_1, x_2, \dots x_n, P_{\min}) \tag{7.4}$$

where $\mu(t_L)$ is the mean service life.

If sufficient service life data are available the mean service life can, of course, be modelled directly without first modelling degradation or performance. This is, however, seldom the case. When the mean service life is determined from degradation or performance models in the above manner, the result is not exact but it can often be used as a close approximation of the mean.

7.2.3
From deterministic to stochastic durability models

In the stochastic models not only the mean degradation (or performance or service life) is given but also the assumed form of distribution and methods for evaluating the scatter. Instead of a single value a distribution can be obtained for each combination of parameter values by a stochastic model.

In stochastic design deterministic design models are normally used for evaluating the mean degradation. To evaluate the standard deviation a constant coefficient of variation is given. With increasing degradation the standard deviation is also increased.

$$\sigma = \nu\mu \tag{7.5}$$

where

σ=the standard deviation of degradation,
μ=the mean of degradation and
v=the coefficient of variation.

Evaluation of the standard deviation can also be based on the differentiation method. In this method the deterministic model of the mean *(X)* is differentiated with respect to every parameter *(x_i)* in the model and multiplied by the standard deviation of that parameter. The final standard deviation is then determined as follows:

$$\sigma^2(X) = \sum_{i=1}^{n} \left\{ \frac{\partial X}{\partial x_i} \sigma(x_i) \right\}^2 \tag{7.6}$$

where

$\sigma(X)$=the standard deviation of X (degradation, performance or service life),
$\frac{\partial X}{\partial x_i}$=the standard deviation of parameter x_i,
∂x_i= the partial derivative of X with respect to variable x_i, and
n=the number of variables.

The method is exemplified in section 4.3.2.

The Markov chain method can be used to model a typical stochastic degradation process for concrete structures. The method simulates a natural deterioration process starting from the perfect condition and proceeding with gradual and random degradation. Both the scatter and the form of distribution are determined during the process.

The Markov chain method also starts from a known mean curve. However, no parameters or assumptions for the type of distribution are needed. The principle of the Markov chain method is explained in the Appendix (Jiang, Saito and Sinha, 1988; Vesikari, 1995).

Markov chain mathematics has been used in different applications such as network level bridge management systems. A great advantage of the Markov chain method is that it allows linear programming. Thus optimizing analysis can be performed for minimizing repair costs and scheduling repair actions.

8
Durability models for some degradation processes

8.1
ABOUT THE MODELS

In the following several examples of durability models for different degradation factors are introduced. They are not presented as the only plausible ones; other models can be used if they meet the formal requirements of durability models and are otherwise appropriate for durability design. To date there is no general consensus concerning durability models.

As the models are intended for use either in deterministic design or in semi-stochastic design with lifetime safety factors, no other information about stochastic properties except the mean is given. The notation $\mu()$ is not presented in the model formulae as it is assumed that the models always express the mean.

Some introductory remarks precede the models where appropriate. These include an explanation of how the incorporation of degradation is attempted in structural design. Problems have been encountered especially in the case of concrete frost attack and other surface weathering, where the degradation of concrete may appear in different forms.

8.2
FROST ATTACK

8.2.1
Forms of deterioration

By frost attack we mean the gradual weakening or disintegration of concrete surfaces as a result of repeated freezing and thawing. In moist and freezing conditions a decrease in concrete strength occurs and eventual disintegration and complete loss of material is expected near the surface.

Concrete disintegrates as a result of water freezing in its capillary pores. One reason for the damage is the roughly 9% increase in volume that occurs when water freezes. Another reason for the pressure within concrete is the fact that ice crystals tend to grow when kept frozen in a moist environment.

The frost resistance of concrete can be understood as a material property. It is the ability of concrete to withstand repeated cycles of freezing and thawing. The frost resistance depends on other properties of the concrete such as strength, tightness, air content, etc. The rate of disintegration depends, however, not only on the quality of the concrete but also on the aggressiveness of environmental conditions.

The aggressiveness of environmental conditions is exacerbated by deicing salts spread on roads and pavements. Deicing salts not only increase pressures within the concrete, but also diminish its ability to withstand them. A typical feature of frost-salt damage mechanism is gradual scaling of thin concrete layers.

8.2.2
Modelling of frost attack

Pure frost damage appears first as a reduction of strength in the edge zone of a concrete structure. Equation (8.1) shows a model for the reduced strength:

$$f_{ck}(d) = f_{ck}\left(1 - \left(1 - \left(\frac{d}{H}\right)^n\right)\right) \tag{8.1}$$

where

$$f_c = f_{ck}\left(1 - \left(1 - \tfrac{d}{H}\right)^n\right)$$
$$N = f_{ck}(a - x)$$
$$x = H / (n+1)$$
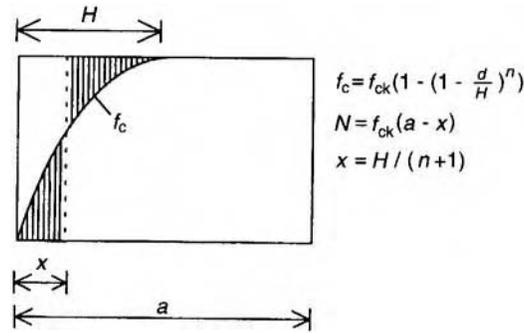
Figure 8.1 Determination of the apparent loss of concrete.
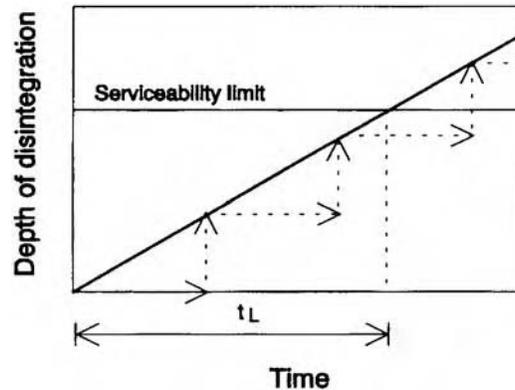
Figure 8.2 Model showing concrete loss due to frost.

$f_{ck}(d) = $ the characteristic compressive strength of concrete at depth d,

$f_{ck} = $ the characteristic compressive strength of undamaged concrete,

$d = $ the depth from the surface,

$H = $ the depth of influence, and

$n = $ an index related to the number of freeze--thaw cycles (or time).

In structural design, the reduction of strength in concrete at edge zones could be handled as decreased design strength, applied over the whole cross-sectional area of concrete or as reduced dimensions of the cross-section. The latter method is here implemented by introducing the concept of apparent loss of concrete.

Figure 8.1 shows the reduction of strength according to equation (8.1) within the depth of influence, H. The value of index n decreases from infinity (at the moment $t=0$) to 0 (after an infinitely long time). The apparent loss of concrete, x, is shown in the figure as calculated. The measure x must be subtracted from the width of the structure, a, to obtain the same capacity for the structure with undamaged concrete. Thus even if no real loosening of the concrete takes place, the damage can be taken into account by reducing the dimensions of the structure by the apparent loss of concrete. The apparent loss is the depth of influence divided by the time-related quantity $n+1$.

Of course, real loosening of concrete also occurs and is highly dependent on other possible forces to which the structure is subjected. Flowing water, ice floats, traffic, etc. are often the final forces causing concrete to loosen. In the absence of such forces, total loss of strength is required before loosening takes place.

The loss of structurally effective concrete as a function of time can be roughly described with a linear model. Figure 8.2 shows such a model in which actual loosening occurs as a series of steps. These can be big or small depending on the presence of chlorides or other possible abrasive forces. The model straightens the steps into a smooth linear process. This is justified by the fact that between steps some apparent loosening takes place in the seemingly whole concrete. The rate of disintegration (meaning the loss of structurally effective concrete in the sense described above) is evaluated from equation (8. 2) (Vesikari, 1994).

The effects of curing, ageing, air content and the compressive strength of concrete were studied with frost-salt tests (Matala, 1991). The applicability of the results was also extended to the area of pure frost attack, as the rate of concrete loss can hardly be measured using methods for pure frost resistance (without salt). The environmental factors are based partly on field tests (Vesikari, 1995).

$$r = c_{env}c_{cur}c_{age}a^{-0.7}(f_{ck} + 8)^{-1.4} \tag{8.2}$$

$$\text{(8.3)} \qquad\qquad s = rt$$

where

r=the rate of disintegration (loss of structurally effective concrete, mm/year)

c_{env}=the environmental coefficient,

c_{cur}=the curing coefficient,

c_{age}=the ageing coefficient,

a=the air content (%),

f_{ck}=the characteristic cubic compressive strength of concrete at 28 days (MPa), and

s=the loss of structurally effective concrete.

The model formula (8.2) is very conservative regarding the compressive strength of concrete. The reductive effect of compressive strength on the rate of disintegration may be greater than that suggested by the formula. However, at very high compressive strengths extra internal cracking may occur which cannot yet be controlled properly. This is why a careful attitude towards compressive strength has been maintained to date.

The curing coefficient is calculated from

$$c_{cur} = \frac{1}{0.85 + 0.17\log_{10}(d)} \tag{8.4}$$

where d is the curing time (days).

The ageing coefficient is calculated from

$$c_{age} = \frac{1}{1 - 0.045p_{sf} - 0.008p_{sl} - 0.001p_{fl}} \tag{8.5}$$

where

p_{sf}=the proportion of silica fume with respect to the total weight of binding agent (%),

p_{sl}=the proportion of blast furnace slag with respect to the total weight of binding agent, and

P_{fl}=the proportion of fly ash with respect to the total weight of binding agent.

The environmental coefficient is evaluated using Table 8.1.

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8.3

SURFACE DETERIORATION

8.3.1
Forms of deterioration

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By surface deterioration of concrete structures we mean different types of weathering mechanisms in outdoor conditions, excluding frost attack which is modelled separately. These include temperature and moisture fluctuations, leaching of minerals from concrete, and physical salt weathering.

Daily temperature changes, especially on surfaces exposed to sunshine, cause gradual cracking on concrete edge zones. Wetting and drying cycles with climatic moisture changes also cause slight cracking and small changes in the porosity of concrete. These phenomena are promoted by the possible incompatibility of aggregate quality with the cement-stone matrix.

Water in contact with concrete surfaces causes leaching of concrete minerals. Loss of material is promoted by chemical reactions of these minerals with diluted gases and ions such as

Table 8.1 Classification of conditions and environmental coefficient values

Class	Conditions	Value of environmental coefficient
1	Very hard	80–160
	frost, snow, ice, numerous freeze-thaw cycles	
	salt water or deicing salts	
	temperature and moisture variations	
	latitudes 60°±5°	

Class	Conditions	Value of environmental coefficient
2	Hard	40–80
	frost, snow, ice, numerous freeze-thaw cycles constant contact with water (no chlorides) temperature and moisture variations latitudes 60°±10°	
3	Moderate	20–40
	normal outdoor conditions freeze-thaw effect latitudes 60°±10°	
4	Favourable	<20
	no freeze-thaw effect	

CO_2, SO_2 and Mg. Flowing water increases the rate of material loss.

Salt weathering is due to a variety of mechanisms associated with the crystallization of salts in the pores of concrete. These mechanisms normally include capillary suction of saline water from the ground or sea, followed by precipitation of salt crystals in pores and cavities while the water is evaporating. An associated mechanism is the expansion and shrinkage of salt crystals as a result of hydration and dehydration leading to cracking and disintegration of concrete.

8.3.2
Modelling of surface deterioration

The concept of apparent loss of concrete which was introduced in conjunction with the frost attack model (cf. section 8.2.2) is also applied to the model for surface deterioration. This means that weakening of concrete at the edge zones of a structure is taken into account as a corresponding apparent loss of concrete. The reduction in load-bearing capacity of a structure is evaluated from the loss of effective cross-sectional area of the concrete, not from reduced strength.

Permeability is probably the most influential property related to the durability of structures. The rate of penetration of water and diluted harmful agents depends on permeability, likewise the rate of leaching of important concrete minerals. The permeability of a concrete is related to its compressive strength.

For concretes of moderate and high strength (30–100 MPa) the rate of disintegration (loss of structurally effective concrete) is considered to be constant and inversely proportional to the power of compressive strength. Using the power −3.3 means that doubling the strength reduces the rate of concrete loss to one-tenth. Equation (8.6) is used to evaluate the rate of disintegration (Pihlajavaara, 1994).

$$r = c_{\text{env}} c_{\text{cur}} f_{\text{ck}}^{-3.3}$$

(8.6)

where

r=the rate of disintegration (loss of structurally effective concrete, mm/year)
c_{env}=the environmental coefficient,
c_{cur}=the curing coefficient, and
f_{ck}=the characteristic (cubic) compressive strength of the concrete.

Table 8.2 Classification of conditions and environmental coefficient values

Class	Conditions	Value of the environmental coefficient
1	Very hard	100 000–500 000
	'Gulf conditions', latitudes 20° ±10° marine structures or structures within the capillary rise of saline ground water temperature and moisture variations	
2	Hard	10 000–100 000
	marine structures or structures within the capillary rise of saline ground water latitudes 40° ±10° temperature and moisture variations	
3	Normal	1000–10 000
	normal outdoor conditions small climatic changes latitudes 40°±10°	
4	Favourable air continuously dry no sunshine	<1000
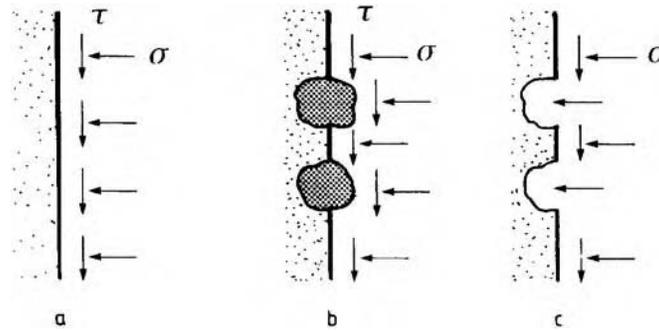
Figure 8.3 Abrasion mechanisms. (Redrawn from S. Huovinen, *Publications No. 62,* published by Technical Research Centre of Finland, 1990.)

The rate of disintegration is highly dependent on the environmental conditions. The service life of good quality concrete may range from 10 to 10 million years depending on the ambient conditions.

The environmental coefficient is evaluated using Table 8.2.

The formula for the curing coefficient is the same as that used in the model for frost attack (equation (8.4)).

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8.4

ABRASION OF CONCRETE BY ICE

8.4.1

Forms of deterioration

</div>

A concrete offshore structure in arctic conditions is subject to various degradation factors. Based on their effect these can be classified as mechanical, physical or chemical. Chemical changes due to sea water include penetration of chlorides and dissolution of lime. Physical changes due to freezing and thawing include microcracking. However, the final cause for detachment of concrete in arctic conditions is almost always abrasion by ice.

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8.4.2

Modelling of abrasion by ice

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The abrasion mechanism due to crushing ice sheets against the concrete surface is of three kinds; abrasion of cement stone (Figure 8.3(a)), abrasion of cement stone+loosening of protruding aggregate stones (Figure 8.3(b)), and abrasion of cement stone when the bond strength between larger aggregate stones and the cement stone is so weak that the stones loosen during the first ice impact (Figure 8.3(c)) (Huovinen, 1990).

The abrasion depth of concrete in arctic sea structures can be calculated as the sum total of the abrasion depth of cement stone measured in icebreaker tests at sea, and the loosening of aggregate stones from the concrete surface. The abrasion rate for cement stone is obtained from the results of icebreaker tests as follows:

$$b = \frac{3}{f_{ck}}s \text{ (mm/km) (mechanism a)} \tag{8.7}$$

where

s=the movement of the ice sheet (km) and

f_{ck}=the characteristic compressive cubic strength of the concrete (MPa).

The total abrasion depth can be calculated with the formula

$$\text{ABR} = \sum_{i=1}^{n} a_i \frac{\log n_s}{\log n_1} R_i + (1 - \sum a_i)b \quad \text{(mechanism b)} \tag{8.8}$$

where

a_i=is the proportional volume of aggregate stones of radius R_i in the concrete,

n_s=the number of ice impacts during ice sheet movements,

n_1=the number of ice impacts when the aggregate stone is loosening (L_{cr}/R=1) and

b=the abrasion rate of cement stone (mm).

Abrasion as a function of ice sheet movement calculated using equation (8.8) is presented graphically in Figure 8.4. The abrasion diagrams are valid when the aggregate distribution of concrete is normal. In addition to the compressive requirement
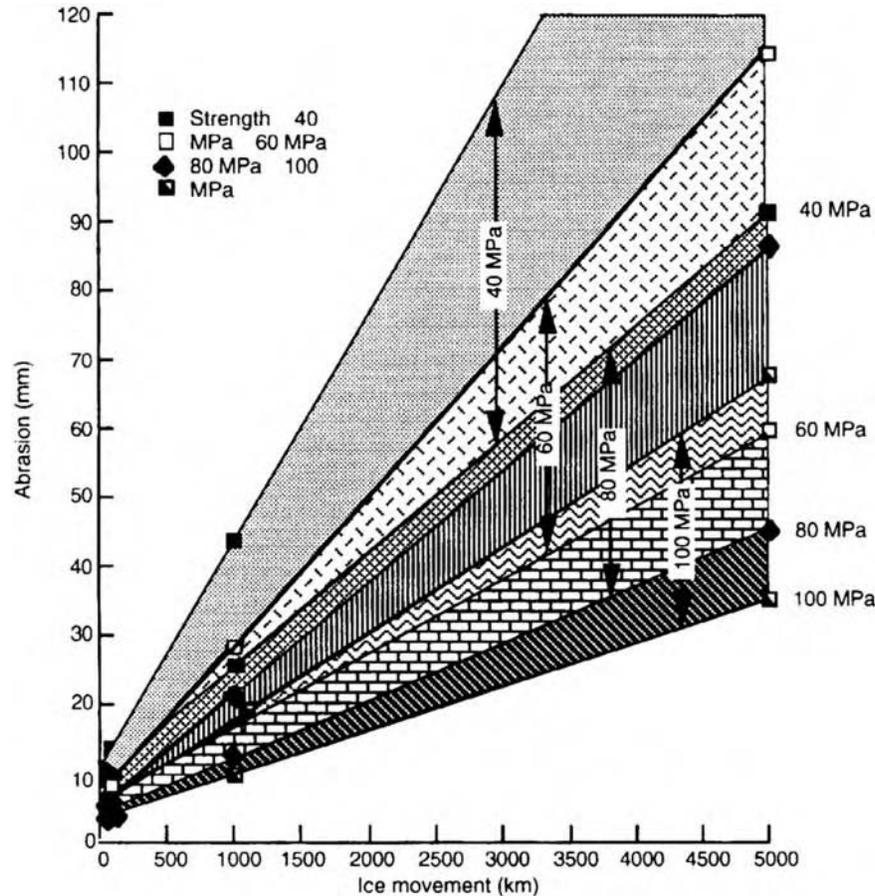
Figure 8.4 Abrasion of concrete for compressive strengths f_{ck}=40, 60, 80 and 100 MPa as a function of ice sheet movement. (Redrawn from S. Huovinen, *Publications No. 62,* published by Technical Research Centre of Finland, 1990.)

f_c=40, 60, 80, 100 MPa it is presupposed that the tensile strength of concrete f_{ct} is at least 10% of the compressive strength and the bond strength between aggregate stones is at least 90% of the tensile strength of concrete (Huovinen, 1990). Figure 8.4 shows that the latter term in equation (8.8) dominates with increasing ice movement.

If the bond between aggregate stones and the cement stone has deteriorated under freeze-thaw cycling the abrasion depth can be calculated with the formula

$$\text{ABR} = \frac{1}{(1 - \sum a_i)} \frac{3}{f_{ck}} s \quad \text{(mechanism c)} \tag{8.9}$$

where

s=the movement of the ice sheet (km) and
$\sum a_i$=the total proportional volume of aggregate stones in concrete.

In the structural design of concrete structures the following approximations for the rate of abrasion can be used:

1. when aggregate stones are not loosening due to frost attack:

$$\frac{d(\text{ABR})}{dt} = \frac{3p'}{f_{ck}} v \quad \text{mm/year} \tag{8.10}$$

2. when aggregate stones are loosening due to frost attack:

$$\frac{d(\text{ABR})}{dt} = \frac{3}{p'f_{ck}} v \quad \text{mm/year} \tag{8.11}$$

where

v=the movement of the ice sheet (km) in 1 year,

p'= the total proportional volume of cement stone in concrete including aggregates up to ø=4 mm (~0.4–0.6 depending on the proportioning of concrete), and

f_{ck}=the characteristic compressive cubic strength of concrete (MPa).

Ice sheet movement may be as much as 10 000 km a year depending on the velocity of wind and sea currents.

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8.5

CORROSION OF REINFORCEMENT

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8.5.1
Concrete as a protective material for reinforcement

The co-operation of concrete and steel is based partly on the fact that concrete gives the reinforcement both chemical and physical protection against corrosion. The chemical effect of concrete is attributed to its alkalinity, which causes an oxide layer to form on the steel surface. This phenomenon is called passivation, as the oxide layer prevents propagation of corrosion. The concrete also provides the steel with a physical barrier against agents that promote corrosion, such as water, oxygen and chlorides.

In normal outdoor concrete structures, corrosion of reinforcement takes place only if changes occur in the concrete surrounding the steel. The changes may be physical, such as cracking and disintegration, exposing part of the steel surface to open air and leaving it without the physical and chemical protection of concrete.

Chemical changes also take place in the concrete surrounding the reinforcement; the most important are the following:

1. carbonation of concrete due to carbon dioxide in air;
2. penetration of aggressive anions, especially chlorides, into concrete.

Carbonation is the reaction of carbon dioxide (in air) with hydrated cement minerals in concrete. This phenomenon occurs in all concrete surfaces exposed to air, resulting in lowered pH in the carbonated zone. In carbonated concrete the protective passive film on steel surfaces is destroyed and corrosion is free to proceed. The effect of chlorides is not based on the decrease in pH but on their ability otherwise to break the passive film.

8.5.2
Modelling of the corrosion of reinforcement

Two limit states can be identified with regard to service life (Figure 8.5):

1. The service life ends when the steel is depassivated. This rule is usually applied to all chloride-induced corrosion as the local attack penetration rate is still not safely quantified and uncertainties concerning the propagation period are therefore high. Thus the service life is limited to the initiation period only (time for the aggressive agent to reach the steel and induce depassivation).

    This rule is also applied to all prestressing steels. The tensile stress of tendons is normally so high that no reduction in the cross-sectional area is permissible and as a result of surface corrosion there is a risk of stress corrosion cracking.
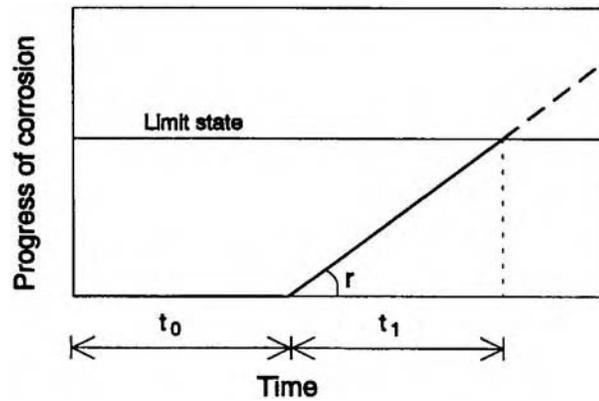
Figure 8.5 Determination of service life with respect to corrosion of reinforcement.

In the cases where no corrosion is allowed the following formula for service life can be used:
$$t_L = t_0 \qquad (8.12)$$
where

$$t_L = \text{the service life, and}$$
$$t_0 = \text{the initiation time of corrosion.}$$

2. The limit state is based on cracking of the concrete cover due to oxides generated during corrosion. In this case the service life includes a certain propagation period of corrosion during which the cross-sectional area of steel is progressively decreased, the bond between steel and concrete is reduced and the effective cross-sectional area of concrete is diminished due to spalling of the cover. This approach is applied in cases where generalized corrosion is developing due to carbonation.

The service life based on cracking of the concrete cover is defined as the sum of the initiation time of corrosion and the time for cracking of the concrete cover to a given limit.
$$t_L = t_0 + t_1 \qquad (8.13)$$
where t_1 is the propagation time. The propagation time t1 ends when a certain maximum allowable loss of the crosssectional area or loss of bond or crack width is reached. These values will depend upon the particular detailing and geometry of each element.

At cracks, originating from the beginning of service life, the initiation time t_0 is much shorter than in an uncracked cover or even t_0=0. In this case it may be written:
$$t_L = t_1 \qquad (8.14)$$
where t_1 is the free corrosion time.

Models for estimating t_0 and t_1 are presented below. When developing these models the assumption has been made that concrete surfaces are free from coatings and sealants.

8.5.3
Initiation time of corrosion

(a)
Chloride-induced corrosion

The commonest sources of chlorides are sea water (marine environments) and deicing salts. Admixed chloride is not considered here.

As a result of chloride penetration a gradient develops near the concrete surface. The time at which the critical chloride content (threshold value) reaches the steel surface and depassivates it, can be regarded as the initiation time of corrosion. The gradient of chloride content is often described by an error function model which fulfils the condition of Fick's second law of diffusion:

$$C_x = C_s \left(1 - \text{erf} \left(\frac{x}{2(Dt)^{1/2}} \right) \right) \qquad (8.15)$$

where

C_x=the chloride content at depth x,
C_s=the chloride concentration at the concrete surface,
x=the depth from the surface of the structure,
D=the diffusion coefficient, and
t=time.
The initiation time for corrosion is obtained from the formula

$$C_{th} = C_s \left(1 - \mathrm{erf} \left(\frac{c}{2(Dt_0)^{1/2}} \right) \right)$$ (8.16)

where
C_{th}=the critical chloride content,
c=the concrete cover, and
t_0=the initiation time of corrosion.
This formula may be simplified by using a parabola function:

$$C_x = C_s \left(1 - \frac{x}{2(3Dt)^{1/2}} \right)^2$$ (8.17)

The formula for the initiation time of corrosion can then be written in the following form:

$$t_0 = \frac{1}{12D} \left(\frac{c}{1 - (C_{th}/C_s)^{1/2}} \right)^2$$ (8.18)

Many standards require threshold values not higher than 0.4% (Cl⁻) by weight of cement for reinforced concrete and 0.2% for prestressed concrete. This corresponds approximately to 0.05–0.07 by weight of concrete (0.025–0.035 for prestressed concrete).

Concerning values of C_s, field experience has shown this quantity to be time dependent at early ages but to tend towards a maximum after a number of years. For the sake of calculation it is usually considered constant. Normal values may be about 0.3–0.4 by weight of concrete.

The coefficient of diffusion is roughly 10^{-7}–10^{-8} cm²/s.

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(b)

Stress corrosion cracking

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Fortunately the phenomenon of stress corrosion cracking is uncommon. It may develop in prestressing wires subjected to corrosive agents, leading to brittle fracture with almost no loss in cross-sectional area. Stress corrosion is incubated in very small surface cracks.

Local steel depassivation is needed to produce surface cracks in which the stress corrosion can incubate. Therefore protecting the prestressing steels from aggressive agents is crucial to their service life, which is always limited to the initiation time of corrosion. As regards intrusion of chlorides the calculation rules presented in section 8.5.3(a) can also be applied to prestressing steels.

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(c)

Carbonation-induced corrosion

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Carbon dioxide in the air penetrates concrete, neutralizing its alkaline substances and producing a carbonation front which advances towards the interior. When this carbonation front reaches the reinforcement, the passive film on the steel becomes unstable and dissolves, enabling generalized corrosion to occur. The initiation time of corrosion is defined as the period of time needed for complete carbonation of the concrete cover.

The rate of carbonation is usually assumed to be related to the square root of time:

$$d = K_c t^{1/2}$$ (8.19)

where
d=the depth of carbonation at time t,
K_c=the carbonation coefficient and
t=time or age.
The initiation time of corrosion can be determined as follows:

$$t_0 = \left(\frac{d}{K_c}\right)^2 \tag{8.20}$$

The carbonation coefficient depends on the strength of concrete, binding agents, cement content and environmental conditions (humidity and temperature). There are several formulae for modelling the carbonation rate. Some of them are analytical, others empirical.

Based on Fick's first law the following expression can be derived for the depth of carbonation (Schiessl, 1976):

$$x = \left(\frac{2D_c(C_1 - C_2)}{a}t\right)^{\frac{1}{2}} \tag{8.21}$$

where

x=is the carbonation depth (m),
a=the amount of alkaline substance in the concrete,
D_c=the effective diffusion coefficient for CO_2 at a given moisture distribution in the pores (m^3/s),
C_1–C_2=the concentration difference of CO_2 between air and the carbonation front (kg/m^3), and
t=time.

This calculation procedure has been extended by Bakker (1993) to cases of fluctuating wetting and drying cycles. During wet conditions the carbonation front cannot progress. During dry conditions moisture evaporates and makes possible progression of the carbonation front.

According to Bakker the time t in equation (8.21) is substituted by t_{eff} which is determined as follows:

$$t_{eff} = \left(t_{d1} + t_{d2} - \left(\frac{x_1}{B}\right)^2 + t_{d3} + \dots t_{dn} - \left(\frac{x_{n-1}}{B}\right)^2\right) \tag{8.22}$$

$$B = \left(\frac{2D_v(C_3 - C_4)}{b}\right)^{1/2} \tag{8.23}$$

where

x_n=the carbonation depth after the nth wetting and drying cycle (m),
t_{dn}=the length of the n th drying period,
D_v=the effective diffusion coefficient for water vapour at a given moisture distribution in the pores (m^2/s),
C_3–C_4=the moisture difference between air and the evaporation front (kg/m^3), and
b=the amount of water to evaporate from the concrete (kg/m^3).

If the drying and wetting periods are of equal length the time elapsed after n cycles is

$$t_n = nt_d + (n - 1)t_w \tag{8.24}$$

where

t_w=the length of the wetting periods and
t_d=the length of the drying periods.

A theoretical model based on the theory of 'moving boundaries' has been presented by Tuutti (1982). The theory deals with diffusion processes in non-steady-state conditions where CO_2 reacts with concrete in such a way that concrete serves as a sink for CO_2. Another theoretical model for the combined effects of frost attack and carbonation has been presented by Fagerlund, Somerville and Tuutti (1994).

Experimental models for evaluating the depth of carbonation have been presented by Häkkinen and Parrot. According to Häkkinen (1993) the depth of carbonation is determined by equation (8.20), the coefficient of carbonation being determined as follows:

$$K_c = c_{env}c_{air}af_{cm}^b \tag{8.25}$$

where

c_{env}=the environmental coefficient,
c_{air}=the air content coefficient,
f_{cm}=the mean (cubic) compressive strength of concrete (MPa), and
a, b=parameters depending on the binding agent.

Instead of the mean compressive strength, the characteristic strength can be used by applying the following relationship (CEB, 1988):

$$f_{cm} = f_{ck} + 8 \tag{8.26}$$

Tables 8.3 and 8.4 show values for the environmental and air content coefficient respectively. The parameters a and b in equation (8.25) are presented in Table 8.5 (Häkkinen, 1991).

Table 8.3 Environmental coefficient for determination of the carbonation rate

Environment	c_{env}
Structures sheltered from rain	1
Structures exposed to rain	0.5

Table 8.4 Air content coefficient for determination of the carbonation rate

Air porosity	c_{air}
Not air entrained	1
Air entrained	0.7

Table 8.5 Parameters *a* and *b* (Source: T.Häkkinen, *Research. Notes 750,* published by Technical Research Centre of Finland, 1991)

Binder	a	b
Portland cement	1800	−1.7
Portland cement+fly ash 28%	360	−1.2
Portland cement+silica fume 9%	400	−1.2
Portland cement+blast furnace slag 70%	360	−1.2

According to Parrot (1992) the depth of carbonation is determined on the basis of the oxygen permeability of concrete:

$$d = \frac{64K^{0.4}t^n}{c^{0.5}}$$

(8.27)

where

    K=the oxygen permeability of concrete at 60% RH,
    t=time,
    c=the alkaline content in the cement, and
    n=the attenuation factor (root power).

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8.5.4
Propagation period

(a)
General rule

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Corrosion begins when the passive film is destroyed as a result of falling pH due to carbonation, or as a result of the chloride content rising above the threshold close to the reinforcement. The volume of corrosion products is many times that of the original metal. The greater need for volume causes tensile stress in concrete around the steel bar, leading to cracking or spalling of the concrete cover.

    When corrosion develops three main phenomena appear:

1. a decrease in the steel cross section;
2. a decrease in the steel/concrete bond;
3. cracking of the concrete cover and therefore a decrease in the concrete load-bearing cross-section.

To determine the length of service life the critical threshold value of the load-bearing capacity has to be defined as related to the aforementioned distressing phenomena. This critical threshold can often be expressed as the critical loss of bar radius provoked by corrosion and, therefore, the propagation period may be quantified in the following manner (Alonso and Andrade, 1993):

$$t_1 = \frac{\Delta R_{max}}{r}$$

(8.28)

where

    t_1=the propagation time of corrosion (years),
    ΔR_{max}=the maximum loss of radius of the steel bar, and
    r=the rate of corrosion.

(b)
Cracking time of the concrete cover

In the case of generalized corrosion the critical loss of bar radius is based on the cracking of the concrete cover. The propagation (cracking) time can be approximated by the following formula (Siemes, Vrouwenvelder and van den Beukel, 1985):

$$t_1 = 80 \frac{C}{Dr} \tag{8.29}$$

where

   C=the thickness of the concrete cover (mm),
   D=the diameter of the rebar (mm), and
   r=the rate of corrosion in concrete (μm/year).
   The rate of corrosion in concrete depends strongly on the ambient conditions. Important environmental factors are relative humidity and temperature. The rate of corrosion of reinforcement in concrete can be evaluated using the following formula:

$$r = c_T r_o \tag{8.30}$$

where

   c_T=the temperature coefficient,
   r_o=the rate of corrosion at +20° C.

Table 8.6 Rate of corrosion in carbonated and chloride-contaminated concrete (anodic areas)

Relative humidity (%)	Carbonated concrete (μm/year)	Chloride-contaminated concrete (μm/year)
99	2	34
95	50	122
90	12	98
85	3	78
80	1	61
75	0.1	47
70	0	36
65	0	27
60	0	19
55	0	14
50	0	9

   Primary factors that affect the rate of corrosion in concrete at +20 °C are the relative humidity of air (or concrete) and the chloride content. Other factors such as the water-cement ratio and the type of cement may also have some influence. The values of corrosion rate in anodic areas of reinforcement presented in Table 8.6 can be taken as approximate averages. They are determined on the basis of experimental data reported by Tuutti (1982).
   The moisture content of concrete surrounding the reinforcing steels is a complex mixture of various climatic and structural effects. The equilibrium relative humidity of concrete in aerial conditions is affected by annual and daily variations of the relative humidity of air, condensation of moisture on the surfaces, rain, splash and melting water, density of concrete and depth from the surface (concrete cover).
   The chloride content also has a great influence on the moisture

Table 8.7 Temperature coefficients and evaluated rates of corrosion for some cities in Europe

City	c_T	Rate of corrosion (μm/year)	
		Exposed to rain	Sheltered from rain
Sodankylä (northern Finland)	0.21	11	2.5
Helsinki	0.32	16	4
Amsterdam	0.47	24	6
Madrid	0.73	37	9

content and the rate of corrosion in concrete. However, the propagation time is normally fully omitted if chlorides are present. The data for chloride-contaminated concrete in Table 8.6 are given mainly for comparison.

The average relative humidity in structures exposed to rain can be evaluated at about 95% (unless the frequency of rains is extremely low) and for structures completely sheltered from rain, about 90%. Consequently the rate of corrosion in carbonated concrete at 20°C would be about 50 μm/year in structures exposed to rain and about 12 μm/year in structures sheltered from rain.

The temperature coefficients determined on the basis of the findings by Tuutti (1982) and average daily temperatures for some European cities are presented in Table 8.7. Rates of corrosion evaluated from equation (8.30) are also listed. The effect of direct sunshine on the surface temperatures of structures has not been considered in Table 8.7. This effect may, however, be considerable. Local microclimatic features should be taken into account when evaluating the rate of corrosion.

It is well known that the rate of corrosion slows gradually with time. However, as few data are available concerning this phenomenon, a constant corrosion rate is recommended in durability design.

(c)
Propagation time of corrosion at cracks

If the concrete cover is cracked from the beginning (due to shrinkage, mechanical stress, etc.) and the crack width is larger than 0.1–0.3 mm, corrosion normally starts without any initiation period. If the steel bars are exposed all around, even corrosion is expected on all sides.

A constructor may set a limit for the minimum diameter of steel bars or the maximum depth of corrosion correspondingly. This may depend on the type of reinforcement—main reinforcement, transverse reinforcement, stirrups, etc.—and the actual stresses in steel bars. No corrosion in prestressing tendons is permissible.

The propagation time at cracks is calculated from the following formulae:

$$t_1 = \frac{S_{\max}}{r} \tag{8.31}$$

$$t_1 = \frac{D - D_{\min}}{2r} \tag{8.32}$$

where
  t_1=the propagation time of corrosion at a crack,
  r=the rate of corrosion at a crack,
  $S_{\max}$=the maximum allowable depth of corrosion, and
  D_{min}=the minimum diameter of the steel bar.

The rate of corrosion in cracks represents an extremely complicated problem which is not yet fully understood. In the absence of more precise data the assumption that the average corrosion rate is of the same order of magnitude as in uncracked concrete is applied. Accordingly the following values for the mean corrosion rates are recommended in the calculations (Andrade *et al.,* 1994):

1. when the only aggressive action is carbonation:

$$RH = 90\text{--}98\% \Rightarrow \text{corrosion rate} = 5\text{--}10 \ \mu\text{m/year}$$
$$RH < 85\% \Rightarrow \text{corrosion rate} \leq 2 \ \mu\text{m/year}$$

2. in chloride-contaminated environments:

$$RH = 100\% \Rightarrow \text{corrosion rate} \leq 10 \ \mu\text{m/year}$$
$$RH = 80\text{--}95\% \Rightarrow \text{corrosion rate} = 50\text{--}100 \ \mu\text{m/year}$$
$$RH < 70\% \Rightarrow \text{corrosion rate} \leq 2 \ \mu\text{m/year}$$

9
Summary and conclusions

9.1
DURABILITY DESIGN IN THIS DESIGN GUIDE

This design guide is in principle based on well-known and accepted stochastic design methods long used in the mechanical design of concrete structures. Here these methods are extended to cover the dimension of time, allowing time-related degradation processes of materials and structures to be taken into account. A new requirement, the target service life, has been introduced in structural design.

This guide is not a formal design code. Rather, it shows in terms of principles, rules and examples how the durability design of concrete structures can be performed and what can be gained by such a design. The purpose of the work has been to bring together the achievements of material research of concrete structures, and to transfer the main results as 'durability models' to the art of structural design.

Structural durability design does not only mean the specification of material properties or the dimensioning of some parts of concrete structures, such as material specifications and thickness of the concrete cover, but it also covers the mechanical design of structures. This is possible by incorporating durability models for concrete and steel into the basic design formulae of load-bearing capacity. In this way, performance models are produced for structural level, showing the reduction of load-bearing capacity with time.

To take the stochasticity of service life into account a new parameter, the lifetime safety factor, has been introduced. The meaning of the lifetime safety factor is the same as that of load safety factors and material safety factors in traditional design. It provides the necessary safety margin against falling short of the target service life. Special statistical methods have been used in the determination of lifetime safety factors.

The guide also gives examples of possible durability models for structural design. Models related to frost attack, surface deterioration and abrasion by ice are presented for concrete; models for carbonation and chloride-induced corrosion are presented for reinforcing steel.

9.2
NEEDS FOR FURTHER DEVELOPMENT OF DURABILITY DESIGN

Only a small part of all durability research has so far focused on methodological problems. Further studies on design methods for concrete structures subjected to time-related degradation processes are still very much needed.

More accurate estimates for the scatter of service life in different working environments are needed in order to optimize the values of lifetime safety factors.

The required safety of structures during service life, and the related failure probability of not reaching the required service life should be further studied. Lifetime safety factors for concrete structures should be agreed upon internationally in norms and standards.

The methods of durability design should also be applied to special types of concrete structures. Analysis of the design methods under diverse action effects still needs to be studied in depth. Not only are 'first level' cross-sectional studies needed, but also 'second level' investigations of the performance of the whole structure, focusing on deformations, sliding and buckling effects, etc.

The durability models chosen for this guide are presented as examples. No doubt there is a need for further development of many models. The accuracy of the models can be improved by appropriate field and laboratory research projects. For some degradation factors complete modelling work has yet to be done.

Additionally the criteria of the service life limit state of each degradation process should be further analysed and agreed upon for international praxis.

The interaction of degradation factors has not been studied in this report. The qualitative and quantitative analysis of different forms of interaction, and the methods of incorporating such interactive processes into durability design, is also a challenge for future research.

This design guide is but one link in the chain of development of the durability design of concrete structures. Its purpose has been to outline the principles of durability design and to reveal, by means of examples, the possibilities and benefits of such a design. However, a more formal durability guide is required for structural engineers, in order to firmly establish durability design. This would be especially important for structures in aggressive environments and for structures at great economic and environmental risk.

The durability design methodology also opens possibilities for applications to the rehabilitation and repair of structures. In such application the performance capacity is increased at the time of repair, from which point of time a new period of service life begins. This requires application of the design procedure, and development of degradation models through theoretical and experimental research of different types of repair methods.

Appendix
Stochastic modelling of degradation by the Markov chain method

The degradation of structures is expressed in terms of discrete condition states. In principle the number of states is not restricted, but the calculations are easier if the number of states is not too large. As an example a degradation index scale of 0, 1, 2, 3, 4 is used, each index corresponding to a state. The degradation index 0 represents the best and 4 the poorest condition. In general we give the interpretation for the five degradation indices listed in Table A.1. Degradation index 3 defines the limit state. Service life is the age at which this state is reached.

The amounts of structures at each state and at a certain age, t, is expressed as a damage index distribution, $Q(t)$. The amounts can be given in any quantity: m^2, m, pieces or percentages.

Changes in condition are expressed as stochastic transition probabilities from one state to another. The transition probabilities are given in a transition probability matrix (or simply transition matrix). Transitions normally mean changes occurring within the space of 1 year (but may be other). Changes after n years can be predicted by multiplying the initial degradation index distribution, $Q(0)$, by the transition matrix n times, as shown in Figure A.1. If we assume that all structures start off in perfect condition, the initial degradation index distribution is of the form | 1, 0, 0, 0, 0 |, meaning that all structures are at the degradation index 0. It is important to note that the transition probability matrix does not change throughout the process. The transition probabilities are not dependent on time.

Table A.1 Interpretation of degradation indices

Degradation index	State of definition
0	Initial state, no degradation
1	1/3 limit state
2	2/3 limit state
3	Limit state
4	Post-limit state

For the transition probabilities the following assumptions are made.

1. The condition of structures cannot be improved during the process.
2. The condition state can either remain the same or shift to the next state within 1 year (transition period).

With the first assumption the structures are never repaired, nor is so-called 'self healing' possible. Because of this assumption, all transition probabilities below the diagonal probabilities are zero. The second assumption can be considered reasonable and moderate for normal deterioration processes of concrete structures in outdoor conditions. Due to this assumption, all probabilities above those next to diagonal ones are zero.

As the structures must either remain at the same state or drop to the next one within a year, the sum of the probabilities of remaining (diagonal elements) and dropping to the next state (elements next to the diagonal ones) must be 1. Thus the probabilities of dropping to the next state can be calculated by subtracting the diagonal probability values from 1. Consequently only the diagonal probabilities of the matrices are unknown parameters.

Since degradation index 4 is the highest possible, structures at that level must always keep their degradation index, and the corresponding transition probability is always 1 (at the lower right corner of the matrix).

The assumed form of the transition matrix is thus as follows:

Figure A.1 Principle of the Markov chain method.

$$P = \begin{vmatrix} p_1 & 1-p_1 & 0 & 0 & 0 \\ 0 & p_2 & 1-p_2 & 0 & 0 \\ 0 & 0 & p_3 & 1-p_3 & 0 \\ 0 & 0 & 0 & p_4 & 1-p_4 \\ 0 & 0 & 0 & 0 & 1 \end{vmatrix}$$

Let us consider that the probabilities p_1, p_2, p_3 and p_4 have numerical values (between 0 and 1). The degradation index distribution for each year, $Q(t)$, is obtained by multiplying the degradation index distribution of the previous year by the transition matrix P (starting from the initial degradation index distribution, see Figure A.1). The result is shown in Figure A.2.

The mean of the degradation index distribution at each year, $E(t, P)$, is obtained by multiplying the scale vector, $R= |0,1,2,3, 4|$, by the degradation index distribution (vector multiplication).

$$Q(t) = Q(t-1) * P \tag{A.1}$$

$$E(t, P) = Q(t) * R \tag{A.2}$$

The principle now is to compare the mean curve obtained with the reference degradation curve, which is assumed to be known. The values for p_1, p_2, p_3 and p_4 are selected to give the best fit to the reference degradation curve. Mathematically the principle is formulated by minimizing the sum of yearly deviations between the reference degradation curve and the Markov estimation for the degradation curve:

$$\min \text{SUMD} = \sum_{t=1}^{N} |S(t) - E(t, P)| \tag{A.3}$$

where

SUMD=the sum of deviations at each year,

N=the number of years within service life (years between states 3 and 4 are not interesting and are omitted),

P=the transition matrix with unknown probability elements, P_i,

$S(t)$=the value of the degradation index of the reference degradation curve at year t, and

$E(t, P)$=the mean of the degradation index distribution calculated by the Markov chain method at year t.

Figure A.2 Degradation index distributions for each year, calculated by the Markov chain method, and curve for the mean degradation index.

In other words, the combination of values p_i that yields the minimum for equation (A.3) is sought. The task is solved as a programming problem.

Let us assume that the reference degradation curves (showing the mean degradation as a function of age) are of three optional types:

1. linear
2. quadratic and
3. square root type.

The curves start from the origin and cross the serviceability limit at a certain critical age, which we call the mean service life. By these two fixed points the curves are fully defined. The types of degradation curve are presented in Figure A.3.

When the reference degradation curve is given, the matrix model is also defined by the process described earlier. As a result we obtain the values for the probabilities p_1, p_2, p_3 and p_4. In addition, all necessary information about propagation of the stochastic degradation process is obtained. The probability functions and probability density functions of service life corresponding to the above-mentioned reference curves and the mean service life of 60 years are presented in Figure A.4.

The tractability of the Markov chain mean curve is best with respect to the square root reference curve and worst with respect to the quadratic reference curve. This is due to the fact that the Markov mean curve is bowed horizontally as the age approaches infinity. The mean can never be greater than the greatest damage index of 4.

For the above reason, comparison between the Markov chain mean curve and reference curves is done only within damage indices 1–3. As damage index 3 is determinative for the service life the form of the reference curve beyond it is not important. The portions of damage indices 3 and 4 are totalled in the probability functions of service life.
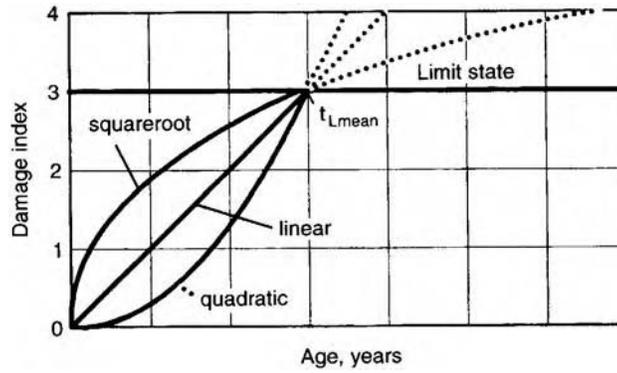
Figure A.3 Types of reference degradation curve.
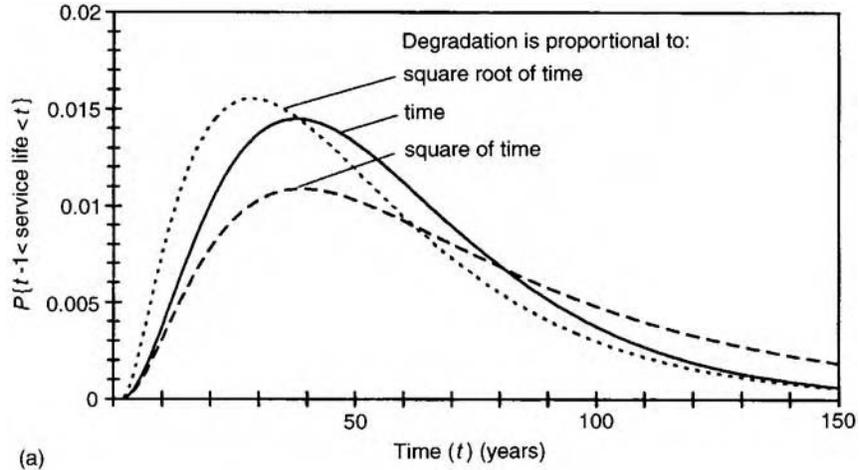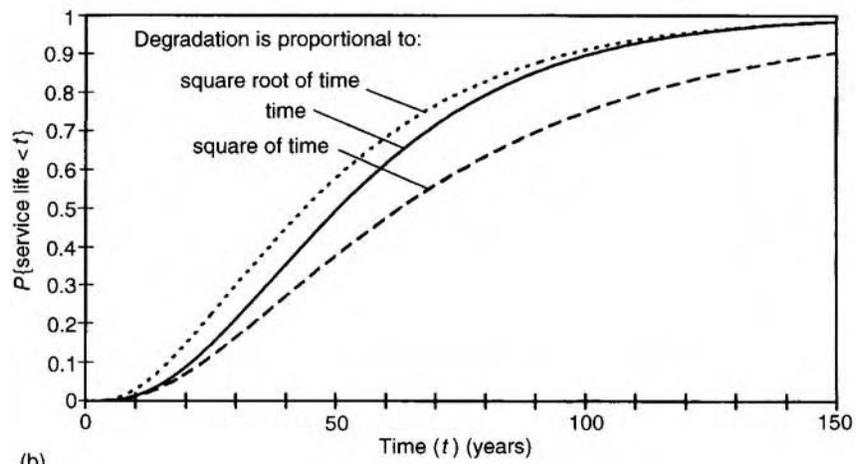
Figure A.4 (a) Probability density functions and (b) (cumulative) probability functions of service life determined by the Markov chain method.

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Index