

Durability of engineering structures

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Design, repair and maintenance

Jan Bijen



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Ever since I became involved in durability, maintenance and repair of civil engineering structures some 30 years ago, I have been appalled by the errors made. While the knowledge on sound design for durability for a specified service life without unexpected maintenance and repair was available through consultants such as me, it was often not applied by designers or contractors. By spending a few thousand euro on a consultant and approximately ten thousand euro on additional measures, millions could be saved. Ranging from the Deira–Shindagah tunnel in Dubai to the Eastern Scheldt Barrier in the Netherlands, knowledge was available but not applied.

Why? In my opinion mainly because the basic knowledge with regard to potential problems was not present in the design teams. Perhaps the knowledge was occasionally present, but not given priority, probably because problems involving durability generally occur long after completion of the works, which makes the people directly responsible for the works less alert. Over the past decade, the importance of design for service life has increased even more because service life is a dominant factor in a building's environmental impact. Some architects even claim that durable buildings are sustainable buildings!

Nevertheless, there have been exceptions where full attention was given to aspects of durability. In 1981, I became involved as an expert in the so-called 'Durability Group' of the contractor who won the tender for the building of the King Fahad Causeway, a 25 km long causeway linking Saudi Arabia to Bahrain. Half of the causeway consisted of a segmental pre-stressed concrete bridge. The initial specifications, written by Worldbank experts, were changed substantially and the changes were based on the recommendations of the Durability Group. This bridge is still in excellent condition 17 years after completion and I am convinced that this is due to the efforts made to design for a 75 year service life without major repair on the concrete structure.

However, experts in durability, such as corrosion engineers and repair consultants, often do not become involved until problems with durability

become apparent; corrosion is visible, structures have failed and owners are distressed. To restore the performance of structures up to a level where an initial requirement (specified or not) for a design service life can be met, can, after all, be very expensive. For instance, the repair costs of the Deira–Shindagah tunnel in Dubai amounted to about half the initial direct investment of building the tunnel, and recoating the steel plates of the Eastern Scheldt Barrier in the Netherlands requires €40 million.

These repairs are, of course, interesting jobs for experts, both with respect to the complexity of the problem, as well as to the fees involved, which greatly exceed the fees for advice on durable building.

I have taught MSc students of Civil Engineering at the Faculty of Civil Engineering and Geosciences on the subject of durability, maintenance and repair for almost 20 years. The present book is an extension and translation of my lecture notes in Dutch. My lectures are organized in a somewhat unusual manner; they include five days of excursion to various projects and institutes such as the Eastern Scheldt Barrier where students can see for themselves what it means to recoat 30 000 m² of steel above sea-level and look at concrete repair work and see the implications of poor design and workmanship. The students have to compile three case studies, based on my own experience in practice. Some of these case studies have been described in this book.

I have found there are no books available that discuss the durability aspects of civil engineering structures in an integrated manner. This has motivated me to compile this book. I expect that those who have read the book thoroughly will have acquired the basic knowledge to design for service life, either by themselves or by involving experts.

Jan Bijen

About the author

Jan Bijen is a former Professor of Materials Science in the Civil Engineering Materials Section of the Faculty of Civil Engineering and Geosciences of the Delft University of Technology. He was the Director of INTRON BV, the institute for quality assessment in the Dutch building industry, for 20 years. He is the Director of FEMMASSE BV, a supplier of software for materials and structural engineering for the building industry, and Director of BouwQ BV, an association between four bodies in the Netherlands: Geodelft, INTRON BV, TNO-Bouw and Wagemaker BV, focusing on the quality assessment of building structures.

He is an expert in civil engineering materials, including durability problems and environmental aspects of building materials, and was a consultant working on design for durability in major projects, such as the building of the Saudi Arabia–Bahrain Causeway and the Great Belt bridges in Denmark. In the areas of maintenance and repair, he worked on the Deira–Shindagah tunnel in Dubai, the Al Hamdi Suez Canal tunnel, the conservation of the Zeeland bridge in the Netherlands, the arbitration of the Dubai Dry Docks, and the conservation of the steel structures of the Eastern Scheldt Barrier and the Maeslant Barrier in the Netherlands.

He is an active member of many national and international scientific committees and is Chairman of the Dutch Standard Committee on Environmental Profiles of Building Products and Chairman of the Dutch National Committee on Sustainable Building.

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My sincere thanks are due to all those listed hereunder.

Prof Dr J H W de Wit (co-author of Chapter 2) is part time professor in corrosion technology at Delft University of Technology. He has written 320 refereed publications in scientific journals and books and is editor of the books *Case Studies in Manufacturing with Advanced Materials*, Volume I, 1992 and Volume II, 1995. He has contributed to several chapters in different books on materials science and technology. From 1996 until 2002 he was Chief Technology Officer of Corus (formerly Hoogovens). Since November 2002 he has been a member of the TNO Board of Management and as such supervises several TNO institutes in the area of advanced products, processes and systems. Prof de Wit is a member of many national and international scientific committees such as the Royal Dutch Chemical Society and a program-committee Innovative Research Program (IOP-Zware Metalen) for the Ministry of Economic Affairs. Among other things, he is chairman of the Board of the Netherlands Corrosion Centre (NCC) and of the Industrial Platform Metals as well as a member of the Executive Board of the Netherlands Institute for Metals Research (NIMR). Internationally, for instance, he is chairman of the International Corrosion Council (ICC), chairman of the Program and Planning committee of the European Industrial Research Management Association (EIRMA) and president-elect of the Governing Board of the EIRMA. He also is a member of the Scientific Board of the Max Planck Institut für Eisenforschung Düsseldorf.

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He is an expert on timber engineering, having completed a PhD study on bolted timber connections from which the results were used in codes on timber structures. He is active in several national and international committees and is National Technical Contact for the Eurocode on timber structures.

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In this job he promotes the simplifying of movable retaining works by sliding guidance and hydrofeet instead of wheel guidance, by the use of alternative coatings and construction materials: fibre reinforced plastic (FRP) and high strength concrete C110 . . . C200. Lock-doors and bridges made of FRP have been designed and erected as well as movable bridges and retaining works of high strength concrete.

Ton Siemes (co-author of Chapter 6) is senior researcher at the institute TNO Building and Construction Research (TNO-Bouw) in Delft, the Netherlands. He has a long experience on durability and fatigue aspects of new and existing concrete structures. Recently he has contributed to a new service life design method 'DuraCrete' for concrete structures. The new method is based on performances, limit states and reliability and is as such a further development of the structural design method. In the Netherlands this new design method is applied in important new concrete infrastructures such as the Western Scheldt Tunnel, the Green Heart Tunnel and others in the High Speed Railway Link between Amsterdam and Brussels.

He is a member of national and international scientific committees on durability. He contributed, for example, to the work of RILEM 130 *Calculation methods for service life design of concrete structures*, CEB V *Concrete in operation and use*, CIB W80/RILEM 175SLM *Service life methods*, and fib TG 5.6 *Model Code on Service Life Design of Concrete Structures*.

Thanks are also due to: Charles Hendriks, my colleague and Professor of Materials Science at the Civil Engineering Materials Section of the Faculty of Civil Engineering and Geosciences of the Delft University of Technology, Head of Section. He was the one who asked me to convert my lecture notes into the present book and obtained the funds for the drawings and the correction of the English text. Gerard Reimerink of the 'Stichting Doelmatig Verzinken' (Foundation of Effective Galvanization), who commented on the sections on galvanizing. Peter Nuiten of BIM BV, a repair and maintenance company, who assisted us with the case study on cathodic protection in practice. Colleagues of my former company INTRON BV, who assisted us on various subjects: Jacques Boosten on the concrete chapter, Jo van Montfort and Martin de Jonker on the case study chapter. Waldeemar Homan and René Klaassen of the 'Stichting Hout Research' (Foundation for Wood Research), who commented on the chapter on wood. Joost Gulikers of the Dutch Ministry of Transport, Public Works and Water Management for providing information on biological corrosion and Ralph Hamerlinck of Bouwen met Staal (Building with Steel), who provided the

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The performance of engineering structures changes with time. In general, deterioration mechanisms cause decreasing performance. This is of course quite natural, but is often forgotten or not given full attention during the design stage. Unfortunately, most standards for structural design and building products do not cover design for continuous compliance with structural and other requirements during service life, or do so only to a limited extent. Designing for durability is left to the structural designer or the architect who often do not have the required skills. The result can be failure with unforeseen maintenance and repair often at high costs and, in the most severe case, human casualties.

Another negative aspect of unforeseen maintenance is that the environmental impact of a structure, considered over its whole service life time, is increased substantially due to the unforeseen measures to be taken. Knowledge of the long-term behaviour of materials, building components and structures is the basis for avoiding such problems. On the basis of the knowledge of durability, it is possible to assess a structure over its whole design service life, including the usage phase. Life-cycle costing and environmental life-cycle assessment become possible. It is this book's intention to provide such knowledge for civil engineering structures.

An engineer may be confronted with a deteriorating structure where repair is required to maintain it and to restore its performance to such a level that structural integrity and other safety requirements are warranted. In such a case the knowledge of the principles of repair, the products and the durability of the repaired structure is then as important as for structures that are newly built. Interaction between the existing structure and the repair materials can complicate the durability assessment. It becomes even more complicated when the deterioration process affects structural safety. Does the structure have to be abandoned? Are temporary measures required to safeguard stability during repair? Which measures must be taken to restore structural safety? Does the repair affect other performance

requirements, such as fire safety? A second objective of this book is to provide knowledge on repair techniques and their durability.

The book is aimed at students as well as at engineers who are involved in the maintenance and repair of structures. It intends to provide sufficient knowledge for decisions to be made with respect to design for service life, maintenance strategies and repair techniques. It is not intended as a manual for maintenance and repair. In general, it provides technical knowledge for assessing the service life of structures and for taking measures to safeguard the functioning of a structure during service life. For detailed analysis and specifications, specialists should be involved.

The main man-made materials used in civil engineering construction are discussed:

- Metals, steel and aluminium (Chapter 2).
- Concrete (Chapter 3).
- Wood (Chapter 4).

Furthermore, some relatively new so-called high-performance materials are discussed in Chapter 5; they include high-performance concrete, high-performance steel and fibre-reinforced polymers (FRP).

Deterioration mechanisms and the measures to counteract these, as well as subsequent maintenance and repair techniques are considered for all these materials. Chapter 6 discusses strategies for durability, maintenance and repair, including life-cycle costing and environmental life-cycle assessment methods. Finally, practical cases are discussed in Chapter 7; they are examples of the case studies my students have to compile within the framework of the ‘durability of engineering structures, design, maintenance, repair’ course.

Please note that specifications given in this book are not suitable for use without adjustments to the specific environmental situation in which structures are built.

2.1 General

Steel is an important material in structural engineering and is also used extensively in non-structural applications such as corrugated steel roofing, cladding and steel window frames. Aluminium is used less as a structural engineering material, but its use in non-structural applications is growing quickly, due largely to its non-corrosive behaviour in most natural climates.

This chapter will discuss:

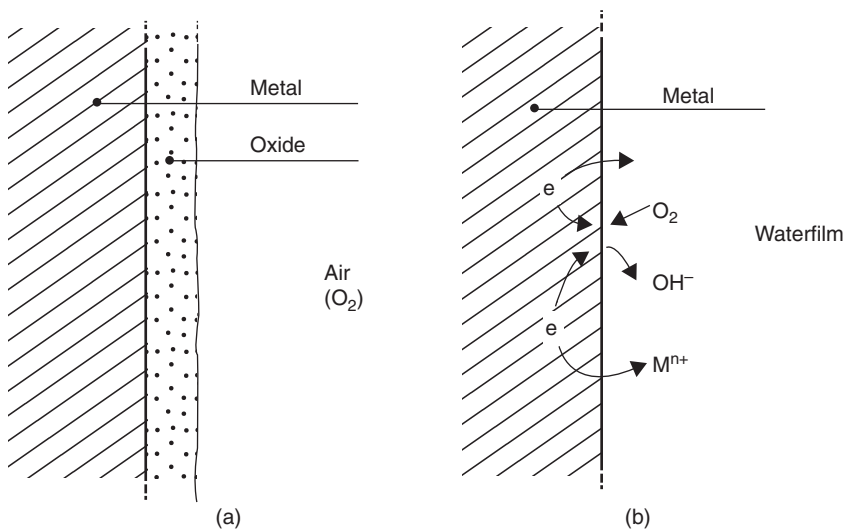
- Corrosion mechanisms.
- Corrosion forms.
- Corrosion in practice.
- Protection of metals.

2.2 Corrosion mechanisms

2.2.1 Introduction

Corrosion can be defined as the degradation or destruction of metals resulting from their chemical interaction with the environment. We can distinguish two types of corrosion: direct reaction of the corrosive compound with the metal and corrosion that occurs through the water present at the metal surface.

The first type of corrosion is referred to as dry or high-temperature corrosion. An example of this is a metal that reacts directly with oxygen. In the metals aluminium and steel this occurs only at higher temperatures during the production process (see Fig. 2.1 a) when a layer of corrosion products is formed. The density of this layer is very important in determining whether further corrosion will take place at ambient temperatures and humidity. If the layer is very dense it can protect the metal against further wet corrosion, as in the case of aluminium. In non-alloyed steel, this layer is called



2.1 (a) High temperature oxidation. (b) Electrochemical corrosion.

mill-scale; it is porous and permeable and does not protect the steel against corrosion.

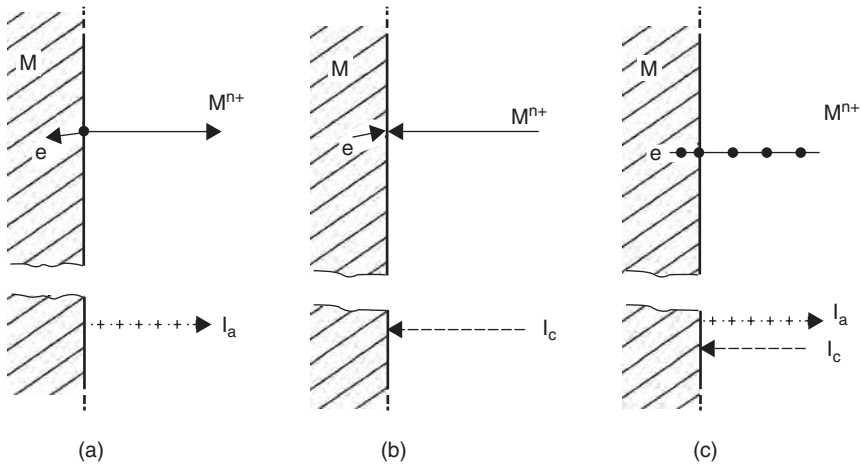
In civil engineering structures, corrosion in a wet environment is much more common than is dry corrosion. Water is, of course, present at the surface of metal structures immersed in water or in wet soils. Yet structures exposed to the atmosphere will also generally have a (thin) layer of water at the surface. This is due to the high surface energy of the metals, which can be decreased by water adsorption. Only in very dry climates will there be no water layer and thus no risk of corrosion. Water in itself is not the reactant, but it is the medium for the corrosion reaction. Wet corrosion is an electrochemical reaction; the metal ions dissolve in water and conduction of current through water is due to the ions dissolved in water. The metal in itself is also a good conductor so that the current chain is closed (see Fig. 2.1 b).

In this chapter we will start by explaining the corrosion mechanism and then go on to discuss the corrosion rate by means of reaction kinetics. Finally, some typical electrochemical corrosion systems will be discussed. References 1 and 2 give further information.

2.2.2 Corrosion mechanisms

2.2.2.1 Metals

Assuming no reaction takes place between water and the metal, the following equilibrium will be established when the metal is in contact with water:



2.2 Subsequent stages of the oxidation process.



The reaction from the left to the right is referred to as oxidation, where the metal (M) is converted into the metal ion (M^{n+}). The latter dissolves in water (see Fig. 2.2 a) and the electrons remain in the metal. The oxidizing reaction is also referred to as the anodic reaction (see Fig. 2.2 b). The reverse reaction where metal ions are converted into metal at the metal surface is referred to as a reduction reaction or cathodic reaction and here electrons are extracted from the metal. This reaction system is called a redox system. It will achieve equilibrium when isolated from other systems and without external influences. This means that the quantity of metal formed in the reduction reaction is equal to the quantity of metal oxidized into M^{n+} ions (see Fig. 2.2 c).

2.2.2.2 Nernst law

From thermodynamics we know that a system consisting of one or more redox systems tends to the lowest enthalpy possible. For electrochemical reactions the Nernst law is applicable:

$$E = E_0 + \frac{RT}{nF} \ln[M^{n+}] \quad [2.2]$$

Where: E = the electrochemical potential
 $[M^{n+}]$ = molar concentration of metal ions
 n = valence of metal ion
 R = gas constant
 T = absolute temperature

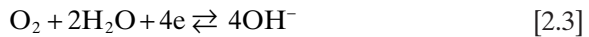
F = Faraday constant

E_0 = standard potential, which corresponds to a situation where $[M^{n+}] = 1$

In the case where the redox system is in equilibrium, as shown in [2.1], the electrochemical potential is called the equilibrium potential, E_e . The equilibrium potential, E_e , standard potential, E_0 , and corrosion potential, E_{cor} , (to be discussed later) frequently fall within the range of -1.5 to $+1.5$ volts and can be determined by means of a voltmeter with a high impedance and a reference electrode with a known potential.

2.2.2.3 Non-metal redox systems

Not only do metals in contact with water have an electrochemical potential, other systems do as well. In civil engineering practice, the most important ones are:



The standard potential for a number of systems is given in Table 2.1.

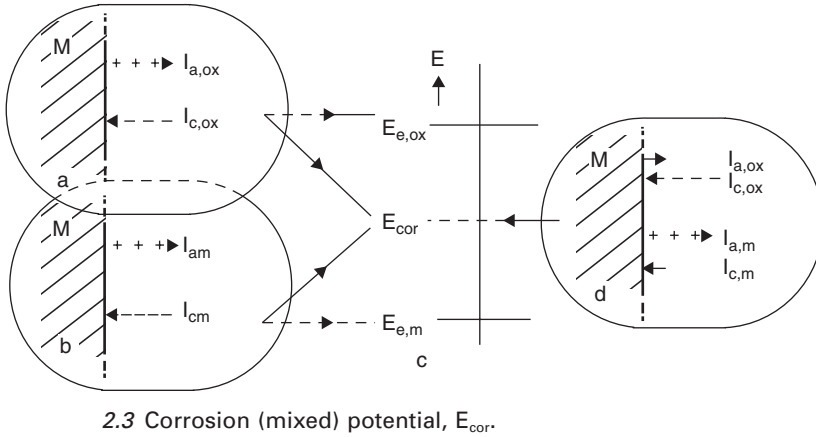
2.2.3 Thermodynamics and corrosion

Corrosion only occurs if a metal is in contact with another redox system of which the equilibrium potential, $E_{e,OX}$, is larger than that of the metal system, $E_{e,m}$. This system then oxidizes the metal and is itself reduced.

A metal in contact with another redox system will adopt one potential, called the mixed potential or, in the case of metals, the corrosion potential. This potential will be between the two equilibrium potentials (see Fig. 2.3). This means that the metal system, M/M^{n+} receives a potential that is higher

Table 2.1 Standard potentials (or electromotive force)

Reaction	Standard potential E_0 in volts
$Mg \rightleftharpoons Mg^{2+} + 2e$	-2.34
$Al \rightleftharpoons Al^{3+} + 3e$	-1.67
$Zn \rightleftharpoons Zn^{2+} + 2e$	-0.76
$Fe \rightleftharpoons Fe^{2+} + 2e$	-0.44
$H_2 \rightleftharpoons 2H^+ + 2e$	0.00
$Cu \rightleftharpoons Cu^{2+} + 2e$	+0.34
$4OH^- \rightleftharpoons O_2 + 4e + 2H_2O$	+0.40
$Ag \rightleftharpoons Ag^+ + 1e$	+0.80



than the equilibrium potential, $E_{e,m}$, and consequently is not in equilibrium. The anodic part of the reaction, $M \rightarrow M^{n+} + ne$ will dominate.

2.2.4 Corrosion rate

2.2.4.1 Single system

Thermodynamics allows us to determine the direction of a corrosion process but not the rate. We can expect severe corrosion from a consideration of thermodynamics in some systems but in reality sometimes nothing appears to happen. Passivated metals, for instance, do not appear to corrode where expected (see Section 2.2.5.3). For the corrosion rate reaction kinetics must be considered.

For electrochemical reactions, the reaction rate is related to the current. For the system



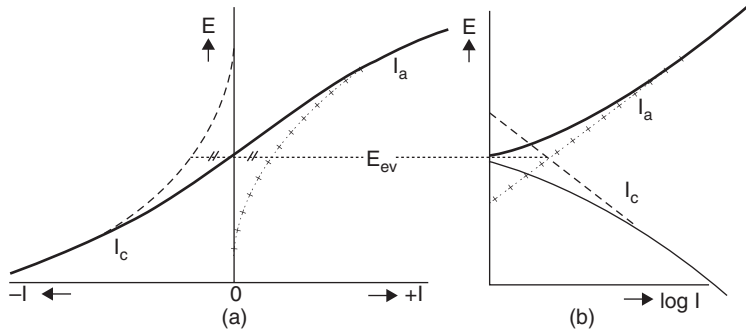
the following equation applies:

$$I_a = I_0 \exp[\alpha n F (E - E_e) / RT] \quad [2.6]$$

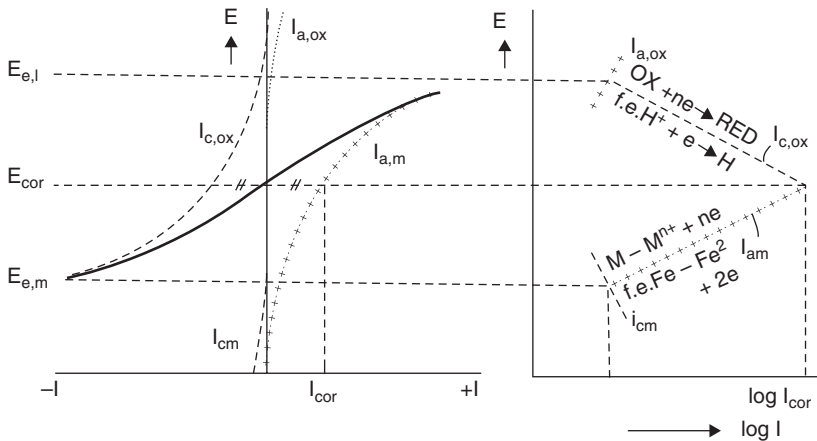
$$I_c = I_0 \exp[-(1 - \alpha) n F (E - E_e) / RT] \quad [2.7]$$

where: α = symmetry coefficient taken to be close to 0.5

The relationship between E , I_a and I_c respectively is given in Fig. 2.4 a. $E - E_e$ is the term that shows how far the potential is removed from equilibrium and is referred to as polarization. Its size is referred to as the over-potential. If the potential is higher than the equilibrium potential the anodic current dominates and it is known as the anodic polarization. If the cathodic



2.4 Partial currents I_a – I_c as function of the potential. The drawn line presents the measurable effective current-potential line: (a) linear scale for current (b) logarithmic scale for current.

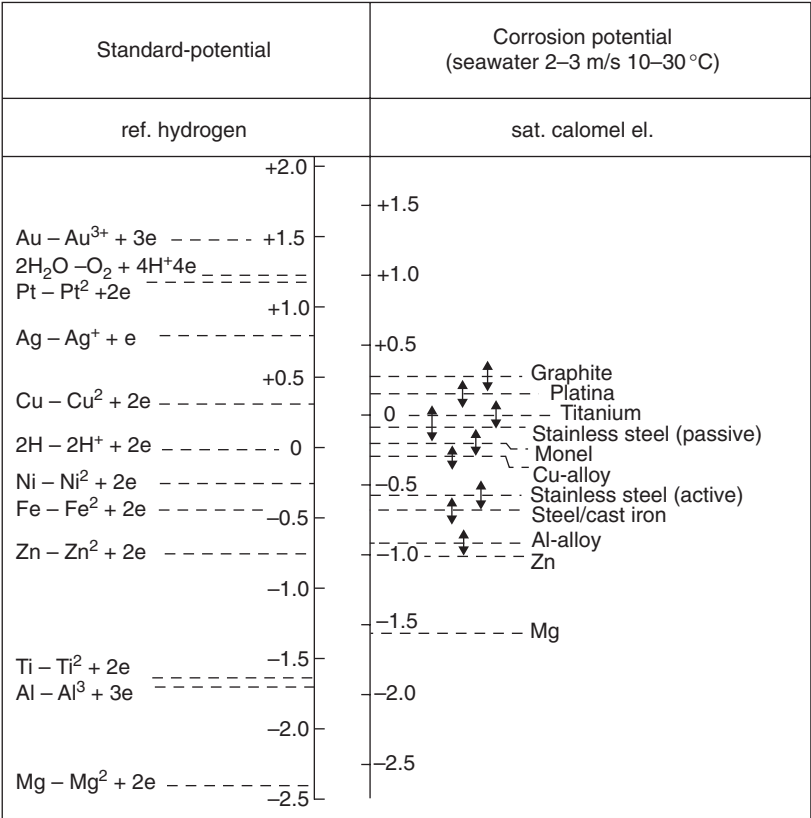


2.5 Current potential lines for a corrosion system consisting of the systems OX/RED and M/M^{n+} .

current dominates, it is known as cathodic polarization. Often a logarithmic ordinate is used for the current density. Such an E – $\log I$ graph is shown in Fig. 2.4 b.

2.2.4.2 Corrosion (mixed) systems

When a metal system is in contact with another redox system, the situation illustrated in Fig. 2.5 will arise. The mixed potential or corrosion potential coincides with the location on the graph, where the anodic partial current densities equalize the cathodic partial current densities. The corrosion rate at this corrosion potential is determined by the size of the partial current of the metal oxidation reaction (I_{cor}). A number of standard and corrosion



2.6 Potential scale with the standard and corrosion potentials of a number of metals and other systems.

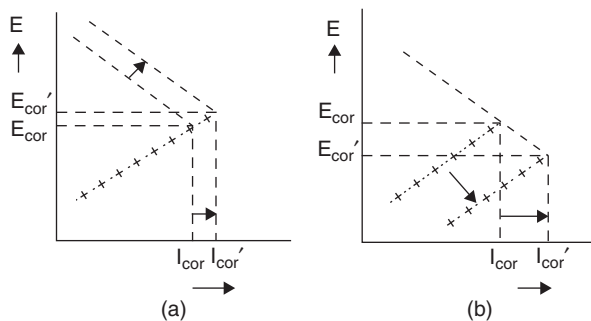
potentials of metals are given in Fig. 2.6; the corrosion potentials relate to aerated seawater.

2.2.5 Corrosion systems

Three types of corrosion system that frequently occur in practice will be discussed in this section using current–potential characteristics.

2.2.5.1 Rate of electron-exchange

In the first type of corrosion system, the corrosion rate is determined by the rate of electron exchange at the metal/water interface which has already been discussed in the previous section. There exists an exponential relation between potential and reaction rate.



2.7 Factors influencing the rate of the corrosion process: (a) shift of partial reduction current, $I_{c,ox}$ (b) shift of partial oxidation current $I_{a,m}$.

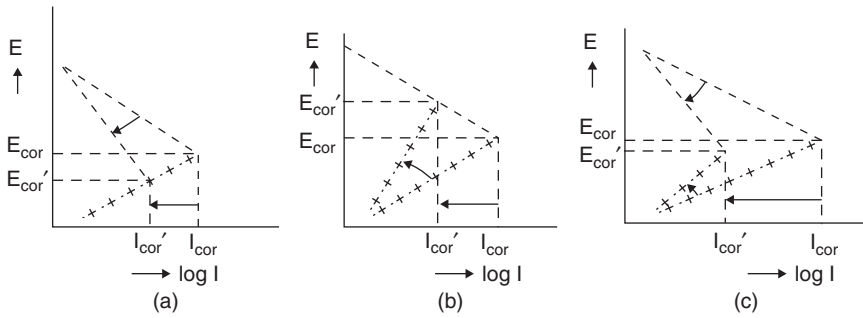
The corrosion rate is determined by all those factors that determine the partial currents of both redox systems. For instance, an increase in the concentration of the component OX will increase the equilibrium potential $E_{c,ox}$ (see Nernst's law in Section 2.2.2) and raise the partial current ($I_{c,ox}$). Consequently, the corrosion current I_{cor} , will increase. This is illustrated in Fig. 2.7 a. An example of such an oxidation reaction is steel corrosion in an acid environment. In the case of increased acidity, there will be an increase in the hydrogen ion concentration, $[H^+]$ and a consequent increase in the corrosion rate.

A similar increase in corrosion rate will occur in the case of a decrease in the concentration of metal ions (M^{n+}). Such occurs when the oxidized metal ions precipitate as a solid and are no longer part of the redox system, as illustrated in Fig. 2.7 b. This actually happens when iron is oxidized in an aerated environment. The ferro ions (Fe^{2+}) are oxidized further to ferri ions (Fe^{3+}), which precipitate as oxides e.g. Fe_2O_3 .

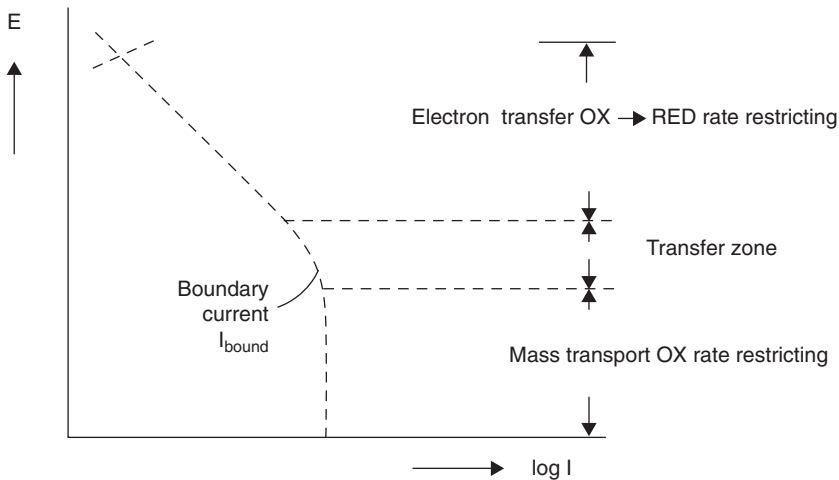
Apart from the equilibrium potential, the slopes of the E -log I curves are of importance. A steeper slope indicates slower kinetics of the electrode processes and leads to a lower corrosion rate, as shown in Fig. 2.8. A steeper slope can be obtained by adsorption of certain chemical compounds at the metal surface and if these additives are added deliberately they are called inhibitors. Sometimes compounds that are adsorbed at the surface and decrease the corrosion rate are provided naturally.

2.2.5.2 Rate of mass transport

The corrosion rate is often not determined by the rate of electron exchange at the metal/water surface, but by the rate of mass transport. In general, this concerns the transport of reaction components through water. Often this concerns the reduction of the oxidizing agent, OX, which must be transported

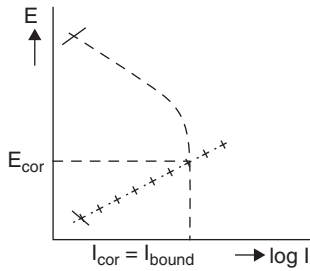


2.8 Change in slope polarization curves: (a) change in angle reduction process (b) change in angle oxidation process (c) both.

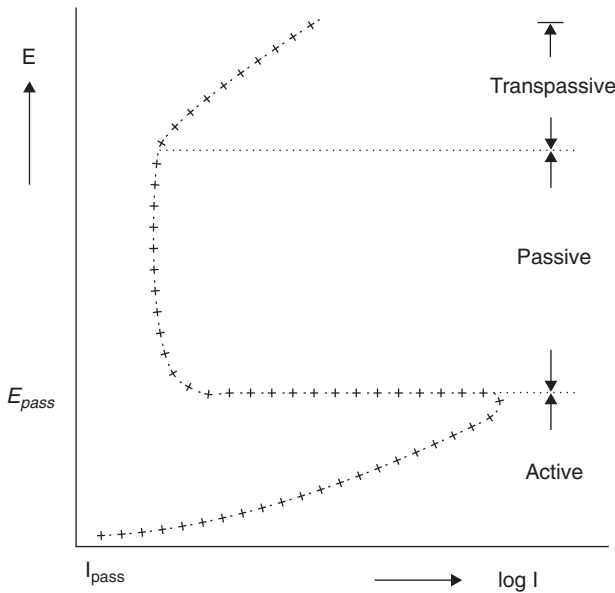


2.9 Reduction process where mass transport restricts the rate.

through water to the metal and if electron transfer at the interface is very fast all OX particles that arrive at the metal surface will be reduced immediately. The magnitude of the current density is then determined by the diffusion rate of OX through the boundary layer at the metal surface. The boundary current density I_{bound} is independent of the potential, as is illustrated in Fig. 2.9. The partial current–potential lines of a corrosion system that belongs to this type is shown in Fig. 2.10. A well-known example is the corrosion of steel in aerated, neutral water ($\text{pH} \approx 7$), in which the oxygen dissolved in the water acts as OX. In many cases in practice, precipitation of salts, lime or fouling of marine life on the steel surface can also inhibit OX transport to the metal surface and thus have a similar effect.



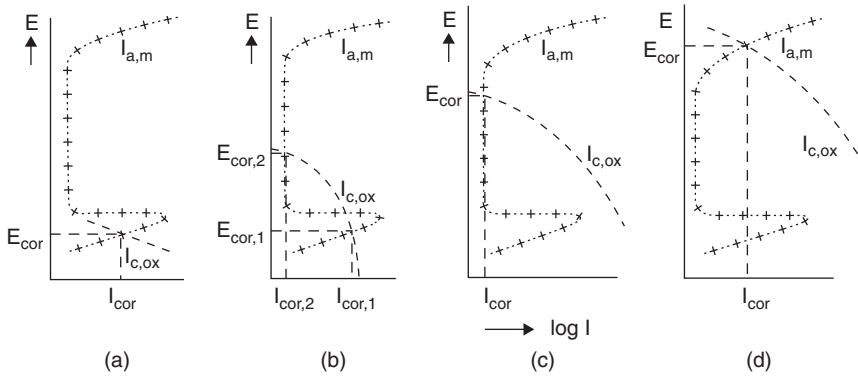
2.10 Current-potential lines of corrosion system of which the mass transport is restricting the rate.



2.11 Example of anodic polarization curve of passivating metal.

2.2.5.3 Passivity

A special case of a mass transport determined corrosion rate is the phenomenon of passivity. Beyond a certain potential, called the passivation potential, E_{pass} , an extremely dense metal oxide layer is formed in a number of metals and alloys. This layer prevents further oxidation or limits it to an extent that it is negligible (see Fig. 2.11). Passivation only occurs in aerated systems. In such a case it is not oxygen transport that is restricted, but metal ion transport. For instance, ferro ions, Fe^{2+} cannot pass the passive layer of steel reinforcement in concrete. The passive layer is often extremely thin, i.e. in the range of nanometres (10^{-9}m).



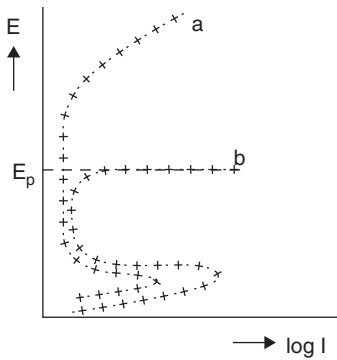
2.12 Examples of corrosion systems of which the anodic current of the metal is partially in the passive area: (a) corrosion potential in active area (b) unstable either in active $E_{cor,1}$ or in passive $E_{cor,2}$ area (c) stable in passive area (d) in transpassive area.

Known metals that passivate are Cr, Ni, Ti, Al and stainless steel. Steel in concrete passivates and is well protected against corrosion.

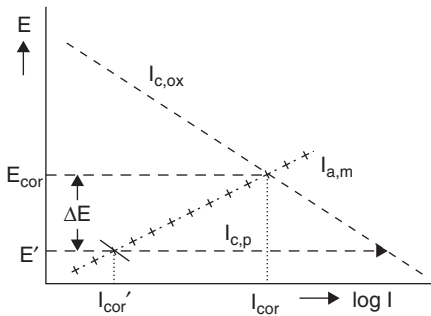
Current-potential curves are shown in Fig. 2.12. Various situations are possible. In case a, the corrosion potential, E_{cor} , is not in the passivated area and the system corrodes actively. In case b there are two intersections. At one of those intersections the situation is similar to that of case a. At the other intersection the metal is passivated. The latter passivation is unstable and can easily change into active corrosion, for instance when the surface is scratched. The worst case occurs where part of the surface corrodes actively and another part is passivated. The reduction of OX in the passivated area then reinforces the corrosion process at the active corrosion sites and this locally accelerated corrosion can be hazardous. For further discussions on this subject, see Section 2.3. Case c is the ideal situation with stable passivation. In case d there is no passivation and corrosion will occur.

To maintain the passive layer, a certain quantity of oxygen is required. If the availability of oxygen drops below a certain point, the situation can change from case c into b or a. A well-known example of this is crevice corrosion, as discussed in Section 2.3.2.5.

It is not only the OX reduction curve that can change, the metal oxidation curve can change also. Chloride ions are notorious for their local change of passivation into active corrosion, while other areas remain passivated (see Fig. 2.13). This causes rapidly corroding cells locally and is referred to as pit corrosion. This is by far the most important degradation mechanism in civil engineering structures manufactured from passivating materials.



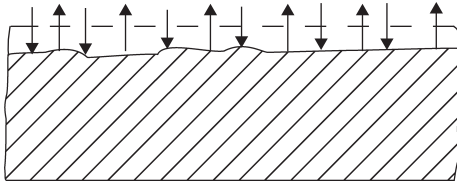
2.13 Polarization curve of: *a* passivated metal, *b* the same metal but in the presence of chloride ions.



2.14 Artificial shift from E_{cor} to E .

2.2.5.4 Deliberate shift in potential

In all corrosion systems, a decrease in corrosion rate can be achieved by decreasing the potential (Fig. 2.14). At this decreased potential, the oxidation current, $I_{a,m}$ is no longer compensated by the reduction current $I_{c,ox}$. The difference in current, $I_{c,p}$, has to be supplied to the metal externally. This method is referred to as cathodic protection, because the artificial shift to the lower potential causes the cathodic or reduction reaction to dominate. Cathodic protection is extensively applied to metal structures that are non-passivating or show unstable passivating. This will be discussed further in Section 2.5.5. From the foregoing it appears that the corrosion process not only depends on the metal, but also on the environment. Both have to be taken into account when considering the necessity of maintenance and preventive measures.



2.15 Uniform corrosion.

2.3 Corrosion types

As shown in the previous chapter, corrosion is a rather complex process involving numerous factors. It is, therefore, not surprising that there are many types of corrosion.

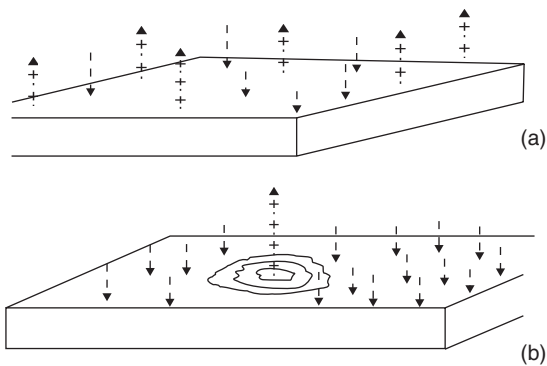
2.3.1 Uniform corrosion

The most common form of corrosion is uniform corrosion, as illustrated in Fig. 2.15. The metal is homogeneously affected at the surface and the anodic and cathodic reaction sites are uniformly spread over the metal surface. The rate of uniform corrosion is usually low and predictable. This makes the corrosion process manageable. An example of such a manageable corrosion rate is the application of unprotected steel sheet-pile walls, where the corrosion process is incorporated into the service life design and part of the steel is sacrificed, see for instance Section 6.8.4. Another example is the corrosion of steel reinforcement in concrete due to carbonation, as discussed in Section 3.4.2.

2.3.2 Local corrosion

2.3.2.1 General

In cases other than uniform corrosion, deterioration is more localized. This is due to non-homogeneities of the metal or of the environment. These non-homogeneities can occur on a micro-scale level. However, they can also occur on the macro-scale level, for instance when a metal is in contact with another metal or when part of the metal is aerated and part is not. The characteristic of localized corrosion is that there is a fixed anodic site where the metal is affected, with adjacent to it a cathodic area where reduction of OX occurs as illustrated in Fig. 2.16. Whatever the type of corrosion, the anodic and cathodic current will, of course, be equal ($I_a = I_c$), but often the cathodic area is much larger than the anodic one, causing high current densities at the anode and consequently a high corrosion rate at that location. In general, the effects of this kind of local corrosion are much worse than the



2.16 (a) uniform and (b) local corrosion.

uniform corrosion discussed in the previous section. Its intensity is difficult to predict and to control.

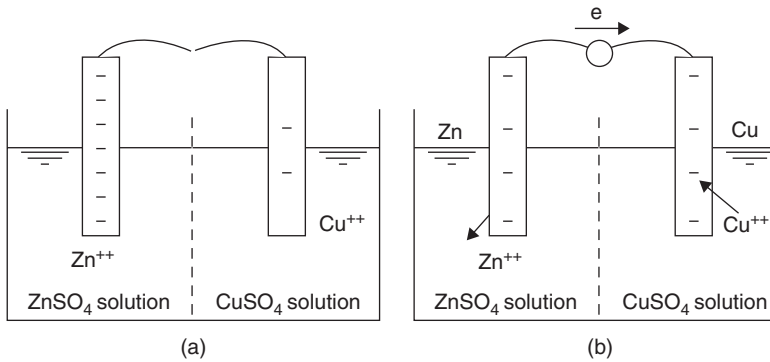
2.3.2.2 Galvanic corrosion

Galvanic corrosion (French: corrosion galvanique, German: galvanische Korrosion, Dutch: galvanische of contact corrosie). When two metals are in contact with each other there will be a potential difference between them. In the case of zinc in contact with copper, the copper is the less reactive metal. According to Table 2.1, the standard potentials are: $E_{0,\text{copper}} = +0.34 \text{ V}$ and $E_{0,\text{zinc}} = -0.76 \text{ V}$.

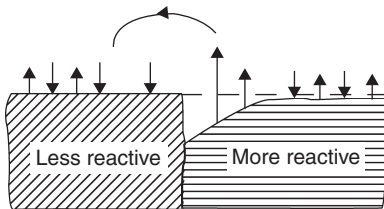
These metals can be short-circuited as in the Daniell cell shown in Fig. 2.17 with the copper electrode in contact with a copper sulphate solution, the zinc electrode in contact with a zinc sulphate solution and the two solutions in electrical contact through a semi-permeable wall. Here there will be a current where the zinc is the anode and is oxidized, and the copper ions are reduced and precipitate on the copper electrode, the cathode. The potential difference V is:

$$V = E_{\text{copper}} - E_{\text{zinc}} = E_{0,\text{copper}} - E_{0,\text{zinc}} + \frac{RT}{2F} \ln \text{Cu}^{2+} - \frac{RT}{2F} \ln \text{Zn}^{2+} \quad [2.8]$$

In time, the potential difference will decrease because the copper ion concentration, Cu^{2+} , will decrease and the zinc ion concentration Zn^{2+} will increase. The corrosion current not only depends on the potential difference, but also on the ohmic resistance between the electrodes. In civil engineering practice, metals are mostly in direct contact, as illustrated in Fig. 2.18. The less noble metal will corrode. The electrical circuit goes through the metals and the water film at the metal surface. The current density also depends on the resistance of the electrical circuit, which, in turn, is mainly



2.17 Copper and zinc electrodes in salt solutions (Daniell cell): (a) non-shortcut (b) shortcut.



2.18 Galvanic or contact corrosion.

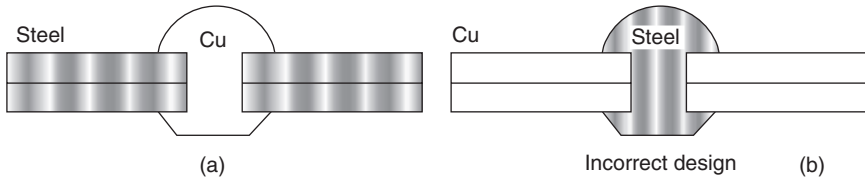
due to the resistance of the water film. Corrosion will be most severe directly at the interface between the two metals.

If zinc and copper are in direct contact, the zinc will corrode, but the reducing reaction will not result in copper ions being converted into copper. Almost immediately, the copper ion concentration will drop causing E_{copper} to become very small and another redox system will take over, in this particular case the reduction of oxygen. Copper then offers only an additional surface to support the reduction of H^+ ions.

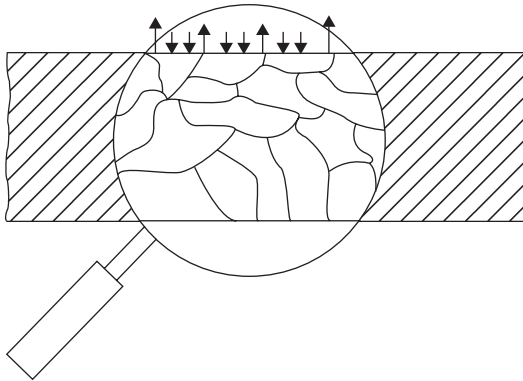
In practice, the relative surfaces of the anodic metal and the cathodic metal are of great importance. For instance, if a copper bolt is in contact with a steel sheet, as shown in Fig. 2.19 a, the cathodic/anodic area quotient is rather low. The local corrosion rate is, therefore, also low. If we have the opposite situation, as shown in Fig. 2.19 b, the quotient is high and the bolt will fail very quickly.

2.3.2.3 Intergranular corrosion

Intergranular corrosion (French: corrosion inter cristalline, German: interkristalliner Korrosion, Dutch: interkristallijne corrosie) is due to a less



2.19 Galvanic corrosion: (a) low rate (b) high rate.



2.20 Intergranular corrosion.

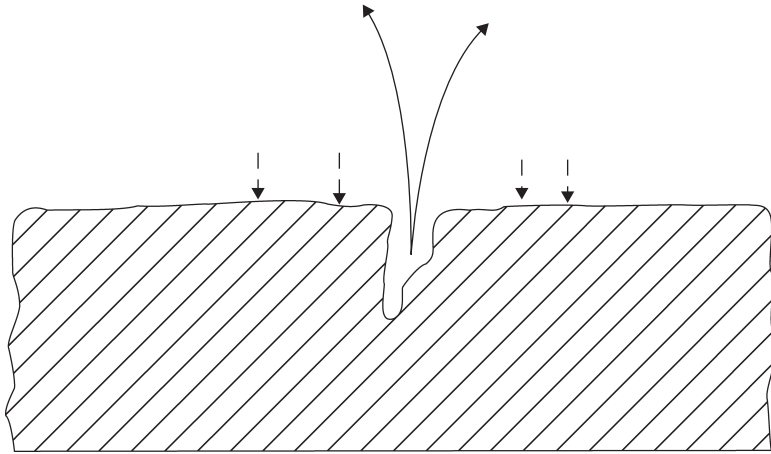
favourable microstructure, as illustrated in Fig. 2.20. The anodic areas are located along the grain boundaries while the crystal planes are acting as cathodes. Corrosion can penetrate into the metal along the grain boundaries and grains can lose their bonds with adjacent grains.

The anodic areas can be the result of local separations within the metal. A well-known example is the precipitation of chromium carbides at the grain boundaries in various austenitic steels, where the alloys stay too long in the 500–800°C temperature area due to welding. Because of the phase separation, there are areas at the grain boundaries that lack chromium and will no longer passivate. This phenomenon is of great importance in welding where this kind of phase separation can easily occur.

Many aluminium alloys are susceptible to intergranular corrosion as well. The heating regime during manufacturing is also of great importance with respect to this type of corrosion.

2.3.2.4 Pit corrosion

Pit corrosion (French: corrosion par piquûres, German: Lochfrass, Dutch: putcorrosie) is a type of highly localized deterioration where pits develop with a diameter that is generally small in comparison to depth (see Fig.



2.21 Pit corrosion.

2.21). This phenomenon is caused by pores in the layer that protects the metal against corrosion. In general, this concerns passivation layers but pit corrosion is also observed under cracks in coated metals. The pit is the anode, while the environment that is still protected can act as a cathodic area. Because the pits are only present locally there exists a large cathodic/anodic surface area. This causes a high current density at the anode and consequently fast corrosion in the pit.

Progress of the corrosion process depends on the type of protective layer.

1 Non-conductive layers

At locations where a continuous pore extends to the surface of the metal, corrosion will occur, e.g. enamel on steel or organic coatings on steel. If this happens in a neutral or alkaline environment, rust will develop in the pore, which slows down the corrosion process. In acid environments, corrosion products are transported further away and removed from the pore. Consequently, the corrosion process will proceed unhampered and rapid pit formation can occur.

In the Netherlands, a well-known example is pit formation under the cracked organic high solids epoxy coating on barriers along the North Sea coast. Figure 2.22 shows cracks of such a coating on the steel barrier doors of the Eastern Scheldt barrier. The cracks bridge the full depth of the coating and beneath the crack the steel shows pit corrosion. In a chloride-rich environment, corrosion products are the result of the complexing character of the chlorides transported easily outside of the pore which therefore do not hamper the corrosion process. See Section 2.5.3 for further details of organic protection systems.



2.22 Cracks in high solids epoxy coating of Eastern Scheldt barrier in the Netherlands beneath which pit corrosion is present.

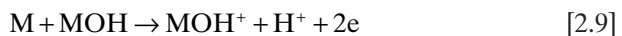
2 Metallic layers

There are two types of metallic layer:

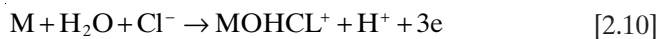
- The protection layer metal is less noble than the substrate metal, e.g. zinc on steel. In such cases, pores or other defects do not cause pit corrosion. At the location of the defect, zinc offers cathodic protection to the steel as long as electrical conductance over the defect is warranted.
- The protection metal is more noble than the substrate metal, e.g. tin (stannum) on steel. In the case of defects, the defect will act as an anode and the surrounding less reactive protection layer as a cathode. Fast pit corrosion occurs.

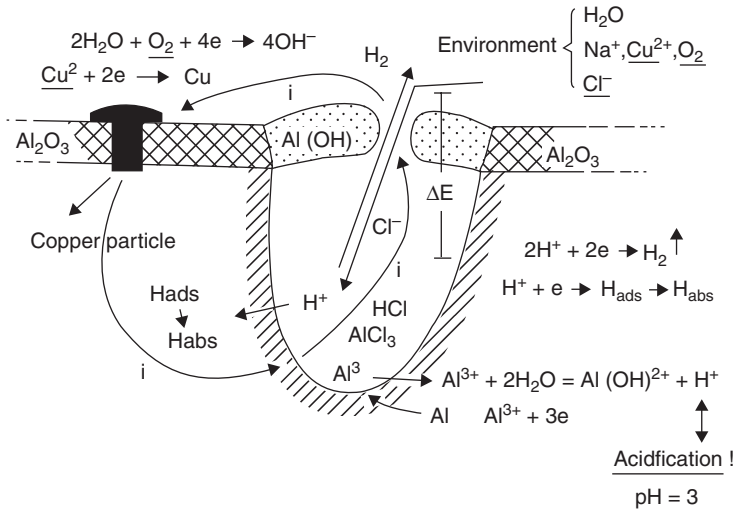
3 Passive metal oxide layers

Metals that owe their corrosion resistance to a passive oxide layer, such as stainless steel and aluminium and steel in concrete, can be subject to pit corrosion when in contact with water containing chloride. This threat exists for structures in marine environments and in colder climates in the case of contact with de-icing salts. The process starts with needle-type holes that deepen and widen rapidly. At the surface, the pit is often covered with a porous metal oxide layer. In the case of steel, this is rust. There is no generally accepted model for the initiation of defects in the passive layer. One of the models available shows that chloride ions are adsorbed at locations with imperfections. Anodic reactions can then take place at the 'poisoned' locations. Reactions such as:



and





2.23 Pit corrosion of aluminium due to copper particles.

will occur at the pit, which in turn leads to local acidification and attracts chloride ions from the environment so as to maintain electrical neutrality. This will hamper repassivation of the area. In most cases, the passive metal oxide layer is conductive, thus allowing for a cathodic reaction around the pit, e.g. oxygen reduction. Since the cathodic area is generally huge in comparison to the anodic area, the current density in the pit can be very high. This happens, for instance, when chlorides attack stainless steel or steel in concrete.

In pure aluminium, the passive aluminium oxide layer is not conductive, so the reduction reaction at the surface proceeds very slowly. Pit formation of pure aluminium in contact with chloride advances slowly. Less pure aluminium often reacts differently and conduction can be much better! In some products, an aluminium alloy exhibiting better mechanical properties than pure aluminium is clad with pure aluminium to enhance corrosion resistance.

Pit corrosion in passivated metals can also be caused by the presence of inclusions, such as sulphides in stainless steels or precipitation of less reactive metal particles at the surface (for instance, copper particles on aluminium, as shown in Fig. 2.23).

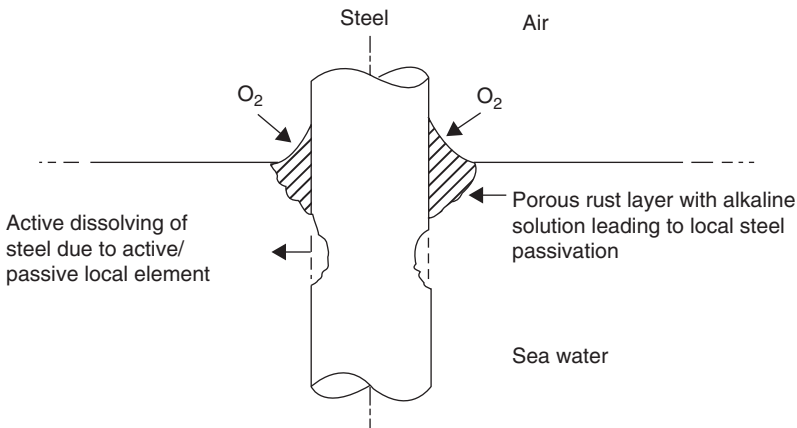
2.3.2.5 Crevice corrosion

A mechanism similar to pit corrosion occurs in crevice corrosion (French: corrosion dans les crevasses, German: Spaltkorrosion, Dutch: spleetcor-

rosie). As the name suggests, corrosion takes place in crevices, for instance between bolts and nuts, between O-rings and their surroundings and between masonry and stainless steel lintels. In crevices where the oxygen concentration drops to a very low level, the oxidation current density and the reduction lines generally no longer intersect in the passivation area (Fig. 2.12). The area in the crevice acts as an anode. Outside the crevice, passivation is intact and this area functions as a cathode. As in the case of pit corrosion, the environment within the crevice will gradually become more acidic and develop a higher chloride concentration. Crevice attack can also occur below deposits of dirt and salts and is then referred to as deposit attack.

2.3.2.6 Waterline attack

Waterline attack (French: corrosion à la ligne d'eau, German: Wasserlinien Korrosion, Dutch: waterlijncorrosie) can occur in stagnant water, just below the water surface. In this case also it is the result of a difference in aeration. Figure 2.24 illustrates this phenomenon. At the water line, water forms a meniscus at the metal surface. Through this meniscus rapid and regular transport of oxygen to the metal surface is possible. In the case of, for example, a non-alloyed steel below the water surface, the steel will initially corrode homogeneously. However, oxygen availability in the meniscus is more abundant through the water film and the environment is favourable to a cathodic reaction. Iron hydroxide and oxides precipitate. Oxygen reduction makes the environment in the meniscus more alkaline (see equation 2.3), which can lead to local steel passivation. The effect of the cathodic area in the meniscus predominates and corrosion is most severe just below



2.24 Waterline attack.

the water line. Due to the ohmic resistance of the electrolyte, the corrosion rate decreases the greater the distance from the surface. Gradually, the corrosion rate diminishes to a level determined by mass transport of oxygen, I_{bound} (see Section 2.2.5.2).

2.3.2.7 Filiform corrosion

Filiform corrosion (French: corrosion filiforme, German: Filgrankorrosion-fadenformige Korrosion, Dutch: draadvormige corrosie) can occur on coated steel and aluminium. It has no bearing on the structural safety of buildings and is mainly a problem of appearance.

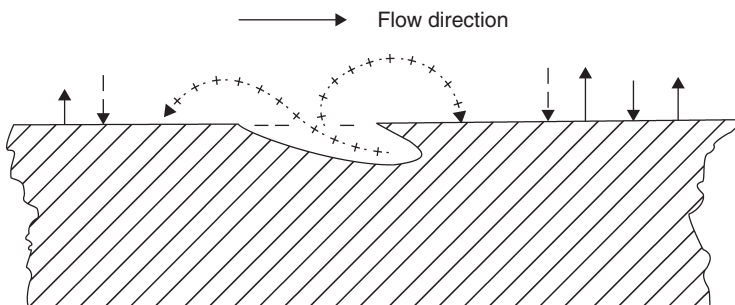
2.3.2.8 Erosion corrosion

Erosion corrosion or cavitation corrosion (French: corrosion erosion or corrosion cavitation, German: Erosion Korrosion, Dutch: erosie corrosie or cavitatie corrosie) can occur in flowing water, especially at locations of severe turbulence, e.g. at restrictions or bends. The protective passivation layer is removed due to erosion, as illustrated in Fig. 2.25. The result is enhanced corrosion at those locations.

2.3.2.9 Biological corrosion

Biological corrosion (French: corrosion biologique, German: biologische Korrosion, Dutch: biologische corrosie) can be caused by life forms proliferating at the metal surface. It is observed at pH values ranging between 0 and 11 and at temperatures between 0 and 80 °C in freshwater, sea water and soils.

Micro-biological corrosion cause by anaerobic sulphate-reducing bacteria in the soil is well-known. These bacteria can produce sulphide which



2.25 Erosion corrosion.

accelerates steel corrosion. Such corrosion can also contribute to hydrogen embrittlement in prestressed steel. Bacteria that form sulphuric acid, which also accelerates corrosion, can oxidize sulphur compounds in aerobic environments. This phenomenon can, for instance, occur in oil pipes and sewers and can result in rapid pipe failure. Another type of biological corrosion is due to macro-organisms living in water, such as fungi that produce acids such as acetic acid and increase the corrosion rate. Algae, barnacles and mussels adhere to the metal and cover the metal surface. This is called fouling and leads to differential aeration and attacks of the crevice corrosion type.

2.3.2.10 Stress corrosion cracking

Stress corrosion cracking (French: corrosion fissurante sous tension, German: Spannungsrissskorrosion, Dutch: scheurvormende spanningscorrosie) can occur in almost all construction metals. In the case of specific environmental conditions, even relatively innocent tensile stresses (stresses lower than the limit state values) can lead to failure.

Material loss in the corrosion process is minor and cannot usually be observed with the naked eye. Because of this, it is difficult to inspect and failure can occur without warning. A well-known case is the failure of stainless steel straps of hanging ceilings in swimming pools (see Section 2.4.3). Another case is the occurrence of stress corrosion cracking in prestressed or post-tensioned steel cables in prestressed concrete. A condition for stress corrosion cracking is the presence of tensile stresses in the metal. It is not the average tensile stress that is important, but the local tensile stress in the metal in the area where the crack is initiated or where it continues. In general, this occurs at defects, such as pits, scratches, or at an already existing crack tip. Such a local stress state is characterized by means of a stress intensity factor K .

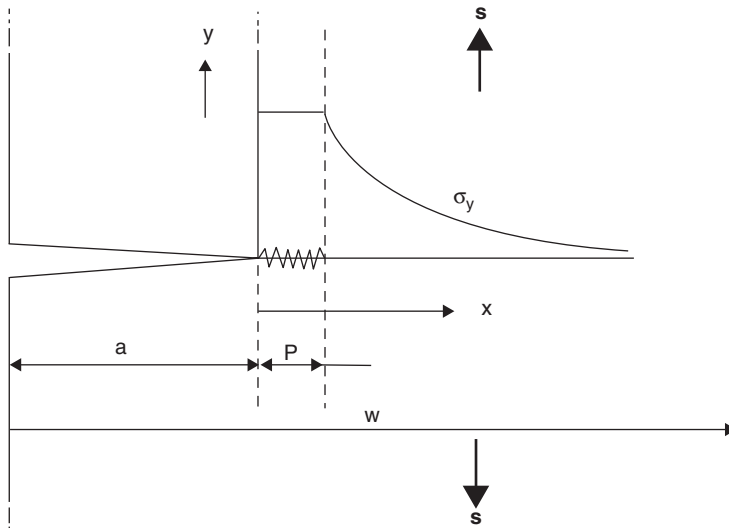
Stress distribution at a crack tip is illustrated in Fig. 2.26. A crack with depth a proceeds due to a tensile stress with an average size S . Stress distribution in the vicinity of the crack can be determined by means of stress mechanics:

$$\sigma_y = \frac{K_I}{\sqrt{2\pi x}} \quad \text{if } x \geq p \quad [2.11]$$

where: p = the size of the area in direction x , which shows plastic deformation when exceeding the yield stress

K_I = the stress intensity factor I indicates that the stress is perpendicular to the crack plane

In the case considered here:



2.26 Stress distribution in the vicinity of a crack tip.

$$K_I = S\sqrt{\pi a} \cdot f(a/w) \quad [2.12]$$

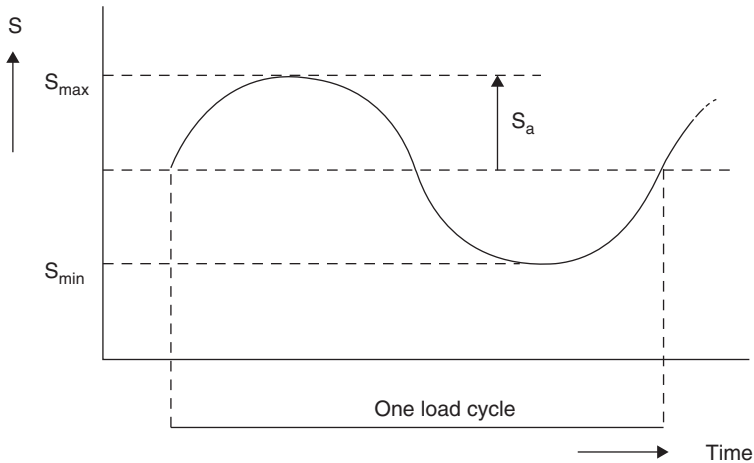
where: $f(a/w)$ = a function of a/w

If the crack proceeds, K_I can exceed a critical value (K_{Ic}) beyond which the crack opening displacement rate will increase rapidly. K_{Ic} is lowered by corrosion. Consequently, crack propagation will proceed at lower stresses where there is corrosion than in those cases where there is none.

For engineering structures attention should be paid to stress corrosion cracking of austenitic stainless steels (see Section 2.4.3) and aluminium in chloride-containing environments. For unalloyed and low-alloyed steel, stress corrosion can occur in special environments such as in the case of contact with nitrate fertilizers. The sensitivity for stress corrosion occurs at increasing concentrations and temperatures.

2.3.2.11 Hydrogen embrittlement

Hydrogen embrittlement (French: fragilisation hydrogène, German: Wasserstoff Versprödung, Dutch: waterstofverbrossing) can occur if metals are under stress and hydrogen atoms are diffused from the surface into the metal. Hydrogen can be formed by reduction of H^+ . The metal is weakened by hydrogen that has penetrated which will result in brittle failure. If hydrogen atoms are formed, they normally recombine to form hydrogen



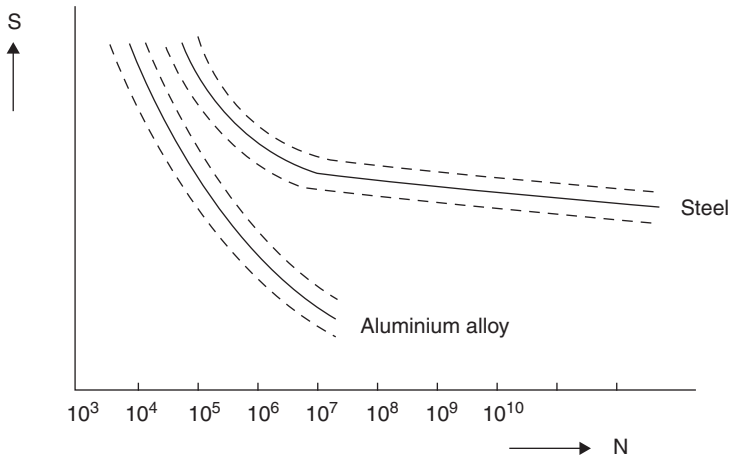
2.27 Sinusoidal dynamic loading pattern.

molecules but this recombination is ‘poisoned’ in some environments and sulphides are notorious examples of this type of poisoning. Welding can be another cause of hydrogen embrittlement. Hydrogen embrittlement is occasionally put forward as the cause of steel cable failure in structures, but there is little proof of this. In general, high strength steels are more sensitive to hydrogen embrittlement than are lower strength steels.

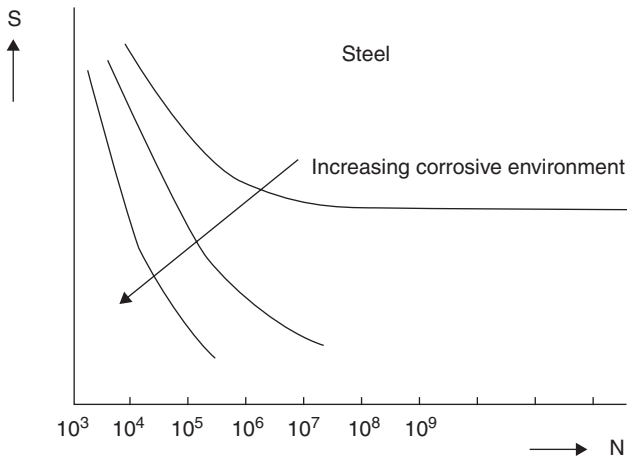
2.3.2.12 Corrosion fatigue

Corrosion fatigue (French: fatigue sous corrosion, German: Schwingungskorrosion, Korrosionermüdung, Dutch: corrosie-vermoeing) can occur when metals are subjected to dynamic loading. They fail at lower stresses than when loaded statically. This decrease in loading capacity is larger in a corrosive environment. In the case of sinusoidal loading, shown in Fig. 2.27, we can find a relationship between load S ($S_{\max}$ or S_a) and the number of load cycles N , as shown in Fig. 2.28. The load-bearing capacity stabilizes in steel beyond a certain number of cycles. There is no such boundary limit for aluminium. The effect of a corrosive environment is illustrated in Fig. 2.29. In such a corrosive environment, load cycle frequency appears to be important, as shown in Fig. 2.30.

In practice, structures showing corrosion fatigue are designed according to the fail-safe principle. It is thus assumed that metal construction parts always have defects, such as inclusions, hair cracks due to welding and corrosion pits, where fatigue cracks will initiate. Service life is then determined by measuring the progress of crack propagation under practical conditions.

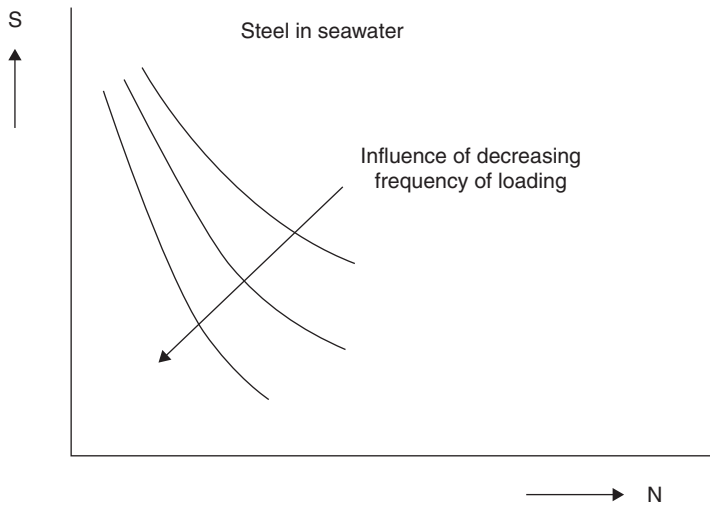


2.28 Load-cycle number (S-N) curve for steel and aluminium alloy in air.



2.29 Effect of a corrosive environment on the S-N curve.

Corrosion fatigue of metals can occur in a low corrosive medium, e.g. water, provided the stress level is substantial. Steel bridge decks are vulnerable to corrosion fatigue cracking. The frequency and the amplitude of loading appear to be unfavourable and especially high axle loading should be avoided. The probability of fatigue corrosion cracking can be decreased by the following measures: applying a non-corroding steel, e.g. a stainless steel, reduction of the amplitude of the dynamic loading, e.g. by increasing the stiffness, avoiding crevices where corrosive compounds can accumulate,



2.30 Effect frequency in dynamic loading on the S–N curve in a corrosive environment.

passive protection with an organic coating or active protection with metallic coatings or cathodic protection.

2.3.2.13 Fretting corrosion^{2,3}

Fretting corrosion (French: fretting-corrosion, German: Reibkorrosion, Dutch: fretting) can aggravate fatigue corrosion cracking substantially. It is due to moving surfaces of metals in contact with each other. Critical movements are in the range of $0.1\text{--}300\mu\text{m}^2$. Some metal is removed from the contacting surface and fast corrosion occurs. Examples of cases where fretting has occurred are in prestressed cables and tendons. Also the anchoring of post-tensioned steel is known to be vulnerable to fretting. Measures to prevent fretting include the use of lubricants, coating with aluminium or zinc or full protection of the steel against a corrosive environment.

2.4 Metal corrosion in practice

2.4.1 Structural steel

Steel is iron with an approximate carbon content of less than 2% and lower percentages of silicon. Cast iron has a higher carbon content. Structural steel contains less than 0.25% of carbon.

2.4.1.1 Atmospheric corrosion

Structural steel corrodes when exposed to air. At the cathode, hydrogen is formed. In general, reduction of H^+ ions dominates in atmospheric corrosion because the atmosphere is slightly acidic and there is an adequate water film present at the metal surface to maintain an electrochemical current. The time of exposure to wetness of the steel surface is of particular interest with respect to corrosivity of the atmospheric environment. Climates are occasionally so dry that this film is absent and corrosion of bare steel and cast iron is negligible. A well-known example is the cast iron statue in New Delhi, India that has stood for over a hundred years. However, due to air pollution, the corrosion rate has increased steeply over the past few decades.

Salt and dust particles can promote condensation of moisture. If precipitated on metal surfaces, salt and dirt particles can locally increase the moisture conditions at the surface and decrease the electrical resistance of the water film. Consequently, the corrosion rate will increase. For instance, SO_2 is oxidized into sulphate and the iron sulphate solution is highly conductive. Another negative effect of these contaminants can be a change in rust layer morphology. SO_2 and the chloride content in the air largely determine corrosivity of the environment and its effect on structural steel. The SO_2 concentration in Western countries increased steadily during most of the twentieth century, but has fallen since the 1980s due to tough emission limits imposed on industry. Due to the concurrent decrease of dust particles in the air, the corrosivity of the atmospheric environment has decreased over the past twenty years. Chloride in the air is bound to aerosol particles and is restricted to maritime areas; it decreases substantially within the first 10 kilometres from the coast.

Metal corrosion rates are given in Table 2.2.

Table 2.2 Metal corrosion rates in Western Europe

Atmospheric environment*	μm corrosion per year				
	Al	Pb	Cu	Zn	Steel
Industry	0.7	0.7	1.3	1–10	100–140
City	0.8	0.4	1.3	0.5–1	40–80
Maritime	0.7	0.5	1.4	1–5	150
Land	0.05	0.4	0.5	0.2–0.5	40–60

* Western Europe

2.4.1.2 Corrosion of structural steel in water

The corrosion rate of steel (French: acier, German: Stahl, Dutch: staal) in neutral water (drinking water, seawater, most surface water) is determined by the cathodic reaction rate:



The rate of this reaction depends on the time required for the water-based oxygen gas to reach the metal surface. This process is determined by the diffusion rate and can be calculated by means of the first Fick's law:

$$\Delta d = 14.3 \frac{C_{\text{O}_2}}{\delta} \quad [2.14]$$

where: Δd = decrease in metal thickness in μm per year

C_{O_2} = concentration of oxygen dissolved in mg/l water

δ = thickness of the water layer over which the transport has to proceed by diffusion

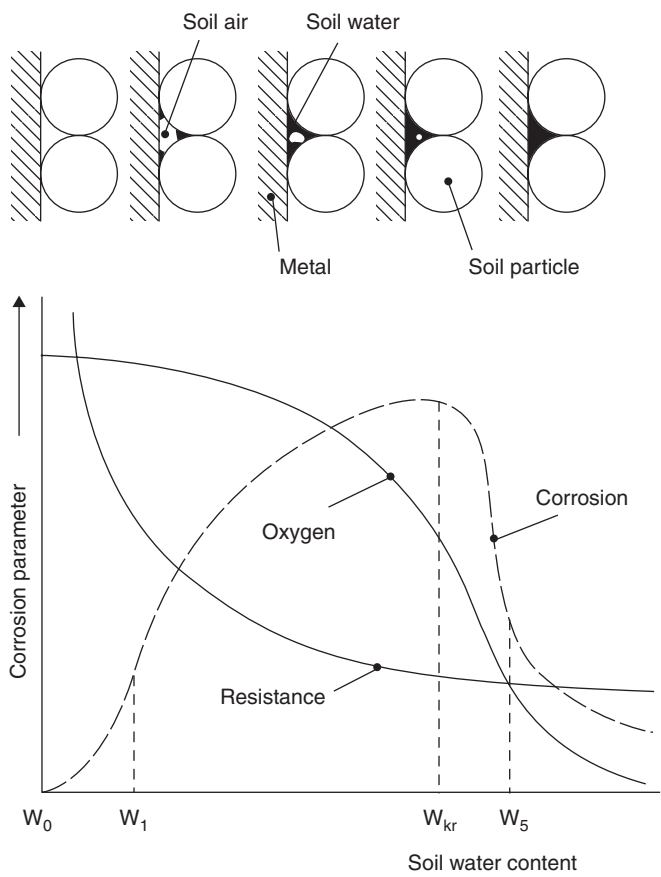
2.4.1.3 Underground corrosion

Underground corrosion processes are similar to those occurring in the atmosphere or in water. The actual rate of corrosion depends strongly on the water content and on the salts dissolved in the water. In Fig. 2.31 typical corrosion parameters are shown as functions of the water content of the soil.⁴ In water-saturated soil and in dry soil the corrosion can be neglected. Between these two extremes corrosive situations can arise. In sandy soils air is usually sufficiently available to maintain the corrosion process and often a kind of pit corrosion is observed. Clay soils often have high water content and contain less air and so corrosion is restricted. Dissolved salts increase the conductivity of the soil and increase the corrosion rate. The electric conductivity is considered an important criterion for the corrosivity of soils.

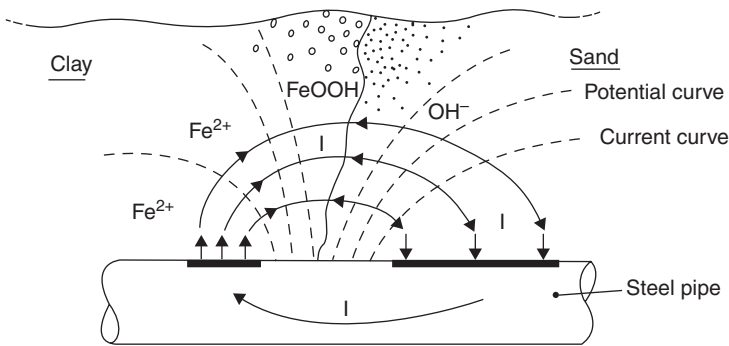
In the case of steel going through different types of soil or in the same soil but with layers of different levels of consolidation a macro cell can be formed with fast corrosion at the anode. Figure 2.32 shows a steel pipe passing through a sand and clay layer.⁵ The steel in the well-aerated sand is acting as the cathode, while the part in the clay acts as the anode. In non-aerated soils sometimes hydrogen sulphide, (H_2S), is present which in contact with steel is highly corrosive.

2.4.2 Corrosion of cast iron

Cast iron (French: fonte, German: Gusseisen, Dutch: gietijzer) can be cast into intricate shapes because of its excellent fluidity at relatively low



2.31 Soil resistance, oxygen content and corrosion rate as functions of the water content of the soil.⁴



2.32 Macro-cell in non-homogeneous soil.

melting point. It was used extensively for construction purposes in the nineteenth century and the early part of the twentieth century. Today, its use in structural engineering is restricted to specific purposes. There are various types of cast iron: white, grey, malleable and nodular. They all contain less than 3% of carbon and silicon. In general, the corrosion resistance of cast iron is better than that of structural steel. With proper alloying, the resistance can be as good as that of stainless steel.

Cast iron shows all the types of corrosion discussed in Section 2.3. One type is graphite corrosion which is observed in grey cast iron. Selective leaching of iron takes place because graphite is cathodic to iron and so a graphite network is left, looking like a sponge under the microscope. Frequently, no dimensional change is observed for this type of corrosion and hence no advance warning is given, which can lead to unexpectedly hazardous situations. By alloying, cast irons can be made more corrosion-resistant. High-silicon cast irons with more than 14% of silicon show an excellent resistance to corrosion that can even be improved in environments containing chloride by further alloying with molybdenum.

2.4.3 Stainless steels

Stainless steels (French: *acier inoxyable*, German: *Edelstahl rostfrei*, Dutch: *roestvast staal*) are iron-based alloys containing at least 10.5% of chromium, Cr. There are many stainless steel alloys available. For every environment there are one or more stainless steel alloys that are resistant. On the other hand, not every stainless steel is resistant in any environment. The selection of a stainless steel grade for a particular application involves consideration of the environment in question as well as the grade's resistance to that environment.

Stainless steels are categorized in five distinct families according to their crystal structure. Within each family, there is a range of grades that varies in composition, corrosion resistance and cost. These families are: the ferritic, the martensitic, the austenitic, the duplex and the precipitation hardening stainless steels. The best-known system for designating stainless steels is that of the American Iron and Steel Institute (AISI) numbering system. Another system is the Unified Numbering System (UNS). In Europe, the designation according to EN 10088 is used increasingly.

Proper stainless steel grades do not corrode due to steel's passivity under atmospheric conditions. As long as passivity is maintained, stainless steel will perform for a very long time with minimal corrosion. However, an inadequate grade can corrode more rapidly than plain carbon steel and will fail by uniform corrosion.

In the construction industry, the austenitic grade AISI type 304 (EN 1.4301) is the most commonly used but this grade is not corrosion-resistant



2.33 Concrete ceiling that collapsed in a swimming pool due to chloride-induced stress corrosion of stainless steel hanging strips in Uster, Switzerland, 1985.

in a chloride environment. It shows pit corrosion and can suffer stress corrosion. Improved resistance against corrosion is usually obtained by alloying with molybdenum, yet many of those alloys are still prone to stress corrosion, such as the much applied AISI 316 (EN 1.4401) stainless steel, which is prone to chloride-induced stress corrosion cracking, notably in the absence of regular cleaning, either intentionally or by rain.

In Uster, Switzerland, a concrete ceiling came down in a swimming pool killing 12 people in 1985 (see Fig. 2.33). The ceiling was fixed to the roof with strips of AISI 304 stainless steel. Vapour containing chloride condensed on the strips and caused stress corrosion cracking. Similar failures occurred in the Netherlands, fortunately without loss of life. These problems have prompted stricter rules on the use of stainless steels in structural applications. An example of stainless steel lintel grades proposed for various natural environments in the Netherlands is given in Table 2.3. Table 2.4 shows stainless steel recommended for structural applications in indoor-swimming pools as detailed by Nürnberger.⁶

Special care is required for welding stainless steels. Welding can make stainless steel more susceptible to pitting, intergranular corrosion, crevice corrosion and stress corrosion. Welding of stainless steel requires highly skilled personnel and expertise so as to ensure the correct procedure.

Table 2.3 Recommended stainless steel grades for lintels directly exposed to the exterior in the Netherlands

Environment	AISI/UNS	EN no	EN name
<10 km from the coast	AISI 317LMN	1.4439	X2CrMoCuN 25.20.5
	–	1.4529	X1NiCrMoCuN 25.20.7
	AISI 904L	1.4539	X1NiCrMoCu 25.20.5
	UNS 32305	1.4462	–
	AISI 316L ^a	1.4404	–
	AISI 316Ti ^a	1.44571	–
>10 km from the coast	AISI 316	1.4401	X5CrNiMo 17.12.2
	AISI 304	1.4301	X5CrNi 18.10

^a provided design does not allow crevice corrosion and welding is strictly controlled

Table 2.4 Recommended stainless steel alloys for structural elements in an indoor swimming-pool atmosphere without regular cleaning⁶

Domestic water (Cl ≤ 250 mg/l)		
AISI/UNS	EN no	EN name
AISI 317LMN	1.4439	X2CrMoCuN 25.20.5
Chloride-enriched water (e.g. saline water)		
–	1.4565	X2CrNiMnMoNbN 23.17.5.3
AISI 904L	1.4529	X1NiCrMoCuN 25.20.7
–	1.4547	X1CrNiCuN 20.18.7

2.4.4 Weathering steel⁷

Ordinary steel rust does not protect the steel against further corrosion and it is generally accepted that the corrosion continues. Weathering steel (French: acier patinable, German: wetterfeste Stahl, Dutch: weerbestand staal) corrodes but the rust layer becomes a barrier restricting further ingress of moisture and oxygen. A stable condition without any further noticeable corrosion can be reached within a few years. This does not apply to environments where salts can be deposited, notably in marine environments or those which are in contact with de-icing salts. Nor does it apply in strongly chemically-polluted environments nor where construction parts would be continuously wet or damp.

Weathering steel gains its corrosion resistance from the alloying elements, usually chromium, copper and silicon. The total content of these alloys is between 1 and 2.5%. In general, for building construction purposes two grades are used, S235 and S355, according to the standard EN 10155. The rusting of weathering steel, although it decreases in time, should be taken into account in structural design. Allowance for loss of thickness should be given, depending on the environment, from fractions of millimetres up to two millimetres.

With respect to connections with bolts and nuts large crevices should be avoided. Bolts should be of a similar weathering steel alloy. Ordinary high strength bolts should not be used. Zinc plating is sacrificial to weathering steel and would result in rapid severe corrosion. Direct contacts with other metals should be avoided because of galvanic corrosion. The fatigue strength of weathering steel is similar to that for steel. Weathering steel shows some rusting and the rust can stain adjacent materials when rust-laden water runs off. Some materials are prone to severe staining and are difficult or impossible to clean: concrete and stucco, galvanized steel, unglazed brick, stone and wood.

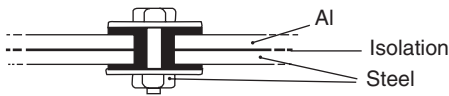
2.4.5 Aluminium

Pure aluminium has very good corrosion resistance due to the protective passive layer and, in general, aluminium alloys are much less corrosion resistant. The environment, alloy composition, temperature treatment and the presence of contaminants are major factors influencing the deterioration rate. The effect of the environment on the corrosion rate is given in Table 2.2. The combination of high SO_2 concentration and dust can be especially detrimental. Surface pollution causes higher moisture contents, better conductivity, and local differences in aeration. Along the coast, the presence of chloride also increases the corrosion rate.

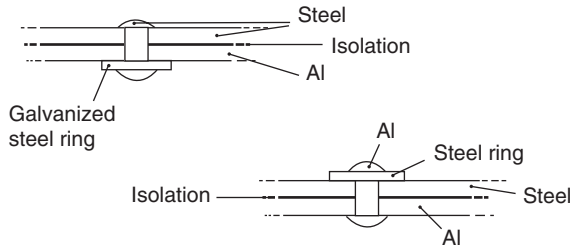
Casting alloys, such as AlSi and AlZn, have good to moderate corrosion resistance. The wrought alloys from the 2000 and 7000 series (AlCu and AlZnMg, respectively), that are more important to the building industry, have poor to moderate resistance. Resistance can, however, be improved effectively by solution heat treatment, although this does affect strength. Aluminium alloys show pitting when attacked.

Aluminium, while being a more reactive metal, is sensitive to galvanic corrosion; in particular, contact with copper must be avoided. Small copper particles that precipitate on aluminium can cause serious trouble. Pitting corrosion of aluminium cladding elements has been observed within 200 m distance from overhead electric railway and tramlines. Even contact with wood impregnated with copper salts must be avoided. Direct contact with steel or iron can also enhance corrosion. In these cases, electrical insulation

(a) Bolt and nut



(b) Rivets



2.34 Electrical insulation between steel and aluminium.

between these metals is required, as shown in Fig. 2.34. However, contact with stainless steel does not result in problems.

Protection of low corrosion-resistant aluminium alloys can be achieved by covering the alloy with pure aluminium or with another less noble alloy, thus providing cathodic protection.

Another protective measure is anodizing. This is an electrolytic oxidation process that produces an amorphous aluminium oxide coating on an aluminium surface that is much thicker than the natural passivation layer. The anodized layer adheres tightly to the aluminium and provides greatly improved protection against corrosion and is a good substrate for colouring the surface. Aluminium protected with organic coatings is often applied. It can, however, be susceptible to filiform corrosion, as discussed in Section 2.3.2.7.

2.5 Protection of metals

2.5.1 General

Structural steel, as well as aluminium alloys, needs protection. We can distinguish between:

- Organic coatings and linings.
- Inorganic coatings.
- Metallic layers.
- Cathodic and anodic protection.

2.5.2 Surface preparation

Surface preparation is a very important aspect of metal protection. Neglecting adequate surface preparation is penalized by early failure of the protective system. Selection of the system used to protect a metal from an environment usually determines the requisite surface preparation.

Some systems are very sensitive to any trace of oil, fat, or remaining rust; other systems are much more tolerant. We will not discuss the sensitivity of the various systems.

In industrial applications complications of a fluctuating environment (weather conditions) can be avoided and in general good quality surface preparation is easier to achieve than is outdoor application. Good quality surface preparation needs skilled labour and should be carried out in accordance with the specifications given in such standards as ISO 12944-4, *Paint and varnishes – corrosion protection of steel structures by protective paint systems. Part 4: types of surface and surface preparation* and applied strictly according to the instructions given by the supplier.

A variety of surface preparation methods are available:

- Mechanical cleaning.
- Solvent and chemical cleaning.
- Cryogenic cleaning (not discussed here).

2.5.2.1 Mechanical cleaning

Mechanical cleaning occurs by applying a force to the surface. It can be done by hand tool, power tool, water blasting and abrasive blasting.⁸

Hand tool cleaning

Hand tool cleaning is used only for removing loosely adherent paint, rust, or mill scale. Since this method is laborious, it is primarily used for spot cleaning. Hand tools include scrapers, wire brushes, abrasive pads, chisels, knives, and chipping hammers. The disadvantages of using these tools, in addition to their slowness, are that they will not remove tightly adherent contaminants and that they may increase burrs or dent the surface, actually causing some damage to the surface. In addition, these tools will not provide a surface profile.

Power tool cleaning

Surface preparation using power tools is faster than hand tool cleaning; it removes loose paint, rust, and scale, and it is good for the preparation of welded surfaces. Power tool cleaning is also good for removing old paint

that has been lifted by rust. It provides a duplication of hand tools in power-driven equipment (electric or pneumatic), such as sanders, wire brushes, grinders, clippers, needle guns, and rotary descalers. Power tools do not leave as much residue or produce as much dust as abrasive blasting does, and they are frequently used where blasting dust could damage sensitive surroundings. However, they may polish the surface too much if used at too high a speed or kept in one spot for too long.

Abrasive blast cleaning

Abrasive blast cleaning is the preferred surface preparation method for paints and coatings that require an anchor pattern and a high degree of cleanliness. Blast cleaning is the only method that can completely remove intact mill scale and give an even roughness with a controlled anchor pattern. Abrasive blasting is the propelling, or shooting, of grit or other types of small, hard particles at a surface. In nozzle blasting, the force that propels the abrasive is compressed air. In the past, silica sand was commonly used; however, because of the risk of silicosis, it has been banned in many countries.

In centrifugal wheel blasting, the cleaning principle is the same. The spinning of large paddle wheels creates the force that throws the abrasive at the surface. This method is usually confined to industrial use. In contrast with open blast cleaning, the abrasives used are recyclable steel grit or shot. Vacuum blasting is another option for dust-free blast cleaning. With this technique, a brush surrounds the blast nozzle, and a vacuum is created within the annular space. The spent abrasive and removed paint, rust, and debris are contained by the brush, removed by the vacuum, and transported to a separator. The debris and fine particles are removed, and the abrasive is returned for reuse.

Water blast cleaning

Water blast cleaning (sometimes called hydro-blasting) may be high- or low-pressure, hot or cold, and with or without a detergent, depending on the type of cleaning desired. This process removes loose flaky rust, paint, and mill scale. It has gained wide acceptance where abrasive blast cleaning, dust, and contamination present a hazard either to personnel or to machinery.

Low-pressure washing (<14 MPa or 2000 psi) is effective in removing dirt, mildew, and chalk from the coating surface. Low-pressure water blast units use the same components as high-pressure water blast equipment. Low-pressure cleaning is referred to as power washing and should be recognized

as such. For the cleaning of steel, water pressures as high as 69 MPa (10000 psi) are used.

Water alone does not etch a metal surface or remove tight paint, rust, or mill scale. Therefore, abrasives can be injected into the water stream to remove tightly adhering materials, to hasten the cleaning or to roughen the surface profile. Any type of abrasive that is commonly used with abrasive blast cleaning can be used in water blast cleaning. Injecting abrasives into the water eliminates the dust that normally accompanies dry use of such abrasives.

Flash rusting is a concern when using these methods. Flash rust is a light rust layer that forms on steel after cleaning, particularly after water or wet abrasive blasting, or in humid marine environments.

2.5.2.2 Solvent and chemical cleaning

Solvent cleaning

Oil and grease can be removed using solvents. Many of the solvents used are volatile, highly flammable and toxic. In general, their use is only permitted in an enclosed working space with natural ventilation, where mechanical exhaust ventilation is present and if vapour concentrations are within legal boundaries during the entire work period. Solvent emissions are increasingly restricted so as to protect the environment.

Chemical cleaning

In chemical cleaning, two types of agent are distinguished: those that do not affect the metal surface and those that do. In the former, use is made of either detergents or alkaline cleaning agents for steel.

Agents that affect the surface are strong acids, such as hydrochloric acid, sulphuric acid and phosphoric acid. This is called pickling (French: *décapage*, German: *beizen*, Dutch: *beitsen*). An inhibitor is usually added to prevent too much removal of the metal. Agents are used to prepare the steel surface. Aluminium is chemically cleaned using alkaline agents.

Steam cleaning

High-temperature high-velocity wet steam is used in steam cleaning to remove heavy soil, oil, or grease. It does not remove rust or mill scale, nor does it etch or roughen the surface. Commercial detergents can be added to the steam to improve cleaning.

2.5.3 Organic coatings

We can distinguish three types of organic protectors of metals:

- Paints or coatings
- Powder coatings
- Linings (these are less relevant for civil engineering structures)

2.5.3.1 *Paints or coatings*

Conversion coatings

A conversion coating is often applied to enhance corrosion resistance and to improve adhesion. Conversion coatings are formed by the reaction of chemical agents applied on the metal surface. The conversion coating usually contains metal ions of the substrate metal. Often the conversion coating is applied directly after cleaning the metal.

The following conversion coatings are used as a substrate for organic coatings.

On steel

- iron phosphate
- zinc phosphate

On zinc

- zinc phosphate
- zinc chromate (not recommended due to harmful environmental effects)

On aluminium

- anodizing (see Section 2.4.5)
- aluminium chromate (not recommended due to harmful environmental effects)
- aluminium phosphate

Organic paints or coatings

The most common method for protecting metals against corrosion consists in applying an organic coating or paint.

Paints comprise the following constituents:

- Binders which are responsible for cohesion of the film and adhesion on the metal surface. To ensure protection against corrosion, binder permeability to aggressive compounds such as salts, is of primary importance. Over the past decade, the binders used have changed a great deal due to environmental and health reasons.
- Pigments fulfil various functions. In prime coatings that are in direct contact with the substrate, corrosion protective coatings are of interest.

Zinc-bearing paints containing zinc dust and ZnO_2 are used extensively. The zinc protects the steel sacrificially if in electrical contact with the steel. Zinc-rich coatings with 92–95% zinc in the dry film have much in common with zinc layers applied by galvanizing as discussed in Section 2.5.4. In top coatings, sheet-type pigments, such as glass or aluminium flakes, can prevent penetration of aggressive compounds. Some pigments used in the past are now banned because of environmental and health reasons, for instance, pigments containing cadmium. Apart from corrosion protection, pigments are also used for aesthetic reasons.

- Solvents and thinners are usually constituents of the unhardened paint only. They evaporate or react with the binder during curing. They provide the required flow ability (viscosity). After applying paint on the substrate, solvents and thinners lose their function. Organic solvents and thinners, like turpentine, ethyl acetate and toluene cause air pollution and, on evaporation, pose a health hazard to the workforce. Governments in various countries have banned the use of these organic solvents.

Paints are often distinguished according to the manner of film formation:

- Physical film formation, with no change in the binder's chemical composition.
- Chemical film formation where chemical reactions occur in the film forming process.

A survey of various ways of film formation of paint binders is given in Table 2.5 and examples of various paint systems used to protect metals in structures are given in Table 2.6.

As a result of growing social pressure to decrease emissions of evaporating solvents and thinners, major developments have occurred in paint technology. These concern:

- The development of water-borne paints, i.e. water is used as the solvent or medium for emulsions (see Table 2.5, 'coagulation').
- High built coatings; these are coating systems with a strongly reduced solvent content.
- Powder coatings. Powder coat systems are generally applied by electrostatic spraying. The powder particles are subsequently melted and then may undergo further reactions, depending on the type of organic compounds applied. Solidification and baking mentioned in Table 2.5 are techniques that can be used for film formation of paint systems applied as powder coatings.

These new paints systems must be investigated thoroughly prior to application in practice as must all new developments. High built epoxy coatings

Table 2.5 Film formation of paints

Type of drying	Explanation
Physical drying	
Evaporation	The binder film is formed solely by drying of solvents and thinners
Coagulation	This occurs with emulsion-based paints (latex paints). Polymer droplets are dispersed in water and when the water evaporates or is absorbed by the substrate, the polymer droplets flow together (coagulate)
Solidification	Molten materials, e.g. bitumen and hot-melted plastics, solidify on cooling
Chemical drying	
Oxidative	Drying under uptake and reaction with oxygen. Occurs in paints containing oils, such as alkyd paints
Two-component drying	A short time before applying a second component, the curing agent or cross-linking agent is added to the resin. The two components react. Film-forming starts and the paint will flow only during a restricted period (minutes to hours)
Baking	Chemical reactions are triggered causing hardening by increasing temperature and/or radiation curing (infrared, ultraviolet, electron radiation etc)

did, for instance, fail on storm surge barriers in the Netherlands, notably the Eastern Scheldt barrier and the Maeslant barrier.⁹ The coatings contained substantial amounts of compounds that were not volatile but evaporated slowly and shrinkage then occurred due to volume reduction; this shrinkage was restrained by the steel substrate. Substantial tensile stresses were built up because of the already developed Young's modulus. In combination with the normal temperature stresses, this caused failure. Restoring the coating systems proved to be extremely expensive, since pit corrosion had also occurred under the cracked coating (see Section 2.3.2.4 and Figure 2.22). Examples of coating systems are given in Table 2.6.

Inorganic coatings¹⁰

Inorganic zinc coatings are applied to protect steel mainly as a primer. Mostly the binder consists of alkali silicates that react to a silicate. Zinc is applied as filler and protecting agent. There are both water-borne and solvent-borne systems on the market. After the self-curing of the binder, the salts have to be removed from the surface before further coats are applied.

Table 2.6 Examples of protective systems on metal structures

Application/environment	Coating system
Aluminium cladding elements	Degreasing by solvent cleaning, phosphating (conversion layer) one or two layers powder coat (TGIC, epoxy)
Senzimir zinc-coated steel cladding elements	Degreasing by solvent cleaning, epoxy-polyamide base layer, top layer of alkyd-melamine or acrylic coating, alternatively PVDF top layer
Steel pylon	Hot dipped zinc-coating, phosphating in case of chlorinated rubber paint and an alkyd-polyurethane coat in the case of in-factory production
Transformer boxes <i>in situ</i> protection	Abrasive blast clean, zinc-spraying, two layer of two components epoxy coat, top layer of polyurethane for durable gloss
Steel structures	Abrasive blast cleaning, epoxy zinc coat, epoxy top layer Alternatively, high built epoxy coatings or polyurethane coating can be applied containing less solvent (see comments in previous paragraph)
Steel structures in water works	Abrasive blast cleaning, zinc-silicate or epoxy-zinc coat
Underground steel containers and ducts	Abrasive blast cleaning, epoxy-zinc coat. Epoxy high built top coat, cathodic protection system with monitoring facilities to detect corrosion

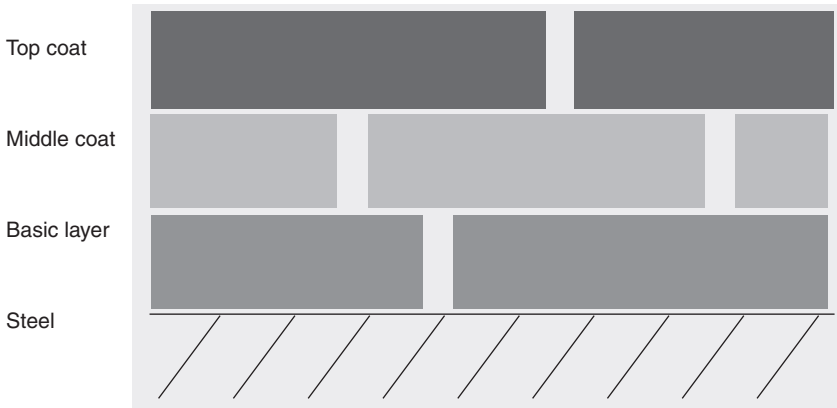
2.5.3.2 Paint coats

The probability of defects or holidays can be much reduced by applying more coats, as illustrated in Fig. 2.35. The tendency in the past decades to decrease the number of coats is not a fortunate one!

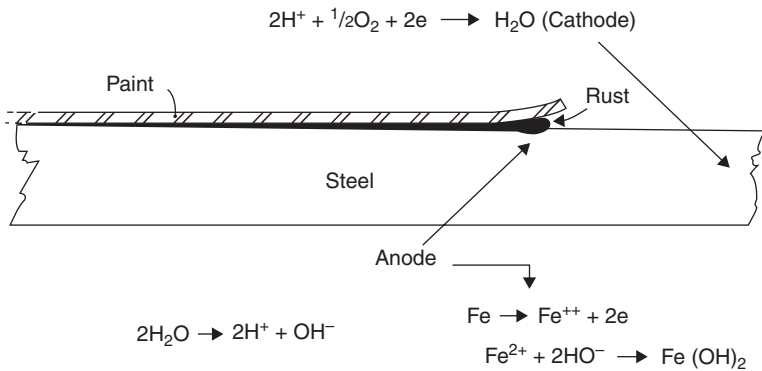
2.5.3.3 Coating failure

Potential causes of coating systems failure are:

- Insufficient cleaning of the metal surface. In the case of salt residue, such features as corrosion pits that stay behind will act as corrosion centres under the paint system applied. In general, coating systems are not impermeable to water and oxygen, which means that corrosion can occur in the case of corrosive materials present at the metal surface. The use of conversion coats or zinc-pigmented primers will decrease the vulnerability to cleaning quality.
- Coating systems can occasionally crack or delaminate, as occurred in the high built epoxy coats applied on the Dutch storm surge barriers



2.35 The effect of the number of coats on the probability of a defect passing the whole coating system.



2.36 Underrust at the holiday in an organic steel coating.

discussed above. This is a basic failure of the system applied, while the previous case is due to inadequate quality of the system's application.

- Defects or holidays in the coating. The probability of continuous defects, as discussed in the previous section, can best be avoided by applying the coating system in at least three coats. If a defect is present, it is the nucleus for corrosion. Due to the underrust mechanism, this will not be restricted to the area of the defect, but expand over the surface. Along the circumference of the defect just under the coating, anodes will form while the 'open' defect area will act as the cathode (see Fig. 2.36).

2.5.3.4 Standards

ISO Standard 12944 Parts 1–8 *Paints and varnishes – Protection of steel structures against corrosion by means of paint systems* is the most frequently used standard for the protection of steel structures. Specific protection systems have been standardized for specific products, such as EN 845 *Specification for ancillary components for masonry*.

2.5.3.5 Performance requirements

Coating systems in many countries are changing a great deal as a result of environmental regulations. The old prescriptive standards and specifications that specify the type of coating and thickness to be applied often are not relevant to these new systems and can lead to failure if adopted uncritically. A better approach is the specification of performance requirements. These requirements could, for instance, include at least five years of proven good experience under similar exposure conditions and resistance to accelerated natural weathering tests. This forces manufacturers to investigate their new systems thoroughly before putting them on the market.

In the Netherlands, the Ministry of Transport, Public Works and Water Management has adopted such a performance-based system for the protection of steel works. This system is discussed under Section 6.4.2.2.

2.5.4 Protection of metals by metallic coatings

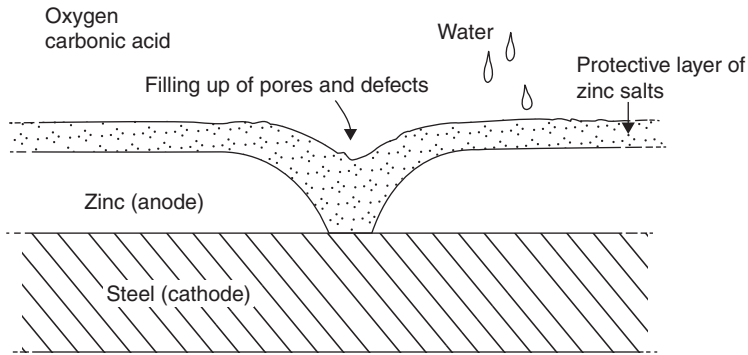
2.5.4.1 Types

Protection of steel by metallic coatings is of great importance in the construction industry and zinc especially is important. Besides zinc, aluminium for spraying applications will be discussed in this chapter. Protection by other metals such as nickel, chromium, tin and lead will not be considered.

2.5.4.2 Galvanizing

Principles

Zinc is less noble than steel and will corrode prior to steel if applied as a steel coating. In such a case, zinc protects steel cathodically, as an active protection (see Sections 2.2.5.4 and 2.5.5). This protection would have a short life if zinc could not passivate in the air but zinc forms a dense skin of zinc hydroxides and carbonates (zinc-patina). Due to passivation, the corrosion rate of zinc is very low. Thus, if the zinc coat fully covers the steel surface, it will also protect in a way similar to passive organic coats. Whenever a defect occurs in the zinc layer, the area not covered with zinc will be cathodically protected, as illustrated in Fig. 2.37.



2.37 Cathodic protection of steel by zinc coat at defect.

Under water, zinc is not passivated and protects only cathodically and can thus be consumed rapidly. Zinc bars are used for cathodic protection of steel of under-water structures, as discussed in Section 2.5.5. The corrosion rate of zinc in the air depends on the climate. Table 2.2 shows corrosion rates. Obviously, these are high in industrial and in marine environments. Industrial climates are sometimes highly polluted. Figures given in Table 2.2 are for the 1970s; however, air pollution has decreased considerably since then, especially in the case of SO_2 emissions that are aggressive to zinc. In western countries, a marine environment is, at present, considered most aggressive to zinc. In the Netherlands, corrosion rates can be as high as $5\mu\text{m}$ per annum along the coast. This means that, in such an environment, a $75\mu\text{m}$ thick zinc coat will be corroded away within 15 years.

Zinc alloys

Zinc alloys can show better corrosion resistance than does pure zinc. Examples of alloys with a higher corrosion resistance are: galfan and aluzinc with 3% of aluminium, and galvalume with 50% of aluminium and 1.5% of silicon.

Organic coating on zinc

Zinc can be protected by applying an organic coating (duplex system). This provides an extra protection. Such a duplex system is required for a service life of steel parts along the coast of approximately 50 years.

Environmental pressure

The use of zinc steel has come under pressure in the Netherlands because of zinc leaching into the surface water. For this reason the use of zinc for

roofing and roof-gutters is discouraged because rainwater pollution due to zinc in solution is considered too high. Duplex systems of a zinc coat and on top of that an organic coat do not have this drawback.

Methods of galvanizing

The following methods of galvanizing are used in the protection of steel:

Hot dip galvanizing, discontinuous

Cleaned steel objects are completely immersed in a molten zinc bath. A relatively thick zinc coat is formed. The thickness depends on the steel thickness, but is usually thicker than 50µm. At the steel interface, an iron–zinc alloy is formed. Pure zinc is present at the surface.

Examples of hot dip galvanized steel construction products are given in Table 2.7.

Hot dip galvanizing, continuous

Coils of sheet or threads are cleaned in a continuous process and transported through a molten zinc bath. Because of the short period in the molten zinc bath, coat thickness is relatively thin, generally in the range of 7 to 35µm. The advantage of such a thin zinc layer is that the sheet steel can be worked easily. The disadvantage is that corrosion resistance is reduced. The process is also referred to as Sendzimir galvanizing. The zinc coat is mostly deposited on both sides but it is also possible to galvanize on one side only.

To increase corrosion resistance, the galvanized coiled sheet is frequently finished with an organic coat. Such a duplex system has a much longer service life. One has to be aware, however, that at places where the sheet is processed further, e.g. coring a hole, the protective coat will not be present and cathodic protection will often not suffice at these locations.

Table 2.7 Typical examples of hot-dip galvanized steel applications in the construction industry

Construction steel	Cable ducts
Frames	Crush barriers
Steel ducts	Reinforcement bars (incidentally)
Crash barriers	Fire-escape stairs
Fences	

Electrogalvanizing

In electrogalvanizing a thin zinc layer is electrodeposited on the steel. Steel objects are immersed in a water bath in which zinc salts have been dissolved. The steel objects are connected to the negative pole and the zinc anodes to the positive pole of a direct current source. The thickness of the zinc coat varies between 3 and 30 μm . Often corrosion resistance is improved by applying conversion layers, such as phosphates or chromates. Applications in the construction industry are for bolts and nuts, screws, sheets and pipes.

Thermal-sprayed coatings of zinc and aluminium

Zinc and aluminium and their alloys are used widely for thermal spraying anti-corrosion coats on steel structures. This technique has no limitations with respect to the size of the structure and can be applied on site. For maintenance of structures, the latter offers a major advantage. Zinc spraying is more widely used, but the importance of aluminium as a spraying material is growing due to better corrosion resistance and improved application techniques. An example of a structure with a thermal sprayed aluminium coat is given in Fig. 2.38.

Molten zinc or aluminium is sprayed in small droplets onto the steel surface. When the droplets land on the surface they adhere but do not flow together. The coat is, therefore, rather rough and porous. Thickness varies considerably over the surface and can be varied in a wide band between various applications, i.e. between 50 and 400 μm . Although pores in the zinc or aluminium sprayed coat will be protected cathodically, it is common practice to apply organic topcoats or sealers to extend the service life and to improve appearance. In general, at least two coats are applied. The process is used for bridges, lock doors, storm surge barriers, pipe systems, steel window frames and other items.

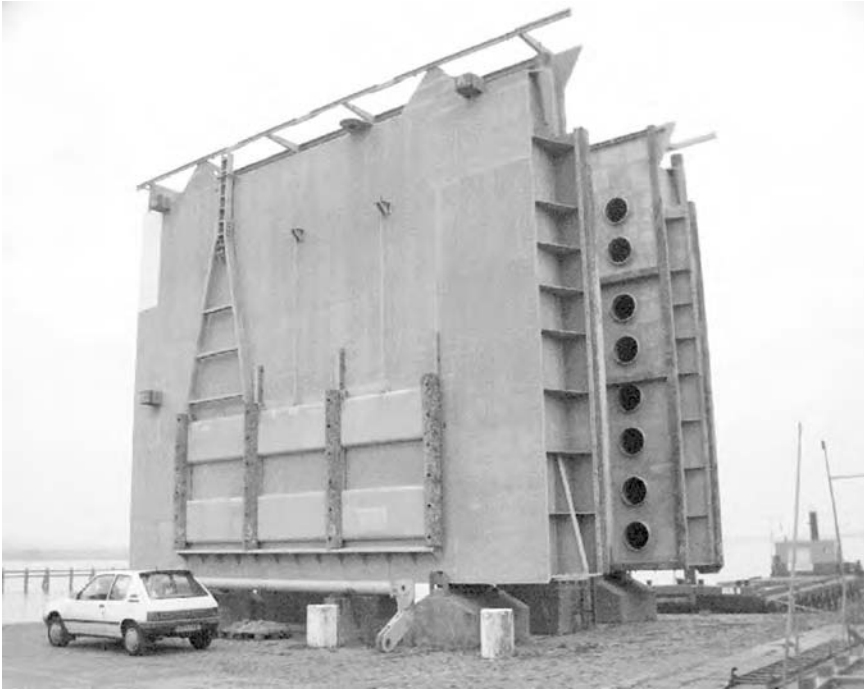
Other application techniques

Other less important techniques where a zinc coat is applied on construction products are electrodeless zinc plating and mechanical plating.

2.5.5 Cathodic protection

2.5.5.1 Principles

In this type of protection, the potential of the metal is decreased to a level close to the equilibrium potential $E_{e,M}$ of the metal (see Fig. 2.5). The anodic or corrosion current is then low. This method can be used if the metal to be protected is embedded in a highly conductive environment, e.g. if it is immersed in water or in conductive soil.



2.38 Sluice door of Zandkreeksluis in the Netherlands with thermal sprayed aluminium (courtesy: Ministry of Transport, Public Works and Water Management, Department of Conservation, the Netherlands).

There are two types of cathodic protection:

- Sacrificial anode (French: protection cathodique par anode sacrificielle, German: kathodisches Schutz mit Opfer-anode, Dutch: kathodische bescherming met opofferanode) or passive systems where the metal is short-circuited with a much more reactive metal.
- Impressed-current (French: protection par courant imposé, German: kathodischer Schutz mit Fremdstrom, Dutch: kathodische bescherming met opgedrukte stroom) or active systems. The metal is connected to an external direct current source.

Both types are widely used.

2.5.5.2 Protection criteria

Full protection can be achieved if the metal potential is polarized to $E_{e,M}$. In practice, this is not easy to achieve, nor is it necessary. In general, polarizing to a level half-way between the corrosion potentials E_{cor} and $E_{e,M}$ suffices to lower the corrosion rate to an acceptable level. In most cases, if

the potential of the metal is 100 mV higher than $E_{c,M}$ it will lower the anodic current to a level below 0.1 mA/m^2 (about $10^{-4} \text{ mm metal/year}$). This is used as a criterion for cathodic protection.

The protection current density depends on the corrosion system. In acid environments with high H^+ concentrations, high current densities are required to achieve the required protection potential. In such cases, hydrogen gas is developed at the metal surface with the risk of hydrogen embrittlement (Section 2.3.2.11). Under those circumstances, cathodic protection makes less sense. A much better option here is an aerated neutral or alkaline environment as, for instance, sea water, fresh-water, non-acid soils and steel bars in concrete. In those cases, the corrosion rate is determined by the diffusion rate of oxygen through water to the metal surface and the current density necessary for protection can be low. Thus, current densities of $90\text{--}140 \text{ mA/m}^2$ are recommended for non-coated offshore structures in the North Sea.

2.5.5.3 *Sacrificial anodes*

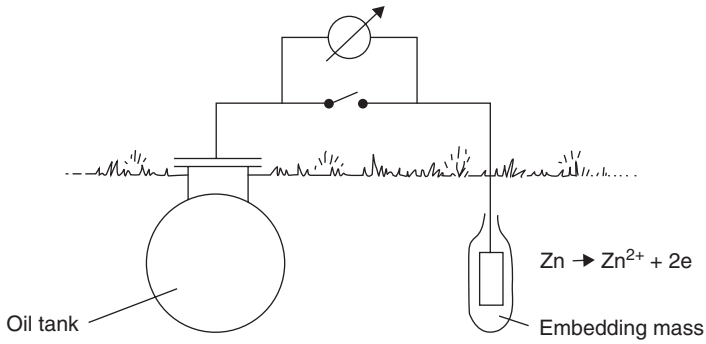
A metal can be used to protect another metal if, given the circumstances, it is more reactive. The standard potentials shown in Table 2.1 indicate when that is the case. Cathodic protection is mostly used for steel structures. Commercial sacrificial anodes include magnesium, zinc and aluminium alloys. These anodes can only be used in environments where no passivation occurs, for example in sea water. Zinc bars connected to steel structures exposed to the atmosphere make no sense!

The quantity of sacrificial anodes required depends on the protection current required, the size of the surface to be protected and the design service life. There are various applications of cathodic protection by sacrificial anodes. The anodes can be placed at some distance from the object to be protected, as illustrated in Fig. 2.39 for an underground fuel tank. The protection current can be measured over the connecting conducting wire. The sacrificial anode is embedded in a conductive backfill, for instance a porous cloth bag with a mixture of gypsum, bentonite clay and sodium sulphate.

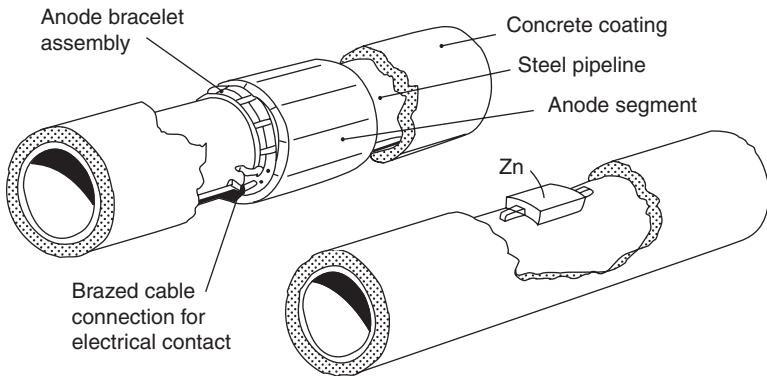
In the case of the protection of underwater pipe systems, the anodes are often placed directly on the pipes, as shown in Fig. 2.40.

2.5.5.4 *Impressed current*

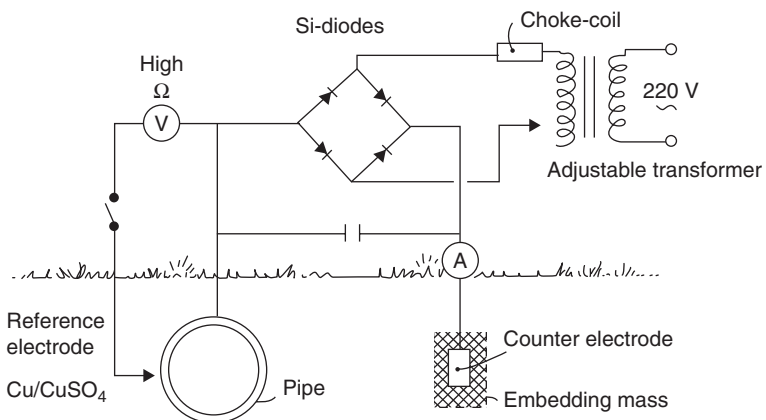
Figure 2.41 shows a buried pipeline cathodically protected by an impressed current system. The buried anodes and the pipeline are both connected to an electrical rectifier. The rectifier supplies direct current to the pipeline and the anode. Similar systems are applied to underwater structures. Most



2.39 Cathodic protection of a buried fuel tank using an embedded buried zinc anode.



2.40 Zinc bracelet anode on an offshore pipeline.



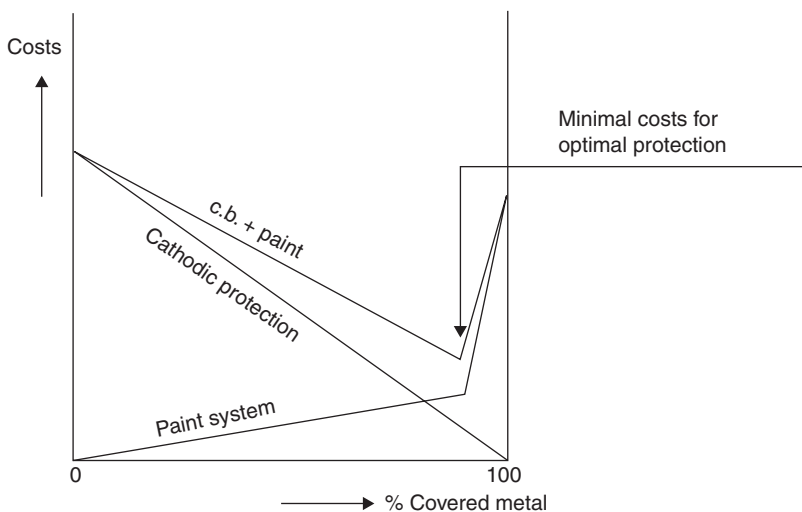
2.41 Buried pipeline cathodically protected by an impressed current system.

impressed current anodes are made from non-consumable electrode materials, which are not easily affected by the current or the environment. Sometimes scrap steel is used, such as old rails. Impressed current systems are more complex than sacrificial anode systems. The capital expenses necessary to supply a direct current to the system are higher than for a simple connection between an anode and a cathode.

The voltage differences in sacrificial anode systems are limited to approximately 1 V, whereas higher voltages (5–8 V) are common in impressed current systems. These higher voltages make it possible to protect large surfaces with only one anode. It is also useful in low-conductivity areas, such as in fresh water and in concrete (Section 3.9). However, there is the disadvantage of likely over-protection. This is especially hazardous in high-strength steels and can cause hydrogen embrittlement. In the case of coated steel, hydrogen generation can also cause the coat to debond. The most common source of electricity is a local power utility. Remote locations can use solar cells, special fuel-driven generators and windmills. Cathodic protection design is conducted by consulting companies who specialize in cathodic protection systems and corrosion control.

2.5.5.5 Cathodic protection in combination with organic coatings

Cathodic protection is often applied in combination with an organic coating system. An organic coat and zinc anodes protect steel ship hulls, for instance. The cathodic protection only has to be active in areas where the



2.42 Cost optimization of a combination of cathodic protection and protection by organic coating.

coating system protects insufficiently (defects, holidays). Pipeline systems buried in the sea are protected by textile or glass fibre reinforced bituminous or organic coats (epoxies etc.) in addition to cathodic protection. Frequently, the entire system is protected against falling stones by a layer of concrete. Attempts must be made at such a double protection system of cost optimization. At decreasing defect rates, costs will increase, while the costs of cathodic protection of the pipeline will decrease concurrently. Figure 2.42 shows such cost optimization. One has to be aware that the costs of pipeline failure are extremely high. An unfailing system for the expected service life is, therefore, a prerequisite.

An example of life cycle cost optimization for sheet-pile walls is given in Section 6.8.4.

2.6 References

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3.1 Introduction

For many decades, concrete (French: béton, German: Beton, Dutch: beton) structures were regarded as virtually indestructible. Concrete was a maintenance-free material. It was thought that the useful life of buildings was not determined by concrete, but by other factors. This image has changed over the last few decades. Multibillion dollar damage to bridge decks in North America due to corrosion caused by de-icing salts, severe wear of concrete structures in the Arabian Gulf Area caused by chloride-initiated corrosion, degradation of sewers due to sulphuric acid attack and deterioration caused by alkali-silica reactions in a great many countries are some of the most striking examples of concrete's tarnished image.

Typically, where problems arise people will try to solve them. In the case of concrete this has led to the development of measures to prevent corrosion, corrective actions when the structural performance is affected (repair techniques), techniques to detect corrosion and to monitor the performance of structures.

The following subjects will be discussed in this chapter:

- Degradation mechanisms
 - chemical reactions in concrete;
 - frost/thaw (de-icing salts) damage;
 - corrosion of reinforcement.
- Principles of protection and repair.
- Repair mortars.
- Protective surface treatments.
- Cathodic protection systems.
- Practical cases of repair.

Much of the information provided in this chapter is taken from References 1 and 2.

3.2 Chemical degradation mechanisms

A distinction can be made into four types of chemical reaction leading to concrete degradation. These are:

- 1 Degradation of cement stone by acids (French: dégradation acide, German: Säure Angriff, Dutch: aantasting door zuren).
- 2 Degradation of cement stone by salts (French: dégradation saline, German: Salz Angriff, Dutch: aantasting door zouten).
- 3 Degradation by expansive salts (French: dégradation saline expansive, German: expansive Salz Angriff, Dutch: aantasting door expansieve zouten).
- 4 Degradation by expansive alkali-silica reactions (French: alkali silica reaction, German: Alkali Silica Dehnung, Dutch: alkali silica reactie).

Occasionally, several of these degradation mechanisms occur simultaneously.

3.2.1 Degradation by acids

Cement stone is mainly composed of lime and silica. Since silica forms a very weak acid in water and lime is a strong base, oxides with a weakly acidic reaction, such as carbon dioxide, can convert these cement pastes, into calcium carbonate and silica-gel. In principle, the deterioration rate is determined by the pH of the acid. The European Standard EN 206-1 *Concrete, Specification, Performance and Conformity* presents limiting values for classes of exposure to chemical attack from natural soil and ground water. 'Determination of the aggressiveness of water, soils and gases' supplies aggressiveness criteria, as shown in Table 3.1. The most aggressive acids are strong mineral acids, such as sulphuric acid, hydrochloric acid and nitric acid. Even if much diluted, these acids still come under class XA3: 'highly aggressive chemical environment'.

3.2.1.1 Biogenic sulphuric attack

Hydrogen sulphide forms a weak mineral acid in water. It can be found in waste water, especially in sewer systems, but also in manure. This acid is too weak to attack concrete severely. However, in the case of closed systems, like sewer systems, it could condense on concrete surfaces above waste water level and subsequently oxidize biogenically to form sulphuric acid.

Sewage water systems

Various sulphur compounds are present in sewage water, both in inorganic and organic form. The most important inorganic sulphur compounds are

Table 3.1 Limiting values for classes of exposure to chemical attack from natural soil and ground water according to EN 206-1 *Concrete, Specification, Performance and Conformity*

Chemical characteristic	Reference test method	XA1 ^d	XA2 ^d	XA3 ^d
Ground water				
SO ₄ ²⁻ mg/l	EN 196-2	≥200 and ≤600	>600 and ≤3000	>3000 and ≤6000
pH	ISO 4316	≤6.5 and ≥5.5	<5.5 and ≥4.5	<4.5 and ≥4.0
CO ₂ mg/l	prEN 13577:1999	≥15 and ≤40	>40 and ≤100	>100 up to saturation
aggressive				
NH ₄ ⁺ mg/l	ISO 7150-1 or ISO 7150-2	≥15 and ≤30	>30 and ≤60	>60 and ≤100
Mg ²⁺ mg/l	ISO 7980	≥300 and ≤1000	>1000 and ≤3000	>12 000 and ≤24 000
Soil				
SO ₄ ²⁻ mg/kg ^a	EN 196-2 ^b	≥2000 and ≤3000 ^c	>3 000 ^c and ≤12 000	>12 000 and ≤24 000
total				
Acidity ml/kg	DIN 4030-2	>200 Baumann Gully	Not encountered in practice	

^a Clay soils with a permeability below 10⁻⁵ m/s may be moved to a lower class.

^b The test method prescribes the extraction of SO₄²⁻ by hydrochloric acid; alternatively, water extraction can be used if experience on the *in situ* use of concrete is available.

^c The 3000 mg/kg limit shall be reduced to 2000 mg/kg where there is a risk of accumulation of sulphate ions in concrete due to drying and wetting cycles or capillary suction.

^d XA1 Slightly aggressive chemical environment.

XA2 Moderately aggressive chemical environment.

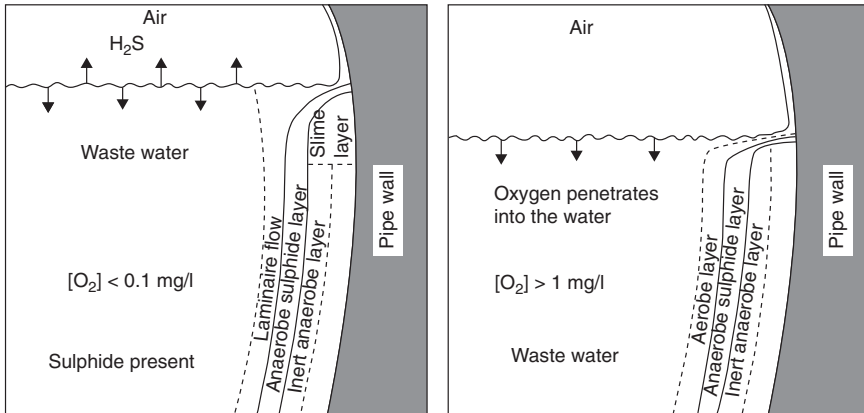
XA3 Highly aggressive chemical environment.

The aggressive chemical environments classified above are based on natural soil and ground water at water/soil temperatures between 5°C and 25°C and a water velocity sufficiently slow to approximate static conditions.

The most onerous value for any single chemical characteristic determines the class.

Where two or more aggressive characteristics lead to the same class, the environment shall be classified into the next higher class, unless a special study for this specific case proves that this is unnecessary.

sulphates and sulphites. Sulphates are found predominantly in domestic waste water. Sulphites can also be present in industrial sewage water. Organic sulphur compounds are found in proteins, amino-acids and certain other compounds and can be present in domestic waste water, as well as in waste water from the food industries. Sulphate-reducing bacilli,



3.1 Slime layer in:
 (a) anaerobic sewage water
 (b) aerobic sewage water.

Desulphovibrio desulphuricans and others reduce sulphates to sulphide. Organic sulphur compounds are also reduced to sulphides or mercaptans. These bacilli can be found in:

- Waste water.
- The mud layer on the bottom of a sewage structure.
- The slime layer on the wall below sewage water level.

The slime layer in particular is biogenically active. This layer can be divided into various zones as shown in Fig. 3.1. In the case where the oxygen content of the sewage water is greater than 1 mg/l, three layers can be distinguished. The outer layer in contact with water is aerobic. The middle and inner layers are anaerobic. Reduction of sulphur compounds can take place in the middle layer. These sulphur compounds diffuse from the waste water into the slime layer. When reduced, the sulphide diffuses backwards into the waste water, but is oxidized again in the aerobic slime layer. There will, therefore, be no net reaction. However, if the waste water itself is anaerobic, i.e. if the oxygen concentration is lower than 0.1 mg/l, the aerobic slime layer will not be present so that sulphide will enter the waste water by backward diffusion. Here it can form hydrogen sulphide that can escape as a gas. The rate of escape is especially high in areas of turbulence.

Domestic waste water generally contains 50 to 100 mg sulphate per litre and 15 to 30 mg of hydrogen sulphide per litre can be formed from the sulphate. Proteins and amino-acids can contribute another 5 mg of hydrogen sulphide per litre. Thus, the total hydrogen sulphide content can amount to 20–35 mg/litre of waste water. Above the water level, a humid atmosphere

will be present, especially in the case of closed structures. Concrete walls and ceilings will be constantly wet and hydrogen sulphide gas will dissolve in this moisture. When other sources of nourishment are present, such as ammonia and carbon dioxide, aerobic bacilli can initiate sulphide oxidation. These aerobic bacilli are present because the above water level environment is often relatively oxygen-rich. Various species of bacilli are able to convert sulphates into sulphides. As a result of this conversion, the concrete surface environment will become more acid. When the acidity of the environment increases, some bacilli populations will die, but other bacilli will encounter favourable growth conditions. The most acidic bacillus, *thio-oxidans* (also called *Bacillus concretivorus*), can exist at pH levels as low as 1. This acidity has indeed been measured in practice. The aggressiveness of sulphuric acid will depend on its concentration. In general, concrete is not resistant to an acid environment with a pH lower than 4. This means that it is certainly not resistant to a pH of 1 as measured in closed sewage systems. The degradation observed is therefore mainly due to the dissolving of the cement paste by a strong acid.

Rate of deterioration

At high hydrogen sulphide concentrations, the rate of degradation appears to depend mainly on the lime content of the concrete. Pomeroy³ defines the rate of deterioration of concrete by sulphuric acid as follows:

$$C = \frac{11.5K \cdot \phi_{sw}}{A} \quad [3.1]$$

where:

C = Corrosion rate in mm/year

K = factor taking into account possible losses during reaction

ϕ_{sw} = Sulphide flux to the surface of the sewer system in g/m² hour

A = the quantity of acid required to neutralize a certain amount of concrete in comparison to a quantity of acid neutralized by the same mass of calcium carbonate

In a highly aggressive hydrogen sulphide environment, standard concrete durability parameters, such as the water/cement ratio, no longer seem to apply. However, at lower levels of aggressiveness, the familiar parameters again seem to dominate. The transition from mildly to highly aggressive hydrogen sulphide environments is not very clear. It is, therefore, quite difficult to take the most appropriate measures for safeguarding service life design.

Preventive measures

In terms of volume, aggregate is the most important constituent of concrete. According to the above formula, less acid will be available for the attack on cement paste if this aggregate is able to also neutralize sulphuric acid. Therefore, in sewage systems subject to high hydrogen sulphide loading, the use of limestone or dolomite aggregates is a measure that can decrease the rate of deterioration.

At higher hydrogen sulphide levels, ordinary concrete in closed sewage structures must be protected and one protective measure is to increase the cover. At an average degradation rate of 3mm/year, an extra cover of 60mm will be required to warrant 20 years of service without the main structure being affected. In large parts of the Netherlands, the soil has a low load-bearing capacity and so an increase in concrete pipe deadweight is not regarded favourably. However, there are many alternative protective measures, as discussed in Section 3.8.

Manure silos

Because of severe restrictions on the use of manure, thousands of manure silos have been built in the Netherlands and in other European countries. There is a ban on spreading manure onto the fields during the winter because of percolation by rainwater and because leaching of manure can pollute the soil, especially with nitrates. Since manure also contributes to acid rain due to evaporation of ammonia, the silos have to be closed so that ammonia cannot escape into the atmosphere.

Contents of liquid manure constituents aggressive to concrete are given in Table 3.2.⁴ The constituents of the manure itself are not especially aggressive, but as in the sewage systems, hydrogen sulphide can be formed by sulphate-reducing bacilli. An investigation into existing manure silos has shown that in low-quality concrete a similar degradation can occur as in sewage systems. In general, however, this degradation is less severe because the hydrogen sulphide largely escapes when manure is stirred periodically.

Table 3.2 Contents of aggressive constituents in liquid manure (mg/l)⁴

	Cattle	Poultry	Pig
Mg	810–1570	1260–2380	640–2000
Cl	1810–4180	80–3220	640–3360
N	3800–7600	5950–15750	4320–11520
SO ₃	2000–3000	2000–4000	1000–2000

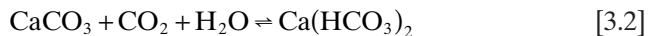
Table 3.3 Field investigation conclusions on concrete manure silos⁴

	No degradation	Degraded
Compressive strength (N/mm ² , MPa)	50–80	12–23
Porosity (% v/v)	11.3–15.2	15.1–22.3
Water penetration DIN 1048 (mm)	7–25	50–75

Table 3.3⁴ shows the conclusions from a field investigation with respect to concrete deterioration in manure silos. Deterioration occurs above the manure level; below this level no significant damage has been observed. It is frequently recommended that concrete exposed to this kind of very aggressive environment should be protected with an organic coating. However, the reliability of coatings as a durable protection of concrete against degradation by sulphuric acid is doubtful, partly because of the reasons discussed in Section 3.8.4.4 (blistering of coating due to gypsum formation beneath the coating). In addition, coating system damage in manure silos is virtually unavoidable because of the operations taking place in them such as filling, mixing and emptying. Therefore, the use of an additional sacrificial cover is recommended in the Netherlands. On the basis of the results available, it is expected that the specified sacrificial cover of 10mm will be sufficient to warrant the silos a 20 year design life.

3.2.1.2 Aggressive water

A special case of acid attack is exposure to low-hardness aggressive water containing carbon dioxide. This water is able to convert calcium carbonate into soluble calcium bicarbonate. The European Standard EN 206-1 specifies the criteria for aggressive water containing carbon dioxide as shown in Table 3.1. The reaction of this aggressive water then reads as follows:



3.2.1.3 Organic acids

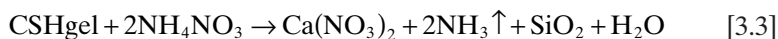
In general, organic acids are weaker than mineral acids. Nevertheless, acids such as acetic and lactic can seriously degrade concrete due to their lime-complexing ability. Concrete attack by the highly acid extracts of corn has also attracted much attention over the past years. The general pattern of this acid degradation of concrete is that it slowly dissolves the cement paste starting from the surface. The aggregates usually degrade less quickly than the cement paste. After some time, these aggregates are no longer

embedded in the cement paste. Sometimes, the aggregate itself is also degraded. Which happens, for instance, when limestone or dolomite aggregates are used. In this case degradation initially looks more severe, but when degradation proceeds, the situation reverses because lime or dolomite provides an additional alkaline buffer that can neutralize the acid.

3.2.2 Degradation of cements by salts

3.2.2.1 Ammonium salts

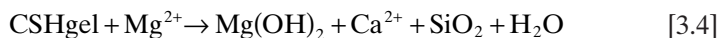
The ammonium ion (NH_4^+) is quite innocent in itself. However, the cation is always found in combination with an anion and this anion usually reacts with cement hydrates. As calcium salts are formed, the ammonium ion changes to and is released as gaseous ammonia. The release of ammonia shifts the reaction to the right, as shown, for instance, in the reaction with ammonium nitrate:



Both water-soluble calcium nitrate and ammonia can easily leave concrete. With respect to ammonia, EN 206 specifies the classes of aggressiveness as shown in Table 3.1. Although the table suggests that ammonium salts are aggressive even in small concentrations, some ammonium salts are less hazardous, e.g. ammonium carbonate, oxalates and fluoride. Even though these salts attack the cement paste, the reaction products are insoluble and non-expansive and therefore less aggressive than salts such as ammonium nitrate and ammonium sulphate. Ammonium sulphate can react with cement paste to produce calcium sulphate. Subsequently, this sulphate can lead to ettringite formation.

3.2.2.2 Magnesium salts

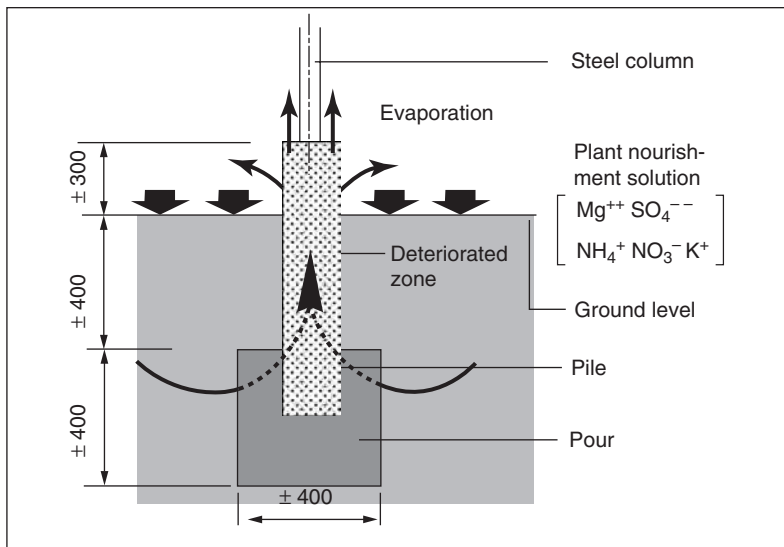
Magnesium salts react first with calcium hydroxide, forming low soluble magnesium hydroxide. However, magnesium is also able to react with calcium silicate hydrate gel, as shown in the following reaction:



Non-cementing reaction products, such as brucite, are formed in this reaction. However, if the magnesium salt is magnesium sulphate, an additional degradation factor due to sulphate deterioration can, of course, occur. EN 206 specifies classes of aggressiveness as shown in Table 3.1.

3.2.2.3 Fertilizers/nutrients

Greenhouse foundations are mostly made of concrete. They consist of small precast concrete piles embedded in concrete poured *in situ* as shown in Fig.



3.2 Greenhouse foundation showing salt transport.¹

3.2.¹ A temperature of 20 to 25 °C is maintained in greenhouses. Nourishment solutions are sprayed onto the soil continuously or at regular intervals and they have the composition shown in Table 3.4. They contain much nitrate, sometimes sulphate, always some ammonium, calcium, and magnesium as well. These nourishment solutions penetrate the soil, are absorbed by the concrete and transported through the pore system down to the ground level where the water evaporates and the salts remain. Severe deterioration has been observed at ground level. Expansion was occasionally obvious. The concrete always appeared to have lost its integrity, often without signs of expansion. Figure 3.3 shows a deteriorated pile.

A reaction of ammonium salts and magnesium salts with the calcium silicate hydrate gel of the cement paste can take place and ettringite formation has also been observed. An accumulation of the most mobile ions could be observed above ground level, whereas the less mobile ions were concentrated below ground level. Expansion due to ettringite formation (Section 3.2.3.1) was occasionally present; in other cases ettringite could be detected but no expansion was observed. In the latter case it was considered likely that cement paste degradation occurred first, while ettringite formation took place later on. Ettringite formation occurred when there was enough space to release the expansive ettringite forces. The results of the research into hothouse concrete foundations has been used for the formulation of CUR Recommendation 8 *Durability of concrete foundations in greenhouses*.⁵ The most important requirements in this Recommendation

Table 3.4 Composition of plant nourishment solutions¹

Ions	Minimum (mmol/l)	Maximum (mmol/l)
NO_3^-	7.5	12.25
SO_4^{2-}	0.75	2.5
NH_4^+	0	1.25
K^+	4.0	7.25
Mg^{2+}	0.5	1.5

3.3 Deteriorated greenhouse foundation pile.¹

are that a Portland blast furnace slag cement has to be applied with a high slag content and that the water penetration must not exceed 30mm in accordance with the European Standard EN 12390-8 *Testing hardened concrete – Part 8 Depth of penetration of water under pressure*.

3.2.3 Expansive salts

3.2.3.1 Sulphate attack

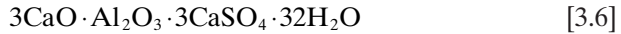
Mechanism

Sulphates are present in many natural waters such as ground water, but they can also be found, occasionally in high concentrations, in industrial waste

water. When sulphate ions penetrate pore systems by absorption of water containing sulphate or by diffusion, they can react with the free lime present in the concrete and form gypsum according to the following reaction:



Subsequently, this gypsum reacts with calcium aluminate hydrates to form ettringite:



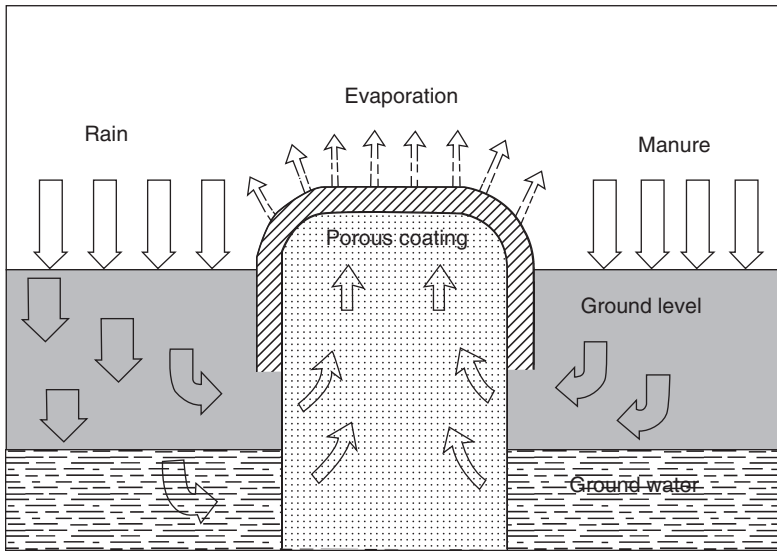
Formation of this double salt in ordinary hardened concrete can be expansive. When the expansion forces exceed the tensile forces of the concrete, cracking can occur.

Foundations of high voltage poles

Some concrete foundations of high voltage poles in the Netherlands have been damaged, especially at the foundation edges above ground level. The concrete appeared to be cracked, spalled and occasionally completely degraded. White salt deposits could be observed at the cracks. Analysis has shown excessive ettringite formation in the concrete and calcium carbonate on the outside. The concrete contains a large number of micro-cracks. All degraded foundations were of rather poor quality concrete.

The environment of the affected concrete foundations appears to be fairly aggressive, mainly due to the use of liquid manure which, as discussed in the previous section, can contain rather high quantities of salts. A possible cause of damage can be suction of water containing sulphates originating from liquid manure. The water containing salt is transported through the capillary pores to the ground level where the water evaporates and the salts remain, as shown in Fig. 3.4.¹ Ettringite can be formed because of sulphate accumulation. However, in some cases there were also signs that simultaneous degradation of the cement paste could occur due to cations exchanging with those ions in the calcium silicate hydrate gel. Furthermore, an alkali-silica reaction of the aggregates was also suspected in some cases (see Section 3.2.4). It was noteworthy that the coating applied on the foundation did not prevent degradation. The coating was not impermeable and was not generally resistant to salts precipitating beneath the coating surface. Blistering was observed. It was not completely clear which degrading mechanism dominated.

It was advised that future similar problems could be prevented by applying very dense concrete with a water penetration rate lower than 30mm, determined according to the EN 123480-8 and manufactured from Portland blast furnace slag cement, CEM III/B with a high slag content (>65%).



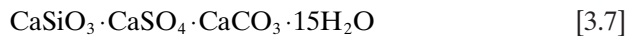
3.4 Degradation mechanism in high-voltage pole foundations.¹

Sulphate attack in sea water

Sulphate resistance has long been advocated as an essential quality of marine structure durability. However, in the case of the joint presence of chlorides and sulphates, strong evidence was found that the hazards of sulphates are negligible. Although ettringite is found, it appears not to be expansive.

3.2.3.2 *Thaumasite*

Another expansive sulphate double salt is thaumasite for which the formula is:



Formation of this salt not only requires sulphate, but also calcium carbonate. Calcium carbonate is not always present in concrete, but could be there if the carbonation frontier advances (Section 3.4.2) or when using limestone aggregates. Thaumasite formation is especially expansive at low temperatures. Damage that can be ascribed to thaumasite formation as a deteriorating salt is only of recent date. During thaumasite formation, ettringite occurs as an intermediate salt. EN 206 specifies the classes of aggressiveness for water containing sulphate as given in Table 3.1.

3.2.3.3 *Delayed sulphate degradation*

Mechanism

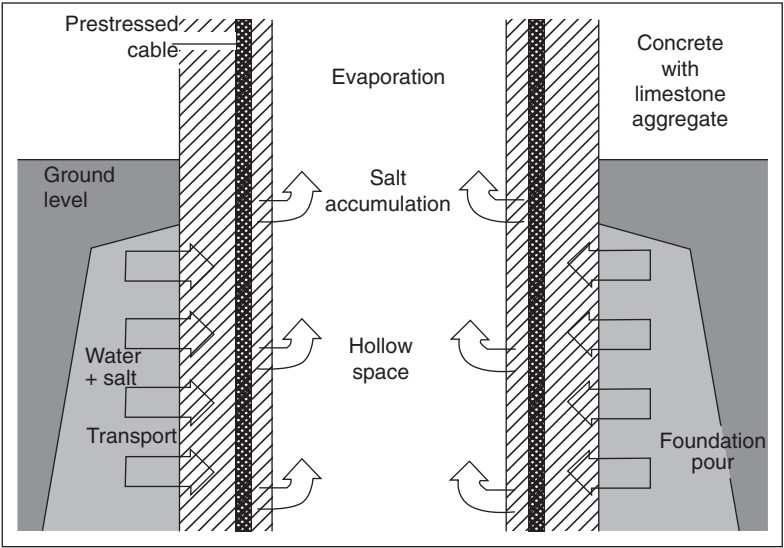
Sulphates that cause the formation of expansive double salts usually penetrate from the exterior. However, under certain conditions the cement paste itself can also supply sulphates. Cement always contains a small percentage of calcium sulphate that acts as a setting time regulator. Normally this calcium sulphate reacts with calcium aluminate and water to form ettringite, which later on is converted into monosulphate. This process is not expansive. In hardened concrete we do not normally find large quantities of ettringite and there is no free calcium sulphate. In the pore water too, hardly any sulphate is present. However, when cured at higher temperatures the calcium sulphate does not react to form ettringite or monosulphate, but remains available as free calcium sulphate. Later on, calcium sulphate can react to form ettringite. Unlike the initial reaction to ettringite, this delayed reaction can be expansive.⁶

Delayed expansive ettringite formation occurs in Portland cement (CEM I) concrete that has been steam-cured at temperatures above 60 °C when it is exposed to a very humid environment. The ettringite appears to precipitate, especially at the aggregate/cement paste interface and in micro-cracks. The micro-cracks are likely to be present due to heat treatment when the concrete is cooled down too quickly. The higher the temperature during accelerated curing, the greater the probability of ettringite formation. The cause of this reaction is still under discussion. This type of degradation does not appear to occur to a deteriorating extent in Portland blast furnace slag cement CEM III.

Prestressed lampposts

In the 1980s, a prestressed concrete lamppost fell down spontaneously on a motorway in Belgium. Figure 3.5 shows the degradation mechanism schematically. Salt-laden water had been continuously transported to the inner hollow space of the lamppost, which acted as a kind of chimney. The water evaporated and salt remained. Thaumasite and ettringite have been observed in the lamppost. Thaumasite formation was possible because limestone was applied as a coarse aggregate. As a result of expansion and the subsequent cracking due to double salt formation, the prestressed cables corroded and the lamppost finally fell down.

The groundwater was not especially aggressive. However, the precast poles had been subjected to accelerated hardening by steam curing at rather high temperatures. In all likelihood, most damage was caused by delayed sulphate attack. Figure 3.6 shows a photograph of a deteriorated lamppost.¹



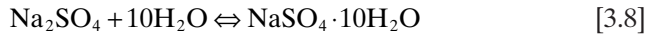
3.5 Cross-section of hollow prestressed lamppost foundation showing salt transport.¹



3.6 Photograph of deteriorated lamppost.¹

3.2.3.4 Salt weathering

Another type of salt attack in concrete structures is surface salt weathering, also referred to as physical salt attack. In the tidal and spray zones, salts will be deposited at the concrete structure surface during dry–wet cycles. Sea salt is deposited on the concrete and partly sucked into the concrete surface skin. Crystallization and hydration of salts cause swelling, which can disrupt the surface and lead to a washed-out surface texture, especially in the spray-zone. A salt well known for its hydration pressure is the thenardite form of sodium sulphate (Na_2SO_4), that reacts with water to form mirabilite ($\text{Na}_2\text{SO}_4 \cdot 10\text{H}_2\text{O}$).



This reaction frequently takes place where the ground water is often very salty and the evaporation rate high, for instance in the Gulf area.

3.2.4 Alkali silica reaction

3.2.4.1 Reaction mechanism

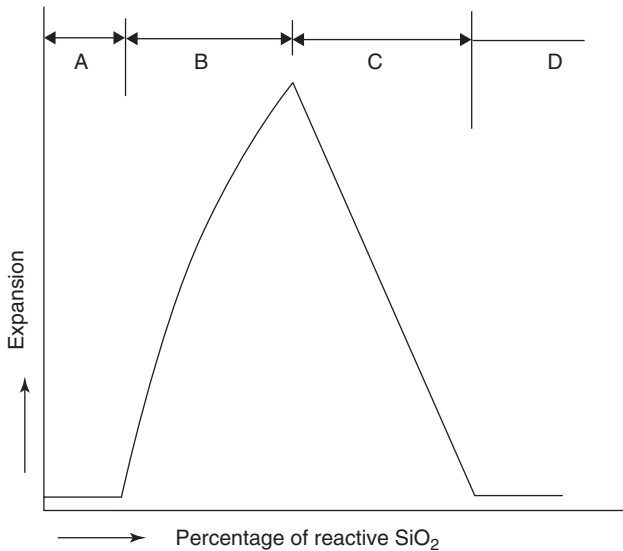
Some aggregates appear to react expansively with cement. Beyond a certain expansion rate, concrete cracks. This expansion is mainly caused by soluble silica minerals in the aggregates.

The following phenomena can be observed:

- Expansion of the concrete.
- Small crack formation at the surface.
- Spalling of concrete.
- Expulsion of alkali–silica gel.

Figure 3.7 shows an example of a structure affected by an alkali–silica reaction. Two reaction mechanisms can be distinguished in this process: the silica solution that reacts with alkalis and the expansion process. Silica dissolves because of the highly alkaline environment of the pore water in the concrete where the pH can be as high as 14. A sodium silicate and potassium silicate solution is then formed. The second part of the mechanism concerns an osmotic process. The silicate solution tries to equalize its salt concentration with the surrounding pore water system. However, around the aggregate a semi-permeable barrier is present in which calcium plays a role. The silicates cannot pass this barrier, but water can so the silicate solution attracts water and, the silicate compound swells.

The rate of this expansive reaction depends on silica solubility.



3.7 Pessimum area in which the alkali-silica reaction can be destructive.

3.2.4.2 Reactive aggregates

Reactive silica is present in many aggregates. Known forms of alkali-reactive silicas are opal, chalcedony, cristobalite, tridymite and crypto-crystalline quartz. However, a reactive aggregate does not necessarily cause expansion. To be expansive, the reactive silica content must be in the so-called pessimum area. Only in cases where the reactive silica content is within certain boundary values does expansion occur, as shown in Fig. 3.7. In area D, there is plenty of alkali reactive material, but no expansion. The boundary values of the critical area depend on the mineral composition of the reactive material. In the case of opal, the boundaries are between 0.5 and 10% (by weight). In the case of crypto-crystalline quartz, a content of more than 10% is critical. In the Netherlands, for instance, flint (Dutch: vuursteen) is present in river gravel from the River Meuse and in sea gravel from the English section of the Channel. In the case of the River Meuse, the content is mostly smaller than the pessimum, while in sea gravel the content is outside the pessimum area. It is difficult to determine whether or not an aggregate will show expansive behaviour. For each type of aggregate, the pessimum area will be different and even the method of determination can differ.

3.2.4.3 Conditions for expansion

For expansion to take place, the following conditions must be met:

- Presence of alkali-reactive aggregates.
- Sufficient water present in concrete. Destructive expansion only occurs in the case of continuously or intermittently wet conditions.
- Sufficient alkali content in the concrete's pore water. Alkali content is expressed in Na_2O -equivalency. It is the sum of the sodium oxide (Na_2O) content plus 0.658 times the potassium oxide (K_2O) content. At an Na_2O -equivalency of less than 0.6%, it was shown that no alkali-silica reaction occurs in Portland cement. This will, however, be different in other cements. In blast furnace slag cements, such as CEM III/A and B, the limit can be substantially higher because the alkali percentage that dissolves in pore water is substantially lower than in Portland cements.
- Even when the initial alkali content is below the boundary values, the alkali content can increase due to alkali ingress from the exterior as will, for instance, be the case when de-icing salts are used.
- Ion mobility must be sufficient. A high diffusion resistance will prevent or decrease the expansion rate. Blast furnace cement CEM III/A and B and Portland fly-ash cement (CEM II/B-V according to EN 197) do have a substantially lower mobility than Portland cement, CEM I.

A decrease of the water/cement ratio also increases the diffusion resistance. However, the effect of the water/cement ratio on the diffusion resistance is much less than the effect of the cement type. Moreover, at decreasing water/cement ratios, alkalinity increases, which promotes alkali-silica reactions.

- There must be free lime in the vicinity of the aggregates. If free lime is not available, there is no expansion, probably because the semi-impermeable layer around the aggregate is not formed. This can be another reason why blast furnace cement and Portland fly ash cement do not show destructive expansion, whereas Portland cement does. It has been observed that the interfacial zone between the cement paste and aggregate is substantially less thick and less free lime is present in blast furnace cement and Portland fly ash cement.

3.2.4.4 Measures to prevent ASR damage

Measures to prevent damage by alkali-silica reaction are:

- Use of non alkali-reactive aggregates.
- Restricting the alkali content in concrete or cement, e.g. the use of low-alkali Portland cements with an alkali content (Na_2O equivalency) lower than 0.6%, but this does help sufficiently in the case of alkalis penetrating from the exterior.
- The use of additions as in blast furnace slag cements (CEM III) and Portland fly ash cement (CEM II/B-V) as discussed in the previous

Table 3.5 Requirements to be met by cements so as to prevent ASR according to Dutch CUR Recommendation 89⁷

	CEM II/B-V	CEM II/B-V	CEM III/A	CEM III/B
Cement, fly ash, slag content (% by weight), resp.	≥25	≥25	≥50	≥66
Na ₂ O-equivalency of fly ash (% by weight)	$1 \leq x \leq 2$	$2 < x \leq 3$	–	–
Maximum alkali content of cement (% by weight)				
• In the case of alkali contribution, other constituents ≤0.6 kg/m ³	1.1	1.3	1.1	1.5
• In the case of alkali contribution, other constituents ≤1.2 kg/m ³	0.9	1.1	0.9	1.3

section. In that case restrictions on the alkali content could be alleviated as shown in Table 3.5 which presents Dutch recommendations.⁷

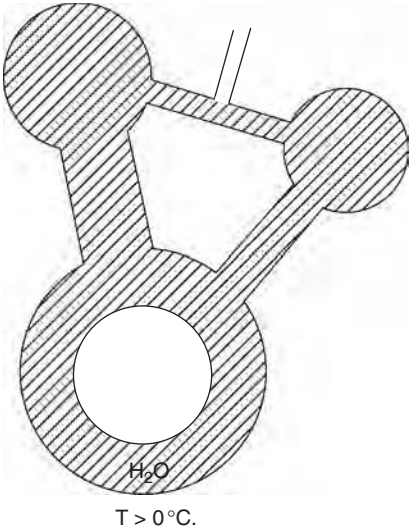
- Restricting the access of water to the concrete: this restriction is often too demanding.

3.3 Frost–thaw and de-icing salt damage

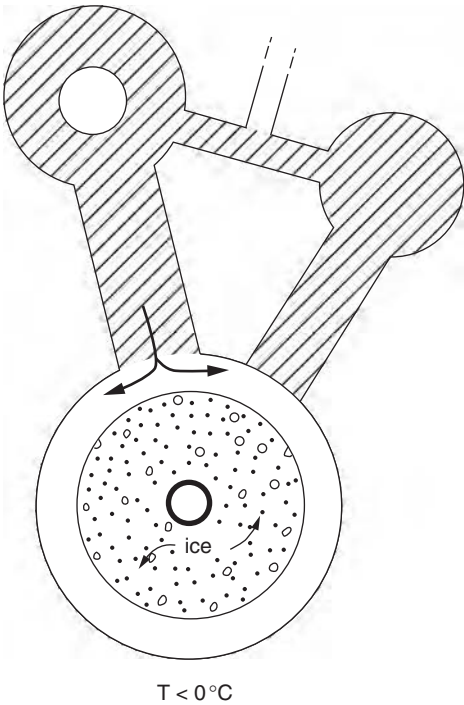
3.3.1 Frost–thaw

Frost–thaw damage (French: gel et dégel, German: Frost–Tau, Dutch: vorst–dooi) is due to expansion that occurs when water is converted into ice and to the difference in the temperature coefficients. At decreasing temperatures, the water present in the largest pores will start to freeze first. When water turns into ice, the expansion is 9%. If concrete is saturated with water, this will cause forces exceeding the tensile strength. Consequently, the concrete will crack. Fortunately, concrete is seldom fully saturated and the larger pores will be filled with water to a lesser degree, as shown in Fig. 3.8. There will, therefore, be space available for expansion and no damage will occur. Expanding ice pushes away the surrounding water into the smaller pores. This causes hydraulic pressure in the pore system. In some models, this hydraulic pressure is considered the cause of expansion, but here we will consider a model based on the assumption that damage occurs not during freezing, but during thawing. This model is illustrated in Figs. 3.9 and 3.10.

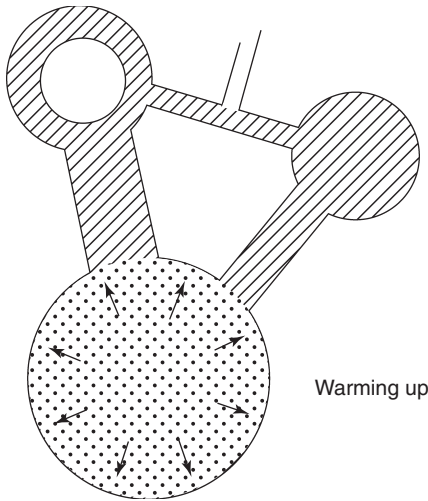
When water in the large pores freezes, water will be attracted from adjacent smaller pores, both in fluid and in vapour form. The smaller pores are



3.8 Water in cement stone pores, temperature above freezing point.



3.9 Frozen water in large pores.



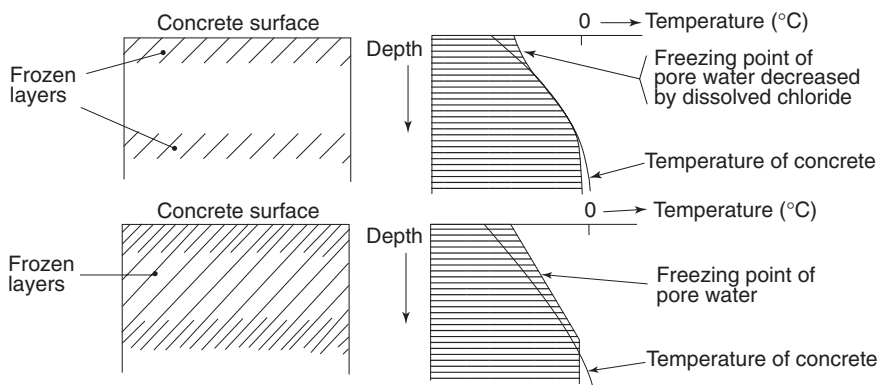
3.10 Expansion of ice in pores during warming-up.

more or less freeze-dried by the ice in the larger pores. Consequently, the ice in the larger pores will grow. This is similar to the process of ice lens growth observed in soil. If temperatures subsequently rise, the ice will expand and since the temperature expansion coefficient of ice is five times larger than that of concrete, it will exert a force. It can then be calculated that a tensile force will be introduced that can be expressed as $0.36 \times \Delta T \text{ N/mm}^2$. This means that at a change in temperature of 15°C , the tensile strength of concrete will be exceeded. (It should be stressed that the foregoing describes only one of the models that explains the frost–thaw deterioration of concrete.)

Because of the complexity of the frost–thaw process, it is understandable that it is difficult to develop laboratory tests that give results representative of practice. Most of the tests concern water immersion cycles and subsequent freezing. In these tests, the water content of the concrete will increase with each cycle. Consequently, the severity of frost–thaw loading will increase with an increasing number of cycles.

3.3.2 Frost–thaw de-icing salt deterioration

In most Western European countries, frost–thaw damage hardly occurs, but frost–thaw de-icing salt damage does. The combination of frost–thaw loading and the use of de-icing salts appear to be more serious. De-icing salts have a number of effects that influence the deterioration rate:



3.11 Frozen layer with intermediate unfrozen layer leading to blistering.

- Salts penetrate the pores and increase the average water content of the pores due to the hygroscopic character of the salts.
- The salts do lower the freezing point. The freezing point will decrease linearly with the salt concentration in the pore water. As shown in Fig. 3.11, this can lead to situations where there is a thin unfrozen layer sandwiched between two frozen layers. The unfrozen layer will freeze when the temperature drops further. However, there is then no space left for the freezing water to be transported out when the ice expands. This explains the well-known scaling off of concrete surfaces that are subject to frost–thaw de-icing salt deterioration.
- The difference in vapour pressure between ice and water decreases at increasing salt concentrations in the water. The freezing–drying of smaller pores by ice in the larger pores will diminish.
- Another difference between frost–thaw and frost–thaw de-icing salt deterioration is that surfaces where de-icing salts are used are generally horizontal and often wet and therefore more prone to damage.

In general, Portland cements, CEM I, show better resistance to frost–thaw (de-icing) salts loading than do blast furnace slag cements, CEM III. This appears to be related to the effects of carbonation on the microstructure of the concrete.⁸ In Portland cements, carbonation makes the concrete denser, while in blast furnace slag cements, CEM III, the surface becomes more permeable. CEM II/B–V Portland fly ash cements show a performance that is similar to CEM I cements. In concrete with powder coal fly ash, resistance depends on the quantity of fly ash applied. In high-strength concretes in which fine and highly reactive additions, such as silica fume, are used, the effect on the frost–thaw (de-icing salt) resistance is not clear. Both positive and negative effects have been reported, see Section 5.2.2.2.

3.3.3 Measures to improve resistance

A good frost–thaw de-icing salt resistance is required for bridges and road surfaces. In general, the performance of ordinary concrete will be insufficient. A number of measures can be taken to improve performance. These are described below.

3.3.3.1 *Air-entrainment*

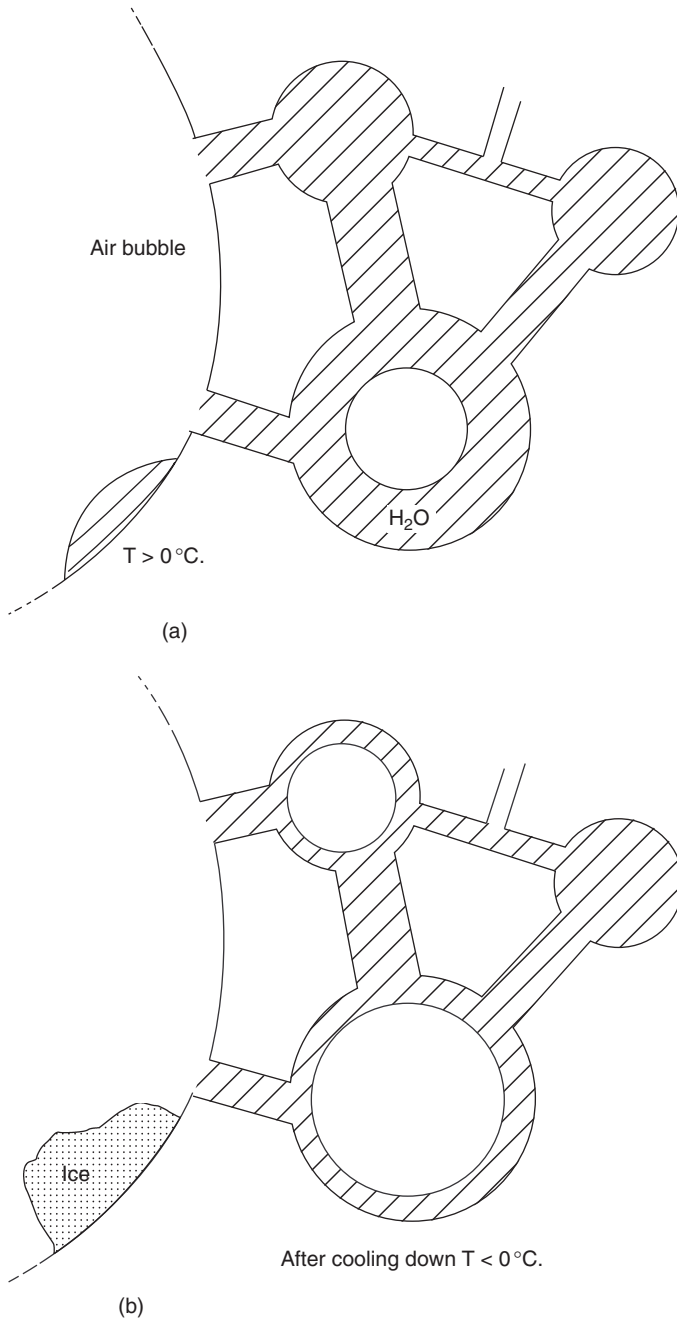
Air-entrainment is by far the most frequently used measure to improve frost–thaw resistance. The air bubbles that are large in relation to the cement pores, are, in practice, barely filled with water. They will act as expansion vessels for water pushed away by the expanding ice and also act as freezing-drying chambers. Figure 3.12 illustrates the latter. The distance of an ambient point in the concrete to the nearest moist air bubble must be less than $250\text{ }\mu\text{m}$ (see Fig. 3.13) and is known as the distance factor. Its determination is quite cumbersome. A minimum air content is, therefore, frequently prescribed; it depends on the nominal size of the aggregate. In EN 206, a minimum air content of 4% is recommended. A percentage of air bubbles smaller than $300\text{ }\mu\text{m}$ is prescribed in Germany.

3.3.3.2 *Surface treatment*

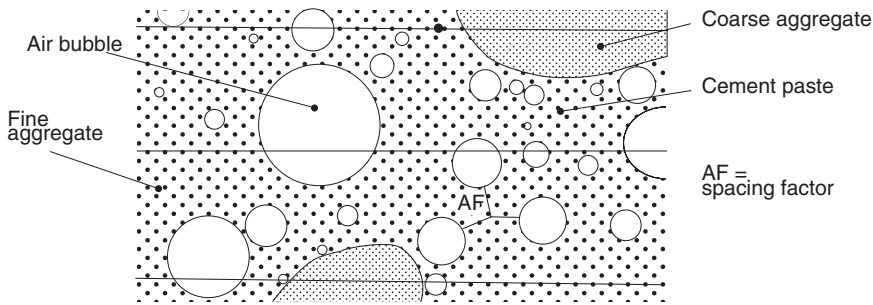
Frost–thaw (de-icing salt) damage of concrete only occurs if the concrete is wet. It is therefore worth while considering ways of preventing the concrete from becoming wet. By applying an organic coat to the surface (see Section 3.8.5.3), the ingress of water can be prevented. However, such a coat also prevents evaporation of the water already present in the concrete or of water that enters from the soil beneath the concrete. In the case of frost–thaw de-icing salt loading, e.g. at concrete bridge parapets, it appears that water condenses behind the coating system. When the concrete just below the coating is saturated, freezing of the concrete will lead to damage of the concrete skin and blistering of the coating can be observed. A better alternative is to apply hydrophobic agents to the surface that do not counteract evaporation in concrete. These hydrophobic agents are discussed in Section 3.8.5.1.

3.3.3.3 *Low water/cement ratio*

In the Netherlands, applying a water/cement ratio lower than 0.45 in combination with good curing is permitted. In such cases the use of air-entrainment is not obligatory. Obviously, this measure only applies to a country with a moderate climate.



3.12 (a) Air bubble and pore system filled with water and air; (b) air bubble and pore system filled with ice, water and air.



3.13 Cross-section of air-entrained concrete.

3.4 Reinforcement corrosion

3.4.1 Passivation

Concrete is highly alkaline and steel appears to passivate in highly alkaline environments (see Section 2.2.5.3). A thin (one nanometre) impermeable layer of iron oxides and iron hydroxides is formed that effectively stops ferro ions from passing. In general, steel in concrete is well protected. However, there are two phenomena that can destroy passivation and bring on corrosion. These are: the reaction of concrete with carbon dioxide from the air, referred to as carbonation, and the presence of ions, notably chloride ions, that can penetrate the concrete or are already present and disrupt passivation locally.

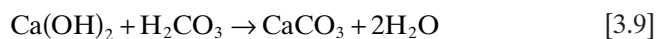
3.4.2 Carbonation

3.4.2.1 Mechanism

Concrete in contact with air reacts with carbon dioxide and is neutralized. If the carbonation frontier reaches the reinforcement, the steel is no longer passivated. Subsequently, the reinforcement can corrode and the concrete can crack. The concrete sculpture by Picasso in the National Museum of Israel in Fig. 3.14 shows such cracking. Carbon dioxide reacts with the alkaline and earth alkaline constituents in the concrete. The reaction occurs at the water–air frontier in the pores, just below the water surface, as shown in Fig. 3.15.

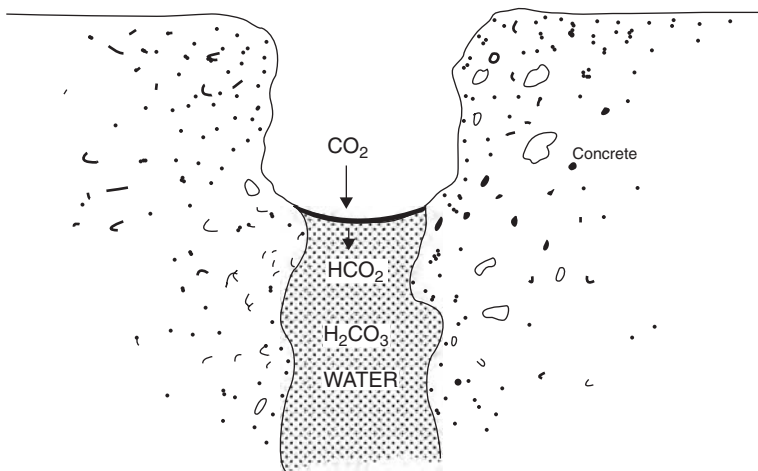
Carbonation is, therefore, strongly linked to drying of concrete. Wet concrete does not carbonate!

One of the main constituents in Portland cement concrete is free lime, $\text{Ca}(\text{OH})_2$. This free lime reacts as follows:

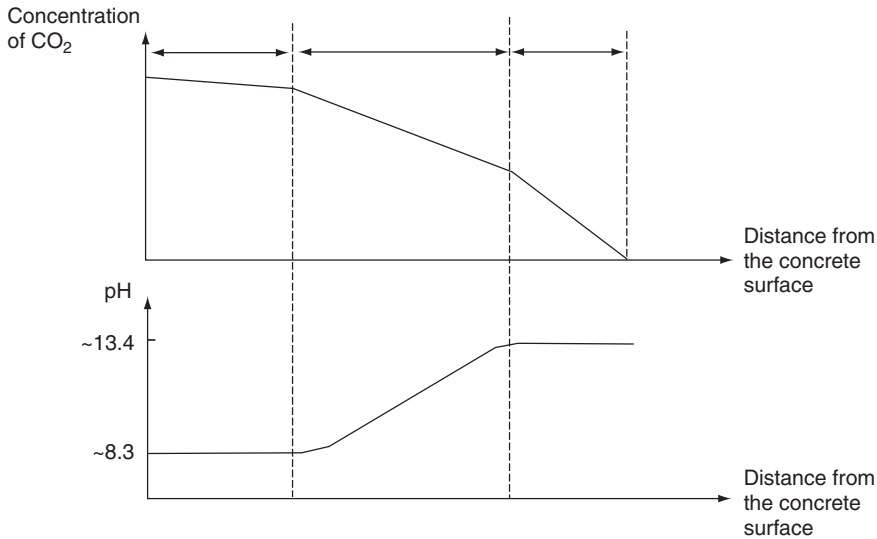




3.14 Reinforcement corrosion due to carbonation in concrete sculpture by Picasso (1967) in National Museum of Israel, Jerusalem.



3.15 Reaction of carbon dioxide in pore water with lime and alkaline constituents.



3.16 Simplified model of carbonation penetration.

However, not only is the free lime carbonated, lime–silica gel, CSH-gel, the main binding component, is also converted. This reaction does not affect concrete strength, but changes alkalinity and the pore system. In general, the pores in Portland cement (CEM I) concrete will become smaller. In the case of cements containing additions, like fly ash, however, this can result in a coarser pore system. Figure 3.16 schematically shows carbonation progressing into concrete. As soon as the alkalinity decreases below pH 10, steel will no longer be passivated. In the case of exposure to constant relative humidity and temperatures, the carbonation frontier follows Fick's second law and can be expressed as follows:

$$X = A\sqrt{t} \quad [3.10]$$

where: X = the penetration depth of the carbonation frontier

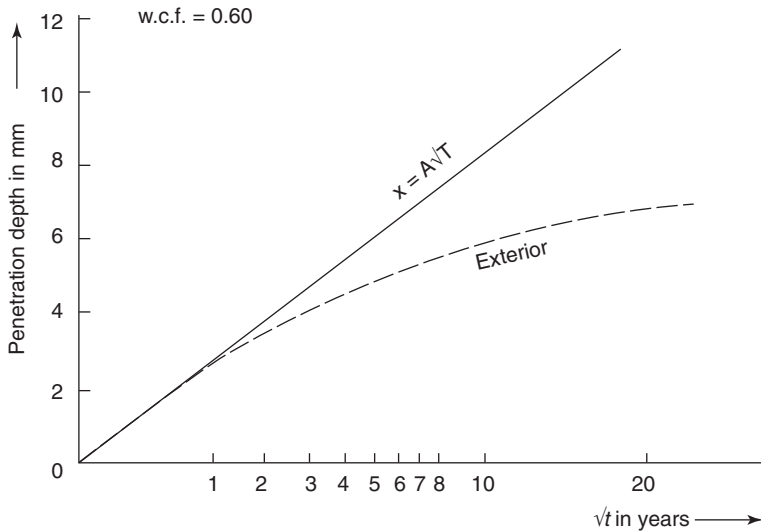
A = constant

t = time

Figure 3.17 illustrates this relationship. In practice, penetration occurs less quickly. This is due to the continuous changes in humidity and temperature, which slow down drying of the concrete and consequently carbonation.

3.4.2.2 Rate of carbonation, concrete parameters

The rate of carbonation depends on:



3.17 Penetration of carbonation frontier according to $x = A\sqrt{t}$ relation and the actual penetration progress of concrete exposed to exterior conditions, although protected from the rain.

- The water/cement ratio (the lower the slower).
- Cement type. In general, cements with a low initial strength develop carbonate faster because of the relatively low hydration rate and the relatively coarse pore system at the moment of exposure to drying conditions. Cements with slowly reacting pozzolans and latently hydraulic agents, such as powder coal fly ash and blast furnace slag, are more vulnerable to carbonation than is Portland cement CEM I.

The free lime content of the cement is also of importance.

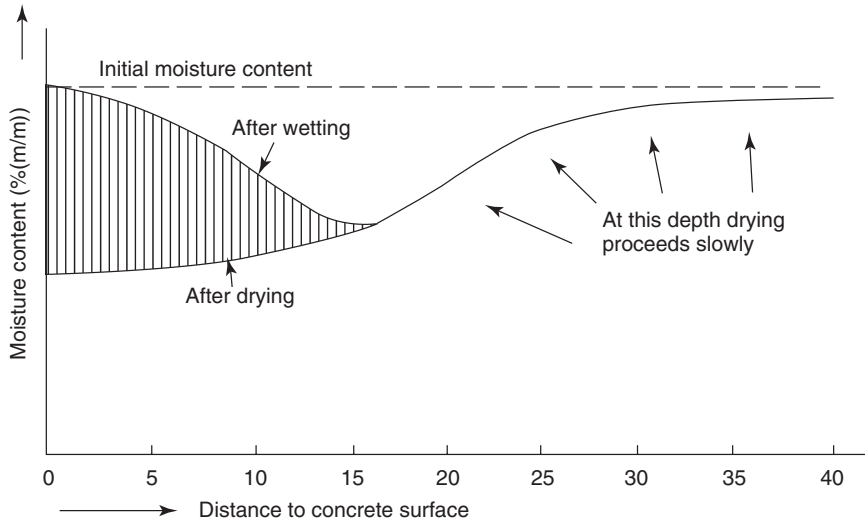
- Curing. Poor curing means rapid carbonation.

It has been observed that if the reinforcement cover meets the requirements, no significant corrosion occurs, even if the concrete has been carbonated beyond the reinforcement.¹⁰ It has been shown that the concrete around the reinforcement is too dry, thus maintaining a high ohmic resistance. The corrosion rate is significant only when the reinforcement is located in the fluctuating humidity zone, as shown in Fig. 3.18. In Western Europe, this zone does not exceed 15 mm.

3.4.3 Chloride ions

3.4.3.1 Penetration mechanism

Chloride ions are another cause of the destruction of passivation. Chloride ions can destroy the protecting passivation layer of iron oxides and iron



3.18 Fluctuation of moisture content in concrete cover exposed to exterior non-shade conditions.¹⁰

hydroxides locally. They compete with the hydroxyl ions in the pore water and corrosion is initiated beyond a certain $[\text{Cl}^-]/[\text{OH}^-]$ concentration ratio. According to Hausmann, the critical concentration ratio is 0.6 at 20 °C. Thus chloride ions can penetrate from the exterior if concrete is in contact with sea water or road structures in contact with de-icing salts. They can also be present in concrete from the very beginning, e.g. when calcium chloride is applied as an accelerator or when beach sand contaminated with salt is used as a fine aggregate.

In the case of wet concrete that is saturated with water, chloride penetration from the exterior can be calculated by Fick's second law:

$$\frac{\partial C}{\partial t} = D \frac{\partial^2 C}{\partial x^2} \quad [3.11]$$

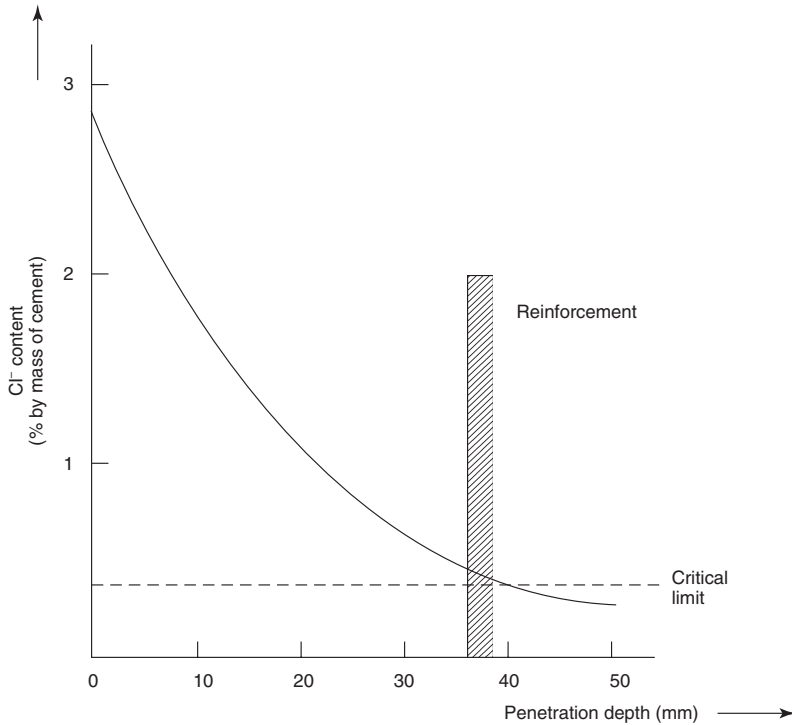
where: C = concentration

t = time

D = diffusion coefficient

When the chloride concentration at the concrete surface and the diffusion coefficient are known, the chloride concentration can be calculated at each location of the concrete, as well as the time left before corrosion occurs. A typical concentration profile in accordance with equation [3.11] is given in Fig. 3.19.

In reality, the penetration process is more complex. Apart from diffusion in the splash and spray zone of marine structures, chloride also penetrates when salt water is absorbed by concrete that has dried out. This is



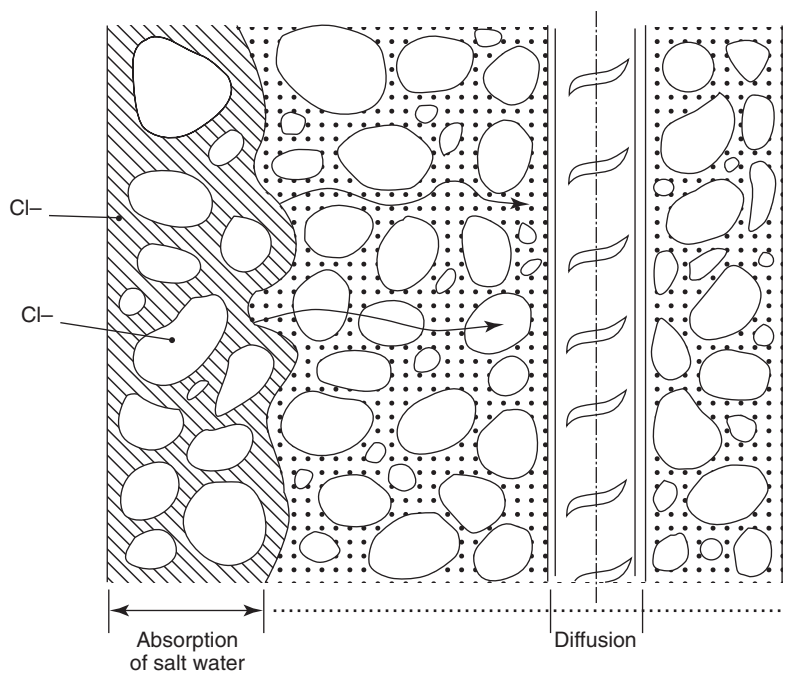
3.19 Typical chloride concentration profile.

schematically shown in Fig. 3.20. Transport then is much faster than transport by pure diffusion. After the water has been absorbed, concrete dries out again until attacked by a new splash of salt water. In Western European countries, penetration due to salt water absorption appears to be restricted to the outer 15 mm. Beyond that 15 mm, diffusion is the controlling mechanism. Models and computer programs are available to calculate chloride penetration, while taking account of these mechanisms.¹¹

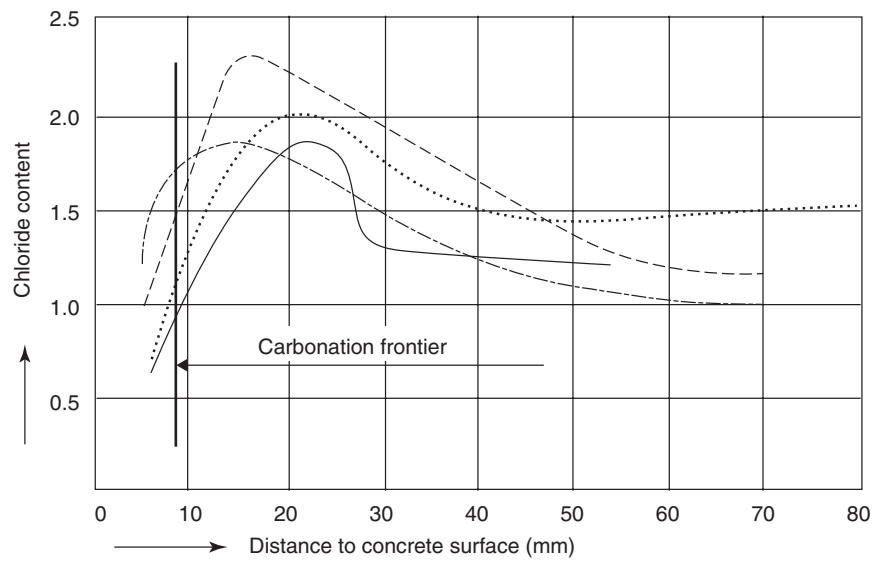
Another complicating factor in the modelling of chloride ingress in concrete is that part of the chloride is bound to the solid cement phase. In general, more than 50% of the chloride is bound. Only free chloride in the pore water poses a direct threat to the reinforcement. A further factor is that carbonation changes the pore structure and the binding capacity. This leads to a typical chloride penetration profile as shown in Fig. 3.21.¹²

3.4.3.2 Penetration factors

The chloride ion penetration rate is determined by the factors listed below.



3.20 Transport of chloride ions by suction and diffusion.

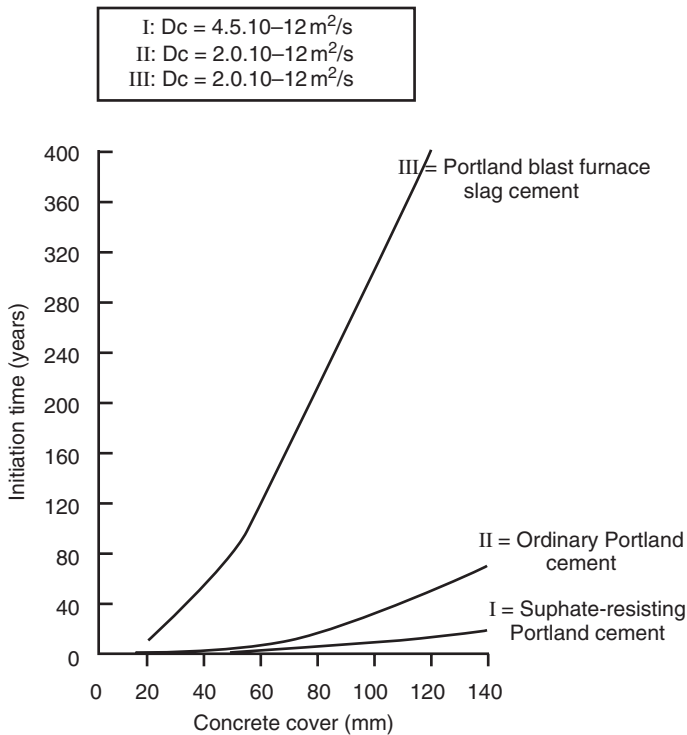


3.21 Influence of carbonation on chloride profile according to Tuutti.¹²

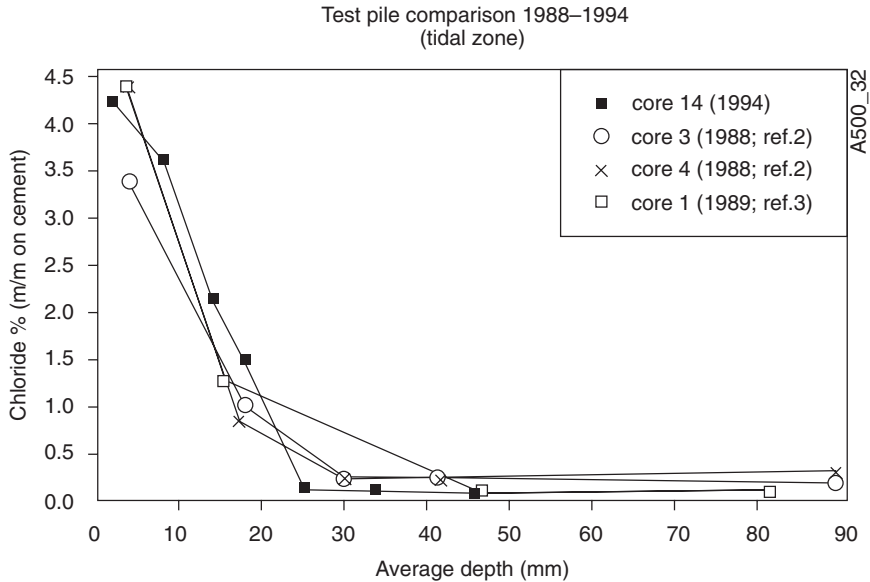
The cement type

The diffusion rate can be extremely slow in blast furnace cement CEM III/A, B, and C and Portland concrete with silica fume, yet it can be extremely fast in concrete with sulphate-resistant Portland cement of the same water/cement ratio and strength class. This is mainly due to differences in the pore structure of these hardened cements. The pore structure is coarse in sulphate-resistant Portland cement and fine in blast furnace slag cements and silica fume concrete.

Another difference is the capacity of hardened cements to bind chlorides, a capacity that is small in sulphate-resistant Portland cement. The effect on chloride initiation is shown in Fig. 3.22. It concerns a calculation made by the author in 1981 for the building of the 25 km long Saudi Arabia–Bahrain Causeway (nowadays called the King Fahad Causeway).¹³ It shows the



3.22 Illustration of estimated corrosion initiation time due to penetration of chloride ions at 35°C for concrete produced with sulphate-resistant Portland cement (CEM 1 HS), ordinary Portland cement (CEM I), and blast furnace slag cement with 65% by weight of slag (CEM III/B), respectively.¹³



3.23 Chloride concentration profiles of cores drilled from King Fahad Causeway test pier at 6, 6.5 and 12 years, respectively, after construction in the Gulf.¹³

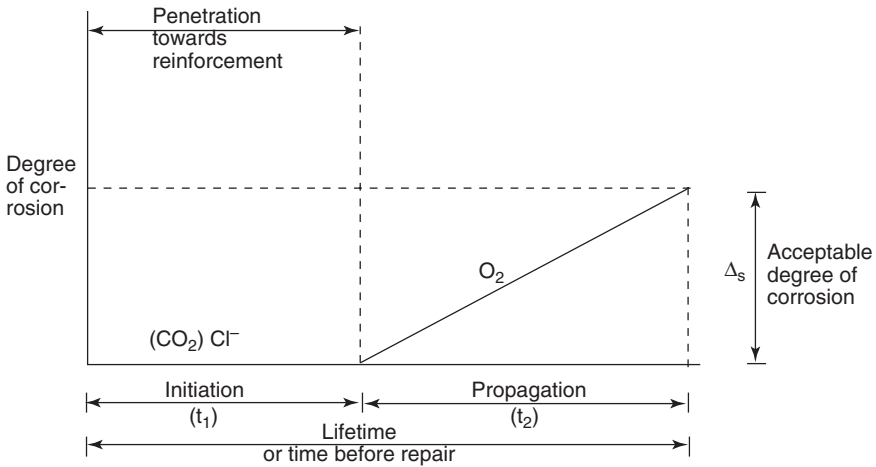
chloride initiation time of the concrete covering the reinforcement for 3 concrete compositions that all have a 0.4 water/cement ratio.

Temperature effects have been taken into account in the diffusion coefficients. Chloride initiation would have occurred within 10 years in the sulphate-resistant Portland cement with the 50mm cover as initially prescribed in the specifications. However, the client requested a service life of 75 years! The specifications were changed to blast furnace slag cement CEM III/B with about 70% of slag and the cover was enhanced.

Figure 3.23 shows chloride profiles of a test pile located along the bridge and measured over a 17 year period. Chloride appears to penetrate very slowly, even more slowly than predicted with the model based on Fick's second law. The use of sulphate-resistant Portland cement in coastal areas and marine structures has led to severe corrosion problems. Yet even today, some countries still promote the use of this type in marine environments. Paradoxically enough, there is no need for sulphate resistance in salt water containing chloride, as discussed in Section 3.2.3.1.

Qualified curing

If concrete dries early, cement hydration stops, the concrete carbonates and the affected layer stay porous, permeable and weak. This affected layer is



3.24 Schematic drawing of the corrosion process of steel in concrete.¹²

easy to penetrate. Such an effect can also be seen in the chloride profiles of Fig. 3.23. Penetration appears to have been fast for the first 10 mm and has subsequently slowed down considerably, perhaps even stopped.

The temperature

The higher the temperature, the faster penetration occurs. As the chloride diffusion coefficient increases, concrete dries out faster and the capacity to bind chloride decreases. Results from moderate climates should, therefore, be interpreted with caution when used in harsh climates.

3.4.4 Corrosion due to stray currents

Stray currents can also cause corrosion. The source of stray currents can be alternating current transmission lines from direct sources, such as from direct current traction systems, or cathodic protection systems that are operated in the vicinity of the reinforced concrete. To avoid corrosion due to stray currents, all reinforcement must be electrically connected and a special earthing system must be considered.

3.4.5 Corrosion propagation

Once the passivation layer dissolves, corrosion occurs and the corrosion propagation phase starts, as illustrated in Figure 3.24.¹² Even when corrosion has been initiated, the corrosion rate can be very low and negligible.

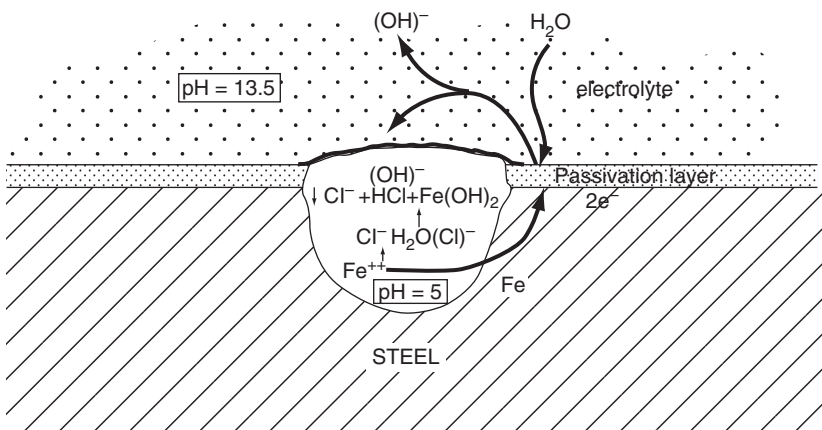
In reinforced concrete structures below sea level, for instance, critical corrosion is often greatly exceeded, but the corrosion rate is very low. This is due to the lack of oxygen. The underwater oxygen flow through concrete towards the reinforcement is very small. This causes oxygen diffusion polarization, as discussed in Section 2.2.5.2.

In the case of dry concrete, such as in houses, the corrosion current will generally be very low. This is due to a lack of water in the pores. Water is the conducting medium. It is often found that carbonation in houses has penetrated far beyond the reinforcement but there is no corrosion. Between the water-saturated situation and the dry situation, there is a moisture area where reinforcement will corrode. Here also the corrosion rate is influenced by the pore system. Concrete with small pores has a higher electrical resistance which decreases the corrosion current.

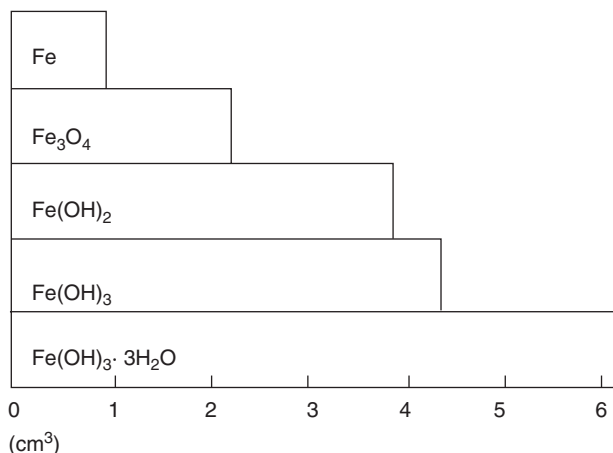
3.4.6 Corrosion types

In the case of carbonation, uniform corrosion occurs. The rate can be 80–150 μm per annum. Chloride causes pit corrosion (see Section 2.3.2.4). A microclimate develops in the pits, as shown in Fig. 3.25. The pit acidifies and attracts chloride, which makes the micro-environment even more corrosive. In the case of chloride-initiated corrosion, the ferro ions (Fe^{2+}) are complexed by chloride and can be transported away from the reinforcement. At a distance, they are oxidized further to ferri-oxides and ferri-hydroxides, showing the familiar rust colour. Sometimes rust stains can be observed at the concrete surface, even though the cover is adequate.

Rust has a substantially larger volume than iron, as shown in Fig. 3.26. This leads to expansion and often to delamination and cracking of the



3.25 Aggressive environment in chloride corrosion pit.



3.26 Volume of various corrosion products in comparison with iron.

concrete. This will naturally happen first at a location where the cover quality is lowest or the microclimate most aggressive. The first crack is also a first warning. It gives us some time to analyse the problem. In the case of chloride-initiated corrosion, this warning can be deferred because of the above-discussed phenomenon of transporting the chloride away from the reinforcement towards the surface.

3.4.7 Preventive measures

Even in the most aggressive natural climates, reinforcement corrosion can be avoided by the right choice of cement type, concrete composition, reinforcement cover and adequate curing quality. Carbonation is not discussed in this section. Corrosion of steel reinforcement in concrete can be avoided by simply following the standards in place and by meeting the requirements of minimum cover.

3.4.7.1 Cement type

The binder type is of the utmost importance, as illustrated in Fig. 3.22. Concrete with blast furnace slag cement (CEM III/B and CEM III/C), with a powder coal fly ash content of at least 30% fly ash, silica fume or other pozzolanic additions can show high resistance to chloride ingress. Maritime concrete structures in the Netherlands are almost exclusively built with blast furnace slag cement and their performance over a 75 year period is excellent.¹⁴ Other Western European countries also have a long and posi-

tive experience with this type of cement. The results shown in Fig. 3.23 for the King Fahad Causeway between Saudi Arabia and Bahrain indicate that performance can be excellent in hot maritime climates also.

Silica fume is used in many countries as an addition to manufactured high-durability concrete. An advantage is that the percentage of silica fume is generally lower than 10%, which makes it more easily tradable than slag cement with slag percentages of up to 80%. A disadvantage is that experience in practice is limited, while there are also some questions with respect to the critical corrosion concentration and the vulnerability to autogeneous shrinkage.

3.4.7.2 *Water/cement ratio*

Another important factor with respect to chloride penetration is the water/cement ratio, although the effect of the water/cement ratio is much less than that of the cement type. Thus the effect on the chloride diffusion coefficient of decreasing the water/cement ratio of 0.5 to 0.35 is about a factor of 4, while changing the binder from Portland cement, CEM I, to one of the binder types discussed in the previous section can make a difference of a factor of 50.

In marine concrete structures, the water/cement ratio specified is mostly lower than 0.45 and can be as low as 0.3. The use of superplasticizers and well-adjusted grain size distribution in the fine particle area are required to achieve these low water/cement ratios. Increasing the binder content is, of course, another possibility for decreasing the water/cement ratio. This increase is restricted by the increase in hydration heat, hydration development, shrinkage and creep of the concrete, which makes the concrete more sensitive to cracking.

A reduction of the water/cement ratio can also be achieved by applying controlled permeability formwork. A popular product has become a polypropylene liner on the formwork with the trade name Zemdrain, the water at the surface drains partly into the liner, which reduces the water/cement ratio of the skin of the concrete and consequently improves the carbonation and chloride penetration resistance.¹⁵

3.4.7.3 *Cover*

Figure 3.22 illustrates the importance of cover. In structures subject to chloride loading, the cover has to be at least 50mm thick. One has to be aware that the concrete skin is often of poorer quality than the concrete core due to imperfect curing and so a layer of up to 10mm that is affected and offers less resistance to chloride ingress is not unusual.

3.4.7.4 Curing

When concrete dries out, the cement stops reacting and the concrete carbonates. The earlier drying out occurs during cement hydration giving a more negative effect on concrete. Careful curing according to specifications is, therefore, of the utmost importance. In practice, this is not always observed.

Binder types with pozzolanic or latently hydraulic additions, such as powder coal fly ash and slag are usually more vulnerable to curing than is Portland cement, CEM I.

3.4.7.5 Additional preventive actions

In countries where Portland cements are applied exclusively and the use of cements with pozzolanic or latently hydraulic additions are not common, various preventive measures have been developed to protect the reinforcement against chloride-initiated corrosion. Examples of such preventive measures are the addition of inhibitors or the use of epoxy-coated rebars or of stainless steel reinforcement.

Inhibitors

Anodic inhibitors are applied in the United States. The most popular one is calcium nitrite, $\text{Ca}(\text{NO}_2)_2$, which is mixed into the concrete. Once the reinforcement starts corroding, nitrite oxidizes the ferro (Fe^{2+}) ions, into ferri (Fe^{3+}), thus forming a dense layer with hydroxyl ions, and this repairs the locally disrupted passivation to some degree.

Chloride ions and nitrite ions compete with each other. A sufficient concentration of nitrite has to be present so as to attain a certain level of protection. One of the drawbacks of nitrite is that if the chloride concentration exceeds the effective nitrite concentration, pit corrosion can be accelerated.

This uncertainty has probably restricted widespread use of calcium nitrite. Another reason for its restriction is the toxicity of nitrite. This has prompted the development of less toxic inhibitors. Justness claims¹⁶ that calcium nitrate has a good potential as an inhibitor but is non-toxic. Organic adsorptive types that curb both anodic and cathodic reactions are also on the market. Inhibitors are applied in North America but hardly at all in Europe.

Epoxy coated rebar

The use of the so-called fuse bonded epoxy coated rebar (powder coating) has become popular in North America. The system's durability is, however,

increasingly contested. It appears that small defects in the coating can harbour pit corrosion. Measuring techniques that determine the quality of the epoxy coated rebar are lacking.^{17,18}

Stainless steel rebar

Stainless steel is advocated as the universal remedy. A good grade of chloride-resistant stainless steel (see Section 2.4.3) can indeed be a technically sound solution, although an expensive one! As discussed, there is no need to use these expensive materials because the reinforcement is well protected by using plain concrete, applying the right cement type or addition, cover thickness, and water/cement ratio.

3.4.8 Prestressed concrete

3.4.8.1 *General*

Prestressing of concrete is an elegant technique to exploit the high compressive strength of concrete in order to alleviate its weakness: the low tensile strength. By loading the steel reinforcement in tension the concrete is put under compression. This technique is widely applied for building structures. Post-tensioning and pre-tensioning stressing are distinguished. The term prestressing is applied to both.

In pre-tensioning, steel is stressed in a stiff steel frame before the concrete is cast around it.

There are two main types of post-tensioning:

- Unbonded and bonded (grouted). An unbonded tendon is one in which the prestressing steel is not actually bonded to the concrete that surrounds it except at the anchorages. The most common unbonded systems are monostrand (single strand) tendons, which are used in slabs and beams for buildings, parking structures and slabs-on-ground. A monostrand tendon consists of a strand consisting of a number of wires coated with a corrosion-inhibiting grease and encased in an extruded plastic protective sheathing. The anchorage consists of an iron casting and a conical, two-piece wedge that grips the strand.
- Bonded systems. Here, two or more strands are inserted into a metal or plastic duct that is embedded in the concrete. When the concrete has developed a certain strength the strands are stressed with a large, multi-strand jack and anchored in a common anchorage device. The duct is then filled with a cementing grout that provides corrosion protection to the strand and bonds the tendon to the concrete surrounding the duct. Bonded systems are more commonly used in bridges, both in the superstructure and in cable-stayed bridges.

With respect to durability prestressed concrete has advantages in comparison with ordinary reinforced concrete:

- There are no cracks.
- There is less dynamic movement.

A disadvantage is that the durability of those structures depends strongly on the durability of the prestressing steel and that failure can occur without any warning.

3.4.8.2 *Deterioration mechanisms of prestressing steel*

The failures observed in practice are due to stress corrosion cracking and hydrogen embrittlement. Both can occur suddenly and without any warning as discussed in Sections 2.3.2.10 and 2.3.2.11.¹⁹ The vulnerability of the prestressing steel for these types of failure is higher than for ordinary reinforcement because high strength steels are in general more sensitive to stress corrosion and hydrogen embrittlement.

Corrosion can be initiated during the construction stage, during transport and on the site when the tendons are left unprotected; even contact with water can induce corrosion. Contact with salt water or salt-laden air in marine environments can be especially detrimental.

For the completed structure corrosion can be induced by:

- Chlorides; either in marine environments or where de-icing salts are applied. Hot marine environments represent the most severe types of exposure.
- Carbonation; the latter can occur if the grout in ducts is not filled completely or the cover is of low quality. The failure of a part of the roof of the Berlin congress hall in 1980 was due to hydrogen embrittlement after carbonation.²⁰

The anchorages of prestressing tendons are vulnerable, especially in the case of unbonded post-tensioned concrete, where stress transfer occurs in this zone.

3.4.8.3 *Prevention of corrosion*

Construction of prestressed concrete

In the case of post-tensioned bonded concrete measures should be taken to prevent early failure during the construction state. The tendons should be protected from water and condensation during transport, storage and the period in the post-tensioning ducts when tendons are not protected by the grout. Examples of protection systems are packing in watertight plastic

with enclosed siccatives or a vapour-based inhibitor and use of greases that should be compatible with the grout applied. When the tendons are placed in the ducts no leakage of water should occur; and there should not be bleeding water from the surrounding concrete, because even this alkaline water can easily be neutralized by carbonation and then cause corrosion. The tendons should not be left long unprotected by the grout in the duct.

In the case of grouting a full and complete filling up of the duct and complete embedment of the tendons are essential. The grouting technology is highly developed and it requires skilled labour to execute the grouting operation. The duct should be filled and full embedment of the strands should be achieved. Naturally, grouts should not contain any corrosive compounds such as chloride and thiocyanates.

Structures in use

In the case of aggressive environments concretes should be used with a high penetration resistance to chlorides and high cover (see Section 3.4.7). For post-tensioned members the use of non-corroding dense ducts, such as polyethylene ducts, help to halt the progress of chlorides towards the tendons. This was, for instance, successfully applied in the building of the King Fahad Causeway.²¹

Special attention should be given to joints between prestressed members and to the anchorages of the tendons. Anchorage appears to be often vulnerable to chloride penetration along the interface of the filled-in interface of box-outs. Good adherence of the box-out fill up mortar and the concrete should be warranted. Especially vulnerable are the anchorages of unbonded post-tensioned systems, because they are the sole areas of stress transfer between tendons and concrete. Applying sufficient cover, a low water/cement ratio and good curing can prevent carbonation problems. It should be realized that repair of affected prestressed systems is difficult, if not impossible, to carry out (see Section 7.3.3 for further information).²²

3.5 Principles of protection and repair of concrete structures

3.5.1 General

The principles for protection and repair of concrete structures as given in EN 1504, Part 9, *General principles for the use of products and systems* are followed in this chapter. Protection systems and repair products are discussed in Sections 3.6, 3.7, 3.8 and 3.9 and two practical repair cases are given in Section 3.10. EN 1504 distinguishes between principles and

methods related to defects in concrete and those related to reinforcement corrosion. The emphasis will be on the latter in this section. The repair of defects in concrete structures will be discussed briefly.

3.5.2 Principles and methods related to defects in concrete

EN 1504, Principles 1 to 7, listed in Table 3.6, cover defects in concrete or concrete structures caused by the following actions that take place either separately or in combination:

- 1 Mechanical actions, e.g. impact, overloading, movement caused by settlement, and blast.
- 2 Chemical and biological actions from the environment.
- 3 Physical actions, e.g. frost–thaw action, thermal cracking, moisture movement, salt crystallization and erosion.

Principle 1 [PI]; protection against ingress

Cracks

Crack widths that do not exceed the limit, for instance 0.2 or 0.3 mm, and that are kept in check by the reinforcement are allowed in many reinforced concrete applications. Cracks may form for several reasons, e.g. due to mechanical loading, plastic shrinkage, thermal contraction, hydration heat or as a result of overloading.

It must then be established which are the causes of cracking and the effect on the structure of ingress or seepage. If an adverse effect is found, treatment can be carried out using one or more of Methods 1.1 to 1.7, given in Table 3.6. Thus a crack below the water table will not have an adverse effect on reinforced concrete durability, but a similar crack above the water table can be detrimental. A crack due to thermal contraction of concrete may not affect the performance of a large foundation slab, but it does affect the water-tightness of a tunnel.

Note: Some cracks form in hardened concrete as a result of corrosion that is already under way. These cracks usually form over the reinforcement and are often the first visual sign that there is a corrosion problem. Cracks caused by corrosion cannot be stopped simply by filling or sealing. These defects should be repaired by methods that take account of the corrosion cause, according to Principles 7 to 11, as discussed in Section 3.5.3.

The possibility of further cracking after applying Methods 1.2 and 1.4 must be taken into consideration. Information on live and dead cracks is given in EN 1504–5. In the case of live cracks, account must be taken of the fact

Table 3.6 Principles and Methods related to defects in concrete (EN 1504 Part 9)

Principle No	Principle and its definition	Methods based on the principle
Principle 1 [PI]	Protection against ingress Reducing or preventing ingress of adverse agents, e.g. water, other liquids, vapour, gas, chemicals and biological agents.	1.1 Impregnation Applying liquid products that penetrate the concrete and block the pore system. 1.2 Surface coating with and without crack bridging ability 1.3 Locally bandaged cracks^a 1.4 Filling cracks 1.5 Transferring cracks into joints^a 1.6 Erecting external panels^{ab} 1.7 Applying membranes^a
Principle 2 [MC]	Moisture control Adjusting and maintaining the moisture content in concrete within a specified range of values.	2.1 Hydrophobic impregnation 2.2 Surface coating 2.3 Sheltering or over-cladding^{ab} 2.4 Electrochemical treatment^{ab} Applying a potential difference across sections of the concrete to assist or resist the passage of water through concrete. (Not for reinforced concrete without assessment of the risk of inducing corrosion.)
Principle 3 [CR]	Concrete restoration Restoring the original concrete of a structural element to the originally specified shape and function. Restoring the concrete structure by replacing part of it.	3.1 Applying mortar by hand 3.2 Recasting with concrete 3.3 Spraying concrete or mortar 3.4 Replacing elements
Principle 4 [SS]	Structural strengthening Increasing or restoring the structural load-bearing capacity of an element of the concrete structure.	4.1 Adding or replacing embedded or external reinforcing steel bars 4.2 Installing bonded rebars in preformed or drilled holes in the concrete 4.3 Plate bonding 4.4 Adding mortar or concrete 4.5 Injecting cracks, voids or interstices

Table 3.6 Continued

Principle No	Principle and its definition	Methods based on the principle
Principle 5 [PR]	Physical resistance Increasing resistance to physical or mechanical attack.	4.6 Filling cracks, voids or interstices 4.7 Prestressing – (post tensioning)^a 5.1 Overlays or coatings 5.2 Impregnation
Principle 6 [RC]	Resistance to chemicals Increasing resistance of the concrete surface to deteriorations by chemical attack.	6.1 Overlays or coatings 6.2 Impregnation

^aThese methods may make use of products and systems not covered by the EN 1504 series and are not discussed in this book

^bInclusion of methods in EN 1405 does not imply these methods have been approved

that completely filling up those cracks will always lead to new cracking within the crack filler on the crack filler/concrete interface or within the concrete. Injection products and methods are discussed in Section 3.7.

Principle 2 [MC]; moisture control

General

Principle 2 is relevant when the moisture content in concrete should be lower than a critical value beyond which deterioration could occur. All Principle 2 methods require the carrying out of a maintenance programme in accordance with EN 1504, Part 10. Since the protection systems may not last for the design life of the concrete structure, renewal and repairability are important considerations in the choice of protection system. Methods and products are discussed in Sections 3.8 and 3.9.7.

Control of alkali–aggregate reactions

The main cause of alkali–aggregate reactions, besides the presence of significant quantities of alkali–reactive aggregate particles and alkalis, is the presence of sufficient quantities of moisture. By reducing moisture to a controlled low limit, alkali–silica gel will not be able to expand sufficiently to cause the concrete to crack.

The alkali–aggregate reaction takes place over a certain period of time, after which the rate of expansion tends to decrease. After this period, protection and repair may be of the same type as for concrete with non-expanding cracks. In order to make sure that the cracks are of the non-expanding type, the crack width should be measured over an adequate period of time.

Protection against frost–thaw damage

Where defects are caused only by the more severe frost–thaw action of salt water, and not by fresh water, protection can be achieved in accordance with Principle 1 [PI] by preventing ingress of chlorides.

Principle 3 [CR]; concrete restoration

The mortar or concrete can be cement-bound, polymer-modified cementitious, or polymer-bound (see Section 3.6).

Principle 4 [SS]; structural strengthening

It is essential when using Principle 4 that repairs should never produce stresses that are harmful to the original structure. Method 4.7, where tendons are installed through or around a concrete structural element and subsequently tensioned so as to confine the element or carry the loads applied to it, can create internal stresses which increase the effects of deterioration. With the exception of the strengthening of structures with fibre-reinforced composites (Section 5.4), methods for structural strengthening are not discussed in this book. Reference is made to the relevant literature.^{23,24}

Principle 5 [PR]; physical resistance

Methods 5.1 ‘overlays or coatings’ and 5.2 ‘impregnation’ that serve to strengthen the physical resistance of the concrete surface are discussed in Section 3.8.

Principle 6 [RC]; resistance to chemicals

Concrete in accordance with EN 206 and products and systems in accordance with the EN 1504 series can be used for concrete that has been attacked in the past by chemicals or is being attacked at present by those chemicals covered by the provisions of EN 206 (see Table 3.1). Products and systems for protecting concrete are discussed in Section 3.8.

3.5.3 Principles and methods related to reinforcement corrosion

According to EN 1504, Part 9, Principles 7 to 11, in Table 3.7 cover reinforcement corrosion is caused by:

- 1 Physical loss of the protective concrete cover.
- 2 Chemical loss of alkalinity in the protective concrete cover as a result of the reaction with atmospheric carbon dioxide ('carbonation') (Section 3.4.2).
- 3 Contamination of the protective concrete cover with corrosive agents (usually chloride ions) that were incorporated in the concrete when it was mixed or that have penetrated into the concrete from the environment (Section 3.4.3).
- 4 Stray electrical currents conducted or induced in the reinforcement from neighbouring electrical installations (Section 3.4.4).

In general, corrosion caused by chloride ingress is more difficult to counteract than corrosion by carbonation. Protection and repair in situations where reinforcement corrosion poses a risk are based on the Principles below. Where reinforcement corrosion exists or there is a risk that corrosion will occur in the future, one or more of the corrosion protection or repair Principles 7 to 11 below, shall form the basis of the method applied. In addition to the above, the concrete itself shall be repaired where necessary, according to Principles 1 to 6.

Principle 7 [RP]; preserving or restoring passivity

General

All methods of restoring passivity are effective only to a certain depth and there may be a risk of contamination from the environment and adjacent contaminated concrete.

Method 7.1 increasing cover to reinforcement with additional cementitious mortar or concrete

By increasing the cover with a cementitious mortar or concrete an additional barrier against ingress is shaped. Furthermore, in the case of carbonation by diffusion re-alkalization can occur both from the additional cover as well as from the background concrete, see also Method 7.4. Chloride profiles existing in the original cover will level out not only towards the core of the concrete in the direction of the reinforcement but also towards the additional cover which decreases the threat for the reinforcement.

Table 3.7 Principles and Methods related to reinforcement corrosion (EN 1504 Part 9)

Principle No	Principle and its definition	Some examples of methods based on the principle
Principle 7 [RP]	Preserving or restoring passivity Creating chemical conditions in which the surface of the reinforcement is maintained in, or is returned to, a passive condition.	7.1 Increasing the reinforcement cover with additional cementitious mortar or concrete 7.2 Replacing contaminated or carbonated concrete 7.3 Electrochemical re-alkalization of carbonated concrete^a 7.4 Re-alkalization of carbonated concrete by diffusion 7.5 Electrochemical chloride extraction^a
Principle 8 [IR]	Increasing resistivity Increasing the electrical resistivity of the concrete.	8.1 Limiting moisture content by surface treatments, coatings or sheltering
Principle 9 [CC]	Cathodic control Creating conditions in which potentially cathodic areas of reinforcement are unable to drive an anodic reaction.	9.1 Limiting oxygen content (at the cathode) by saturation or surface coating^b
Principle 10 [CP]	Cathodic protection	10.1 Applying electrical potential^a
Principle 11 [CA]	Control of anodic areas Creating conditions in which potentially anodic areas of the reinforcement are unable to take part in the corrosion reaction.	11.1 Applying reinforcement coatings containing active pigments 11.2 Applying barrier coatings to the reinforcement 11.3 Applying inhibitors to the concrete^{ab}

^aThese methods may make use of products and systems not covered by the EN 1504 series

^bInclusion of methods in EN 1504 Part 9 does not imply these methods have been approved

Method 7.2 replacing contaminated or carbonated concrete

In some cases, it may be necessary to replace both reinforcement and concrete. Attention should be given to the possibility that adjacent to the repair area the concrete can still be contaminated, which later on can lead to corrosion. It is possible that the repaired area can act as a cathodic area, which aggravates the corrosion progress around the repaired area. This is known as the incipient anode effect. It is often desirable to protect the repaired and adjacent concrete against ingress, as outlined in Principle 1. This is one of the principles applied to the repair of the Deira–Shindagah tunnel, as discussed in Section 3.10.1.

Method 7.3 electrochemical re-alkalization of carbonated concrete

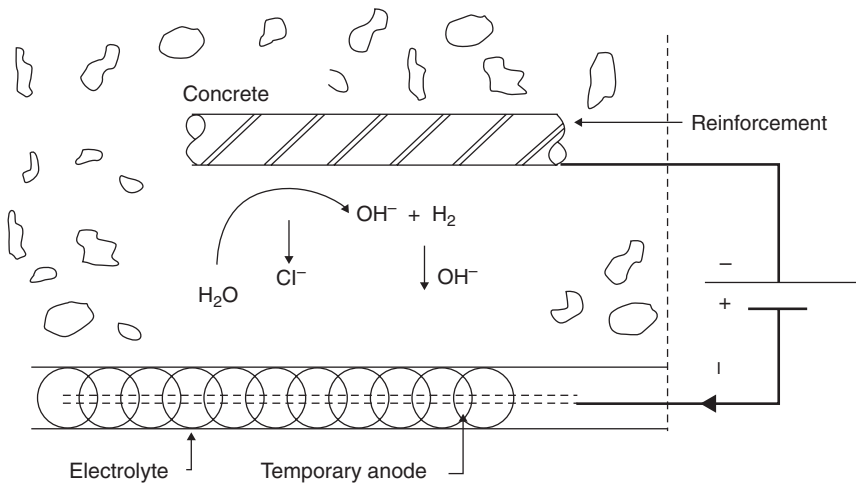
In this method a strongly alkaline paste (soda, cellulose fibres, gel etc.) is applied on the surface of the concrete. An anode mesh is embedded in the paste and electrically connected to the reinforcement via a DC source. A relatively high voltage is applied. The alkaline solution is drawn into the concrete due to electrolytic osmotic forces. Concurrently also the pH is raised by the cathodic reaction at the reinforcement. Because the alkaline buffer built up is small the method has to be completed with a CO₂ resistant surface treatment.

Method 7.4 re-alkalization of carbonated concrete by diffusion

This method requires the application of cementitious concrete or mortar to the surface of carbonated concrete so that it can be re-alkalized through diffusion, see Method 7.1. It depends upon conditions that permit sufficient diffusion and on the ability of the repair layer to prevent carbonation at the reinforcement throughout the required design life of the member. It should be noted that in case of carbonated concrete around the reinforcement, repassivation is possible by increasing the cover with a concrete layer or a barrier coating. Moreover, as soon as moisture fluctuations cease, the corrosion rate will slow down substantially.

Method 7.5 electrochemical chloride extraction

Figure 3.27 shows schematically the extraction progress of chlorides from concrete. The voltage applied over the temporary anode at the concrete/reinforcement surface is substantially higher than in the case of cathodic protection. Electrochemical chloride removal is only possible for concrete situated between the reinforcement and the surface and immediately surrounding the reinforcement. There is a risk of recontamination of this concrete from ions within the concrete from behind the reinforcement.



3.27 Extraction of chlorides from concrete.

Principle 8 [IR]; increasing resistivity

Principle 8 [IR] requires the moisture content of the concrete to be low enough to restrict the flow of ions to a very low value. Inside dry buildings, corrosion is seldom a problem, even if concrete is carbonated at the reinforcement depth. This is because the low moisture content in enclosed buildings tends to raise concrete resistivity to a level where the corrosion rate is insignificant.

Under suitable circumstances, it may be possible to create similar conditions externally by reducing the moisture content through the application of ventilated external claddings, water-repellent surface treatments or pore-filling impregnation or surface coatings. When such methods of protection are used, it is necessary to consider the specific application in detail so as to ensure that moisture cannot be created by condensation at cold spots and that it cannot enter the concrete from the ground, faulty drainage or other sources. Moisture must, however, be allowed to escape freely.

Principle 9 [CC]; cathodic control

Principle 9 [CC] requires the access of oxygen to all potentially cathodic areas to be restricted to the point where corrosion cells are disabled and corrosion is prevented by inactivity of the cathodes.

Saturation of the whole of a self-contained reinforced concrete unit, with a reinforcement that has no electrical connection with reinforcement in any other unit and that has oxygen access, is an example of how this principle

can be applied. Surface coatings can have a similar effect, if the coating is guaranteed to be complete and effective.

Limiting the oxygen content (at the cathode) by saturating the concrete should be used only where the whole of the member is under water and where reinforcement of the submerged member is electrically isolated from all reinforcement in members that are not submerged, or where there is no effective return path for ionic currents through the concrete. This is also one of the principles applied to the repair of the Deira–Shindagah tunnel, as discussed in Section 3.10.1.

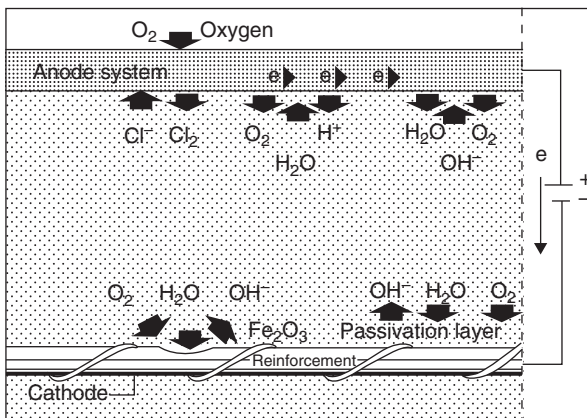
Principle 10 [CP]; cathodic protection

Cathodic protection (see Section 2.2.5.4 for background and Section 3.9 for methods) can be especially appropriate where chloride contamination threatens to cause reinforcement corrosion, but little or no physical damage has occurred. It can also be used where physical damage has occurred but it is not economic or practical to remove physically-sound but chloride-contaminated concrete. Figure 3.28 illustrates electrochemical reactions and transport of reactants and reaction products in reinforced concrete with a CP system. Where cathodic protection is used, sound concrete should be left undisturbed wherever possible.

Principle 11 [CA]; control of anodic areas

Method 11.1 applying reinforcement coatings containing active pigments

Active pigments can function as anodic inhibitors or cause a sacrificial galvanic reaction.



3.28 Electrochemical reactions in reinforced concrete with a CP system.

Inhibitors are chemical agents that discourage the formation of anodic regions on the reinforcement.

There is controversy about the medium- and long-term effectiveness of this type of treatment.

Coating the reinforcement with a substance that is an electrical conductor with a lower electrode potential than steel causes the cations of the coating material (rather than the steel) to dissolve under corrosive conditions. Ions of the coating metal are also attracted to any accidentally exposed (negatively charged) parts of the steel, at which locations they repair the protective layer. A typical example is the use of zinc powder as a filler. Other methods include phosphating with diluted phosphoric acid, followed by a sealing coat (see also Section 2.5.3).

Method 11.2 applying barrier coatings to the reinforcement

Isolating the reinforcement from the surrounding concrete with a coating that is an electrical insulator should prevent metal cations from escaping from the steel and should also prevent incoming anions from being discharged there. This method can only be effective if the steel is clean and the coating complete, i.e. the bar must be completely encapsulated and the coating undamaged (see Section 2.5.3). The method should only be contemplated if the whole of the circumference of the bar can be coated. The effect of coating on the bond should be considered. A barrier coating of the reinforcement was applied in the repair of the Deira–Shindagah tunnel, as discussed in Section 3.10.1. It should be emphasized that effective repair should not depend solely on a barrier coating applied on the reinforcement in a marine environment when the problems with epoxy-coated rebar reinforced concrete are recognized.^{17,18}

Method 11.3 applying inhibitors to concrete by impregnation or diffusion

Inhibitors can be applied as a surface treatment or by electrochemical means. They may also be added to repair products and systems. Inhibitors act by chemically changing the surface of the steel or by forming a passive film on top. Inhibitors applied to the surface of the concrete rely on their ability to penetrate the concrete down to the steel level so as to be effective, given the fact that concrete is permeable to inhibitors that generate high vapour pressure and have good diffusion characteristics. This method is under development and will not be discussed in this book.

3.6 Repair mortars

3.6.1 Classification

Repair mortars are classified here according to binder type:

- Cement-bound mortars are referred to as cement concrete (CC) mortars. These are cement-bound mortars with less than 5% of polymers and other organic admixtures in relation to the cement mass.
- Polymer-modified cement-bound mortars are referred to as polymer-cement concrete (PCC) mortars. These are cement-bound mortars with 5 to 20% of polymer in relation to the cement mass.
- Resin-bound mortars are referred to as polymer concrete (PC) mortars. The resin is the only binder in these mortars.

These mortars contain aggregates, additions, fillers and admixtures.

3.6.2 Cement-bound mortars, CC

Cement-bound mortars are applied by casting, trowelling and spraying.

3.6.2.1 *Casting*

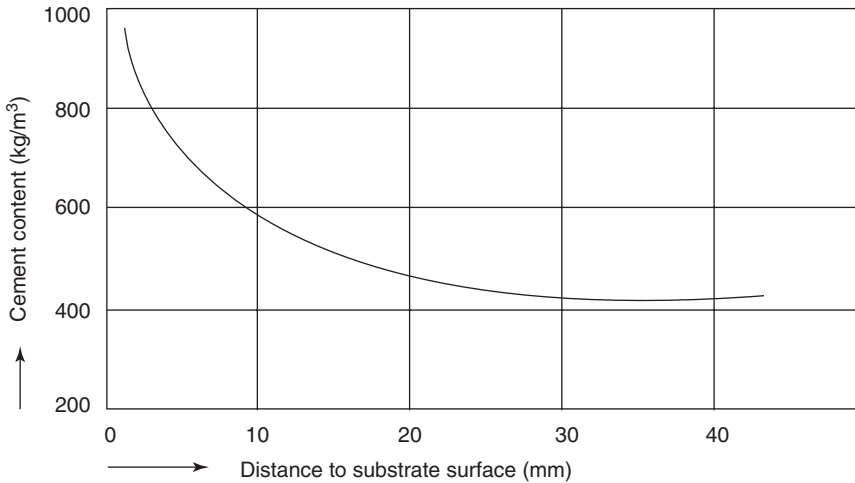
Casting is used for larger repairs of horizontal and vertical surfaces, for instance when a full edge of a balcony is repaired, as will be discussed in Section 3.10.2. A problem with casting application can be the adhesion to the old concrete. Due to shrinkage of the repair mortar, shear or tensile forces are generated at the interface and debonding occurs occasionally.

Sometimes non-shrinkage or swelling mortars are advocated to avoid the drying shrinkage stresses of cementitious mortars. However, one must be aware that these mortars swell during the initial phase of hardening, when the Young's modulus is still low. Drying shrinkage will still occur after swelling and will only be compensated by previous swelling when compressive stresses have built up in the mortar. This will only occur to a substantial degree when mortar is enclosed in all directions, which is usually not the case when repairing.

3.6.2.2 *Sprayed concrete*

Sprayed concrete is often applied in repair work. A distinction can be made into dry sprayed concrete (dry shotcrete or gunite concrete) and wet sprayed concrete.

In the dry process, the solid constituents are mixed dry. The dry mixture is pumped to a spray nozzle where water and other fluid constituents are



3.29 Cement content as a function of the distance to the substrate surface for dry sprayed concrete.

added. The mixture is then sprayed against the substrate with great force. Part of the material is rebound, especially the larger particles of the mortar. This leads to a cement-rich layer at the substrate surface, as shown in Fig. 3.29, which improves adhesion.

Wet sprayed concrete has become more popular because the working conditions are better than for the dry sprayed concrete. Rebound and dust development are substantially less.

In sprayed concrete, adhesion strength on the substrate is an important property. A typical requirement is an average adhesion strength of at least 1.3 N/mm^2 with a minimum value of 0.6 N/mm^2 .

3.6.3 Polymer-modified cement mortars, PCC

To improve the rather poor performance of repairs with cement/sand patching mortars and, simultaneously, maintain their positive effects, especially the (re)passivating capacity and the fact that cement mortars have properties such as Young's modulus and linear expansion coefficient that are similar to the concrete substrate, polymer-modified mortars (also called latex-cement mortars) have grown popular in repair work.

These polymers consist of elastomeric polymers or thermoplastic polymers such as polyvinylidene chloride, polyvinyl propionate, styrene butadiene, styrene acrylates and polyvinyl versatate. Thermosetting resins are also used, mainly water-emulsified epoxy resins. When applied with a percentage of less than 5% by weight of cement, these mortars are referred to as

polymer-modified mortars. When the polymer content is higher, they are referred to as polymer cement concrete (PCC). In the case of epoxies, the name epoxy cement concrete (ECC) is used. The polymers have to meet various requirements that include alkaline resistance. Some of the polymers used when the process was being developed such as polyvinyl acetate, were not alkaline-resistant, which caused early failure in some cases. Polyvinylidene chloride mortars are known to release chloride ions that have induced reinforcement corrosion in some projects in the US. The polymers' halogen content (with the exception of fluoride) must, therefore, be low.

Polymers must have a water-reducing effect, so that mortars with polymers will have a lower water/cement ratio than those without. Polymers must have a film temperature lower than the temperature during application. In Western Europe this means that the film temperature must not exceed 10°C. Polymers must not introduce large quantities of air. If they do, a defoaming agent must be applied.

In the literature, large variations in properties of polymer cement concrete and polymer-modified concrete are reported, depending on factors such as polymer type, polymer content and water/cement ratio. Table 3.8 illustrates this.¹

Polymers increase the strain capacity of the mortars. In dry conditions, the tensile strength is increased up to polymer contents of about 10% by weight of cement. Beyond that level, it decreases again. When wet, the tensile strength hardly increases. The Young's modulus of polymer-modified concrete decreases with increasing polymer contents. Adhesion to substrates could be greatly increased by applying appropriate polymers. An adhesion strength equal to the tensile strength of the substrate concrete can be achieved. In general, the performance of these polymer-modified mortars is much better than that of traditional purely cement-based mortars, mainly because of better adhesion, less drying shrinkage stresses and higher strain capacity. Moreover, polymers make mortars less sensitive to the curing quality.

Table 3.8 Variation in properties of thirty repair mortars¹

Property	Measure
Compressive strength	14–69 N/mm ² (MPa)
Bending strength	6–14 N/mm ² (MPa)
Young's modulus	8–25 kN/mm ² (GPa)
Drying shrinkage	0.3–2.0 mm/m
Adhesion strength	1.3–3.6 N/mm ² (MPa)
Coefficient of linear expansion	10–17 × 10 ⁻⁶ /K

In some countries, the use of polymer-modified sprayed concrete is common practice, such as in Germany. It is referred to as SPCC.

3.6.4 Resin bound mortars, PC

Polymer-bound mortars or polymer concrete (PC) are often used in hand-patching. Epoxy resins, acrylics or polyurethanes are usually used as a binder in these mortars. Polyester resins are not recommended because of their high hardening shrinkage, their moisture sensitivity as well as their proneness to saponification when applied to concrete. Adhesive primers are always applied in this type of mortar. These often contain anti-corrosive agents if treating the reinforcement, although it is not certain that the use of these agents is necessary.

The patching operation is rather critical because polymer mortar has to be applied to the polymer primer when wet.

In a field study in the Netherlands, most of the polymer mortars investigated showed debonding, this was due to insufficient preparation of the substrate. Certain other factors, such as differences in temperature-expansion and exceeding the glass-rubber transition temperature of the epoxy, could also have contributed to this. It is often thought that applying a PC mortar on a cleaned-up rusted bar will prevent renewed corrosion but this is not usually the case. Open space is always present around the bar and the reinforcement is not passivated.

3.7 Crack repair methods

3.7.1 General

Cracks must be repaired for the following reasons:

- To reduce or prevent ingress of adverse agents, e.g. water, other liquids, vapour, gas, chemicals and biological agents; Principle 1 in Table 3.6.
- To increase or restore the structural load-bearing capacity of an element of the concrete structure; Principle 4 in Table 3.6.

While crack injection agents are usually applied other measures can or must be taken occasionally in order to repair a crack. In deciding upon the method to be applied, much depends on which function must be restored and whether or not the cause of cracking is still present and may be reactivated. In the case of live cracks, one must be aware that completely filling up those cracks by injection will always lead to new cracking within the crack filler, on the interface with the cracked concrete or within the old

concrete.*¹ In such cases, increasing or restoring the structural bearing capacity is not possible using the methods described in this section and structural strengthening with tendons or plate bonding must be considered.

3.7.2 Crack injection agents

Requirements for concrete crack injection products are specified in EN 1504, Part 5. The product used depends not only on the function to be discharged, but also on the conditions of the crack, notably the presence or absence of water. The type of crack must be distinguished; it can be dry, humid, water transporting without pressure or water transporting under pressure. Under the more complicated conditions of water pressure, water ingress at the crack is closed off first, for instance by a polyurethane resin that forms a foam in contact with water. Subsequently, the crack is filled up with a massive resin. Table 3.9 presents a survey of the injection agents applied.²

3.7.3 Other methods that prevent ingress through cracks

In the case of cracks that have to be sealed off to reduce or prevent ingress of adverse agents, the following methods can be used to seal off the joints:

- Applying elastic sealants. Cracks can be widened at the surface to reduce the stresses in the sealant due to movements of the concrete, as shown in Fig. 3.30.
- Sealing at the surface with flexible rubber strips, as shown in Fig. 3.31.

3.8 Protective surface treatments

3.8.1 General

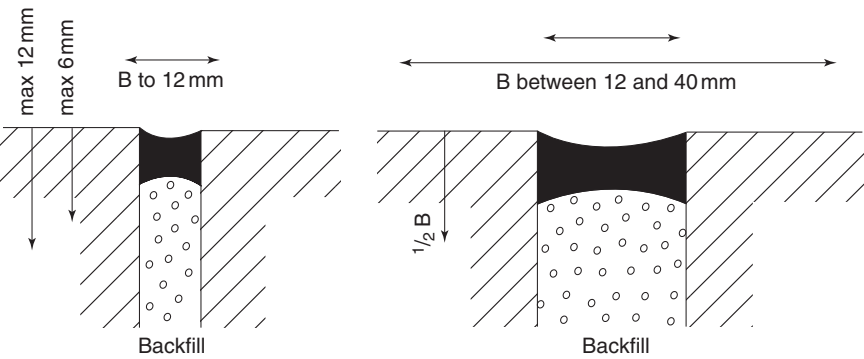
A survey of concrete surface protection systems is given in this section. Such issues as why and how to protect the surface, the general requirements regarding surface protective agents and commercially available agents are considered.

*It is often thought that filling up of the crack with a flexible injection agent will solve the problem. However, materials that show flexible behaviour (low modulus of elasticity, high deformation capacity and elastic behaviour) in uniaxial loading behave inflexibly if three-dimensionally restrained as in a crack.

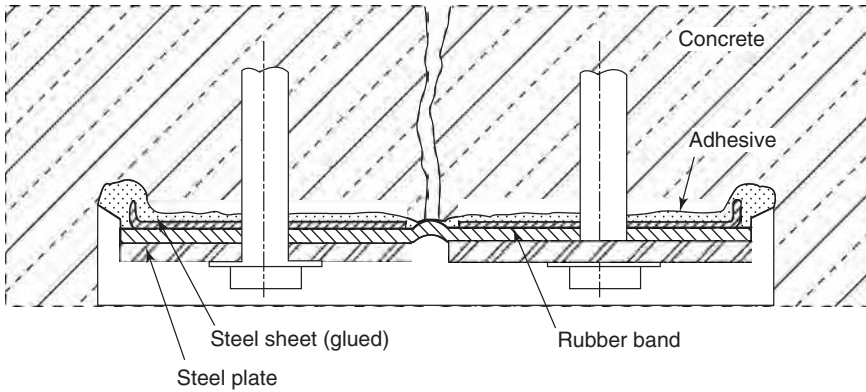
Table 3.9 Survey of crack injection agents in relation to crack condition and application²

Application	Concrete condition ^a		Water transporting	
	Dry	Humid	Without pressure	With pressure
Load-bearing capacity	EP ^b	EP ^{bc}	EP ^{bcd}	EP ^{bcd}
Prevent ingress	EP EP/PUR PUR – CC	EP ^c EP/PUR PUR GEL ^e CC	– EP/PUR PUR GEL CC	– EP/PUR PUR GEL CC

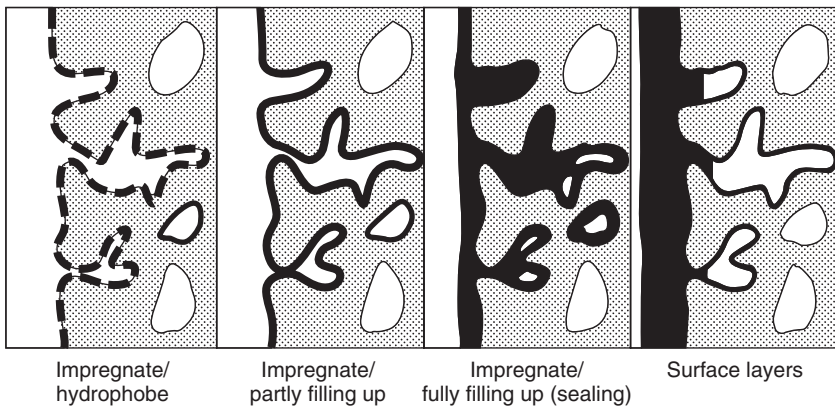
- Not applicable
- EP Epoxy
- EP/PUR Mixture of epoxy and polyurethane
- PUR Polyurethane
- GEL Watery gel bound by polyurethane or acryl amide
- CC Cement suspension
- ^aTo be determined by visual observation
- ^bOnly applicable when crack is not 'live'
- ^cNon-watersensitive epoxy
- ^dOnly applicable after water ingress has ceased by applying a foaming polyurethane or other similar agent
- ^eTo be applied only if the conditions remain wet, for instance under water



3.30 Widening up of a live crack at the surface before filling it up with a sealant to reduce stresses.



3.31 Closing off live crack to prevent adverse agent ingress.



3.32 Types of surface protection.

3.8.2 Types

The following types of surface protection can be distinguished according to EN 1504, Part 2, *Surface Protection Systems* (see Fig. 3.32).

- Hydrophobic impregnation.
- Impregnation that partially or completely fills up the pores.
- Coating.

In practice, hydrophobic impregnation and coating are the most important.

When concrete is made water-repellent by hydrophobing, the walls of the concrete pores are lined with a hydrophobic agent by means of the suction of the agent into the concrete. This process is sometimes aided by previous artificial drying. Hydrophobing does not significantly influence transport of water vapour, but can considerably reduce water absorption.

In cases of impregnation and filling up of pores, the concrete surface is penetrated by an agent that fills up the pores. Impregnation can be performed by making use of the absorptive capacity of concrete, which can be promoted by drying the surface and/or evacuating the air. A distinction can be made between agents that fill up the pores by reacting with constituents of concrete and agents that do not react with concrete.

At locations where the pores of the concrete surface layer become totally filled up by impregnation, the process is referred to as sealing. By applying a coating, the concrete protection is based on the layer covering the concrete. Coatings are distinguished by thickness:

- Thin coatings: layers less than 100 μm thick.
- Thick coatings: layers between 100 and 500 μm thick.
- Plasters both with an organic and an inorganic basis that have thicknesses of between 500 μm and 5 mm.

A survey of the various methods of surface protection is given in Table 3.10. The last four surface protection methods are not considered in this chapter.

3.8.3 Reasons for protection

There are a number of reasons for protecting concrete, as explained in Section 3.5, Tables 3.6, and 3.7 and summarized in Table 3.11.

3.8.4 Durability of protection

3.8.4.1 *General*

An important aspect with respect to protective measures is their durability. Durability can be affected by external factors that can cause degradation of the protection agent. These include UV radiation and oxidation. Of special interest, however, is the internal compatibility of the substrate, i.e. of the 'old' concrete with the surface protection system applied.

3.8.4.2 *Degradation by UV radiation and oxidation*

Organic materials degrade when exposed to UV radiation and to air. Hydrophobic agents will lose their performance within 10 to 15 years. Epoxy coatings will start to deteriorate at the surface and the polymer structure will break down leaving a whitish and dusty surface showing the fillers. Although this does not usually affect the protective performance of the coating, it is often unacceptable for aesthetic reasons. A better UV-resistant top coat, such as a polyurethane coat, is therefore frequently applied (see Sections 3.8.5.3 and 3.10.1).

Table 3.10 Surface protection methods

Treatment	Effect	Substrate conditions
Hydrophobing	Water-repelling/water vapour permeable; not resistant to chemical loading	Fine pores
Impregnate/fill up pores	Decrease water absorption and increase water and water vapour resistance; not resistant to chemical loading	Fine pores
Thin coatings	Water and water vapour tight; sensitive to mechanical loading; restricted chemical and thermal resistance	Smooth surface, free from large pores and cracks ^a
Thick coatings	Water and water vapour tight; more resistant than thin coatings	Smooth surface, fine non-living cracks allowed
Inorganic plasters	Fairly watertight, water vapour permeable; no chemical resistance (excluding special types)	Free of large macro pores (air bubbles, honeycombs) Non-living Fine cracks allowed ^a
Organic plasters/high built coatings	Water and water vapour tight; resistant to chemical loading; less resistant to mechanical loading	Free of macro pores, Non-living, Fine cracks allowed
Film membranes	Water and water vapour tight; resistant to chemical loading, less resistant to mechanical loading	Smooth surface, Cracks to 3mm width allowed
Rubber lining	Water and water vapour tight, resistant to chemical loading, temperature and mechanical loading	Smooth
Lining with thermoplastic sheeting or pipes	Water and water vapour tight, resistant to chemical loading, temperature and mechanical loading	Smooth
Tiling	Resistant depending on kind of tiles, adhesive and joint filler	Smooth

^a Crack bridging ability can be increased by fibre reinforcement

Table 3.11 Reasons for the protection of concrete structures (EN 1504 Part 9)

Principle No	Principle and its definition	Examples
Principle 1 [PI]	Protection against ingress Reducing or preventing ingress of adverse agents, e.g. water, other liquids, vapour, gas, chemicals and biological agents.	Prevention of ingress of chloride in splash and spray zones of marine structures To avoid ingress of salt-laden water below the (ground) water table that can be transported to above ground level where the water evaporates and the salts remain (tanking) To avoid progress of carbonation
Principle 2 [MC]	Moisture control Adjusting and maintaining the moisture content in concrete within a specified range of values.	Prevention of frost-thaw de-icing salt attack, for instance in edge beams of bridges
Principle 5 [PR]	Physical resistance Increasing resistance to physical or mechanical attack.	Resin-bound floor to increase tear and wear Reducing dust development
Principle 6 [RC]	Resistance to chemicals Increasing resistance of the concrete surface to deterioration by chemical attack.	Coatings in factories on floors and walls in contact with aggressive chemicals To avoid degradation by sulphuric attack in sewer pipes and closed manure silos; see Section 3.2.1
Principle 8 [IR]	Increasing resistivity Increasing the electrical resistivity of concrete.	Decreasing corrosion rate of reinforcement in the case of carbonated concrete
Principle 9 [CC]	Cathodic control Creating conditions in which potentially cathodic areas of reinforcement are unable to drive an anodic reaction.	Limiting oxygen content (at the cathode) by surface coating leads to decrease of corrosion rate; see Section 3.10.1 practical case

^aEN 1504 'Products and Systems for Protection and Repair of Concrete Structures', part 9 'General principles for the use of products and systems'

3.8.4.3 Compatibility with concrete substrate

Frost–thaw de-icing salt damage

When the protective treatment renders the surface very dense, as in the case of sealings or moisture-impermeable coatings, a high moisture content can build up under the protective layer that can make the concrete vulnerable to frost attack. This appears to occur especially on surfaces with frequent frost–thaw cycles,²⁵ e.g. edge beams of bridge decks exposed to de-icing salts. To avoid these problems, Künzels has proposed the following limit values:

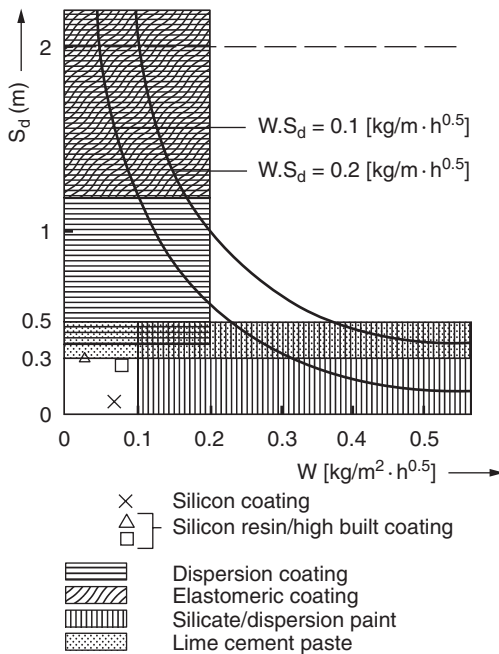
$$\begin{aligned} W &\leq 0.5 \text{ kg}/(\text{m}^2 \cdot \text{h}^{1/2}) \\ S_d &\leq 2 \text{ m} \\ S_d \cdot W &\leq 0.1 \text{ kg}/(\text{m} \cdot \text{h}^{1/2}) \end{aligned} \quad [3.12]$$

where:

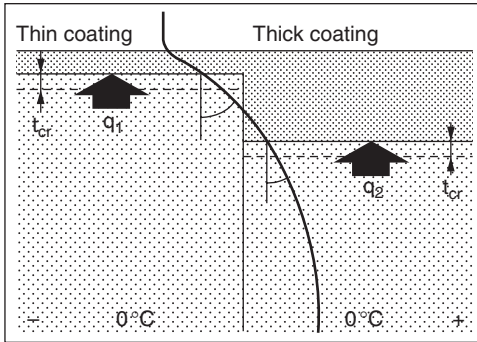
W = water absorption coefficient

S_d = water vapour diffusion resistance

To express this concept in words: the protection must not allow water absorption, but must permit water vapour to leave the concrete. Figure



3.33 Künzels relation between water absorption coefficient (W) and water vapour diffusion resistance (S_d) and positions of various protective agents.²⁶



3.34 Temperature gradient and moisture transport for a thin coating and a thick coating.²⁷

3.33 shows that a large number of protective materials can meet these requirements.²⁶

Obviously, when these criteria are considered, hydrophobing is a better protective measure than sealing or a dense coating.

Problems with sealings or impermeable coatings can be avoided if concrete is dried before it is protected but this is usually not a very practical solution. The danger of frost–thaw damage caused by freezing of condensed water underneath an impermeable layer can be decreased by applying a thick coating or plaster instead of a thin coating. The rate of condensation and the probability of freezing will then decrease, as illustrated in Fig. 3.34.²⁷

Debonding and cracking

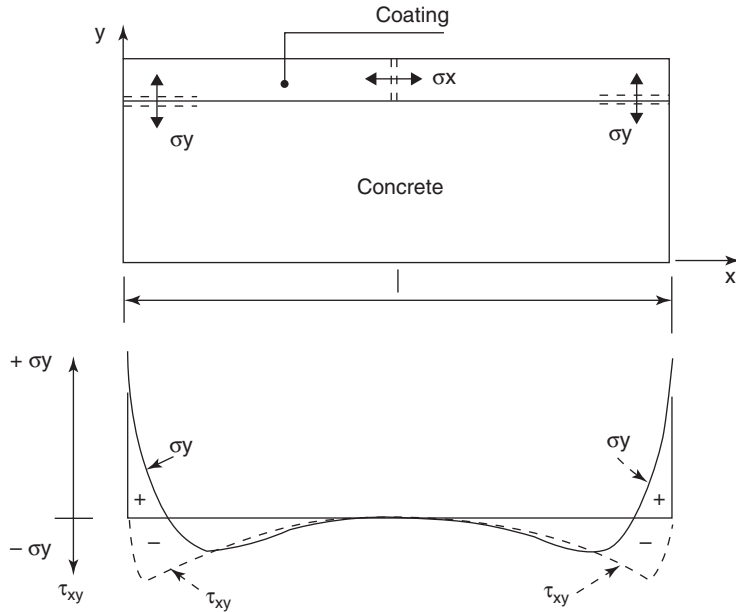
Another aspect of durability is the debonding and cracking of a coating system. In general, even when the concrete surface is well prepared, the surface layer/concrete interface will still be very vulnerable. When the substrate is not prepared properly, the concrete itself is often the weak link in the system. The problem of cracking and debonding of concrete coatings due to restrained stresses is similar to that of metal coatings discussed in Section 2.5.3. Stresses leading to debonding are often temperature-related. Figure 3.35 shows the stresses at the interface in question.

The stresses that develop concern:

- polymerization shrinkage and shrinkage due to evaporation of solvents.
- temperature movements that are dynamic in nature.

The basic formula with respect to linear expansion stresses is as follows:

$$\begin{aligned}\sigma_x &= E_{\text{coating}} \cdot (a_{\text{coating}} - a_{\text{concrete}}) \cdot \Delta T \\ \sigma_{x+} &= E_{\text{coating}} \cdot (a_{\text{coating}} - a_{\text{concrete}}) \cdot (T_{\text{coating}} - T_{\text{concrete}})\end{aligned}\quad [3.13]$$



3.35 Typical stress distribution in the contraction of concrete coating.

where:

σ_x = tensile stress in coating due to difference in thermal expansion coefficient at temperature change ΔT

σ_{x+} = tensile stress in coating due to difference in thermal expansion coefficient at difference in temperature of the coating related to the concrete substrate

$E_{coating}$ = modulus of elasticity coating

$\alpha_{coating}$ = thermal expansion coefficient coating

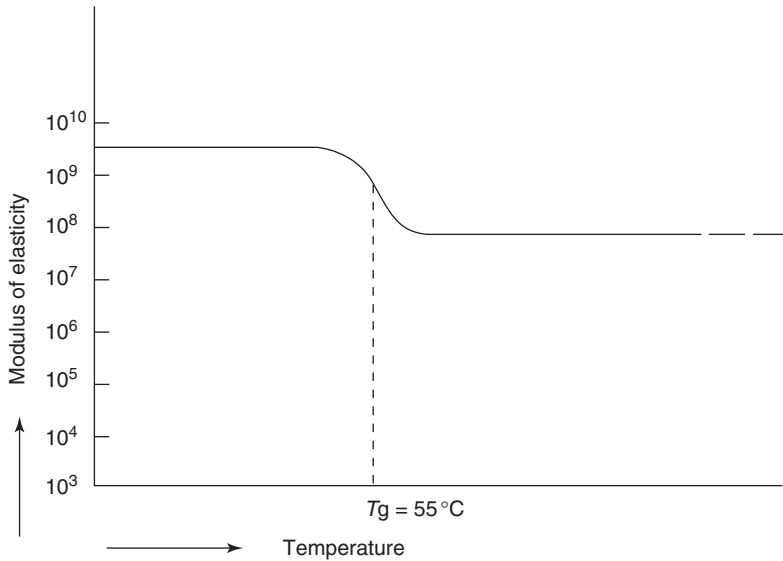
$\alpha_{concrete}$ = thermal expansion coefficient concrete

Stresses are induced because the thermal coefficient of expansion of the coating is, in general, substantially higher than that of the concrete.

$$\alpha_{coating} \gg \alpha_{concrete} \quad [3.14]$$

The difference in temperature $T_{coating} - T_{concrete}$ will be small for thin coats, but will increase as thickness increases. Restraining of movements of the coating by concrete leads to fatigue stresses. The stresses developed can be especially high in the case of direct solar radiation and a black coating.

Failure of epoxy-based repair coatings and protective layers can also be enhanced by the fact that epoxies mostly have a glass-rubber transition temperature (T_g) within the service temperature range (see Fig. 3.36). As a



3.36 Glass-rubber transition temperature (T_g) of a typical epoxy resin.

consequence, the coating can show 'plastic' deformation at temperatures above T_g . When subsequently cooled down suddenly, for instance by rain, the deformation becomes more or less frozen in and this induces stresses. This phenomenon is repeated over time so that fatigue failure can occur after a certain period. A protective polymer material in a Western European climate must, therefore, have a T_g of at least 65°C .

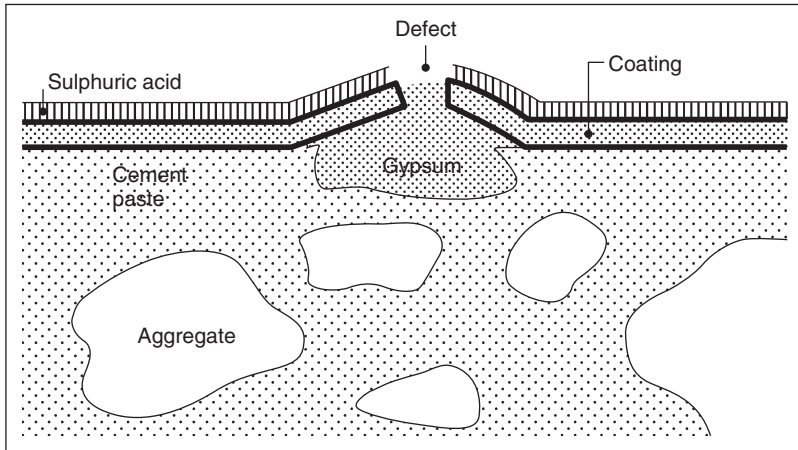
Crack bridging

In the case of cracks in concrete, only dead cracks can be successfully bridged by coatings.

Live cracks cannot be bridged unless the cracks are widened and the area to be bridged is made larger so that stresses in the coat are reduced (see Section 3.7). A coating system on concrete can never bridge a newly developing crack.

Saponification

An important condition for every protecting agent to be applied on concrete is that it must be resistant to the highly alkaline environment of concrete. A well-known binder used in the past that was not alkaline-resistant was polyvinyl acetate.



3.37 Blistering of coating on concrete due to attack of sulphuric acid through defects.

3.8.4.4 Blistering

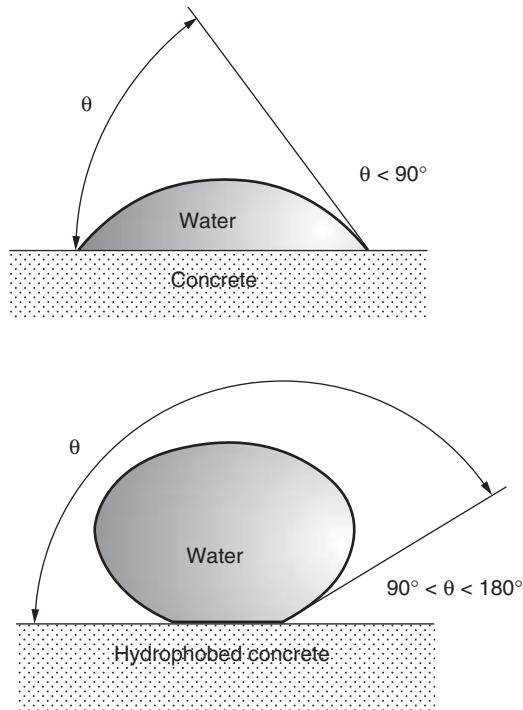
Blistering of coating systems can occur when water-soluble constituents are present within them. Water from the substrate or water present at the outer surface of the coating can migrate through the coat to these soluble constituents, due to osmotic forces, and make the coating swell. The water-soluble constituents can be unreactive compounds such as the amine hardener in epoxy coatings, or they can be dispersing agents.

Epoxy coatings will be especially vulnerable when exposed early to wet conditions and when cross-linking of epoxy and hardener is incomplete. When ambient temperatures are low and the relative humidity is high, blistering can occur easily. Another cause of blistering can be gypsum formation during biogenic attack in sewer systems or manure silos, as shown in Fig. 3.37. Sulphuric acid that penetrates the coating at holidays reacts with lime in cement to form gypsum, the volume of which is substantially larger than that of cement. This mechanism causes *in situ* applied coatings in sewer pipes to fail in areas with substantial waste water turbulence.

3.8.5 Agents

3.8.5.1 Hydrophobic agents

The hydrophobic agents most commonly applied are those that consist of organo-silicon compounds. The right choice of organo-silicon compounds will give concrete very good hydrophobic properties and water-repellence



3.38 Increase in contact angle by making concrete water repellent.

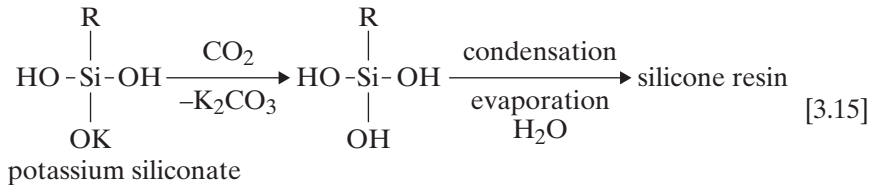
should be maintained for at least 10 years. A distinction can be made between the following compounds:

- Siliconates with a molecular weight $M = 100\text{--}200$.
- Silicon resins, $M > 2000$.
- Alkylalkoxy silanes (silanes), $M = 100\text{--}200$.
- Oligomeric alkylalkoxy siloxanes (siloxanes), $M = 400\text{--}600$.
- Polymeric alkylalkoxy siloxanes (similar to resins), $M > 1000$.

These agents can bind chemically with concrete. The non-polar groups provide the water-repellent properties which means that the contact angle between water and the concrete surface will be larger than 90 degrees (Fig. 3.38.) resulting in water-repellent behaviour instead of the original capillary suction. Hydrophobic impregnation is used, not only as an isolated measure, but also as part of complete protective systems, for instance in combination with coatings. The advantage of such systems is that when the coating shows defects, water and aggressive agents cannot easily enter the concrete through them, thus preventing the coating from being attacked from the rear.

Siliconates

Siliconates are water-soluble or alcohol-soluble products (often with about 40% of solids). When applied to concrete they react with carbon dioxide according to the reaction:



The R-group in the reaction formula is ethyl or propyl. To be effective, siliconates need carbon dioxide from the air. The K_2CO_3 released during the reaction with carbon dioxide may blemish the concrete but blemishing will be temporary because it will be washed away by rain. Siliconates are seldom used for concrete surface protection.

Silicon resins

The second generation of water-repellent impregnants for concrete or other stone-like building materials are the silicon resins. These resins are dissolved in an aliphatic-soluble agent (typically containing about 5% of solids). This agent does not have the disadvantage of K_2CO_3 formation. The resin reacts and is able to bond (cross-link) with the pore walls in place. A disadvantage is that this resin can only be applied effectively on dry materials with relatively coarse pores and these silicon resins can easily lead to sticky surfaces. They are not very suitable for application on concrete, mainly because the pores in this material are too fine.

Silanes

Alkylalkoxy silanes are dissolved in aliphatic or aromatic solvents. The silane content is relatively high, i.e. 40%. Moisture and a catalyst are required so as to enable the reaction to polysiloxanes. Concrete catalyzes the polymerization.

The advantages of silanes in comparison with hydrophobic agents are as follows:

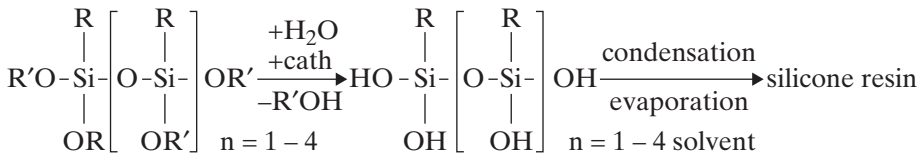
- Better and deeper impregnation as a consequence of the low-viscosity solvent when compared to siliconates and the small molecules in relation to silicon resins.
- The high percentage of active material, which is about 40%, as compared to 5 to 10% for silicon resins.

A disadvantage of silanes is that the material itself is volatile and can evaporate together with the solvent, which makes the impregnation results very weather dependent.

As a result of environmental pressure to reduce the evaporation of volatile organic compounds this type of treatment is no longer considered acceptable in many countries.

Oligomeric alkylalkoxy siloxanes

To overcome the problem of evaporation of silanes, slightly polymerized silanes have been developed. These are known as oligomeric alkylalkoxy siloxanes, or siloxanes for short. The advantages of silanes have been maintained, while the agents are less volatile. Siloxanes are available as solutions in aliphatic agents or in alcohols. Their active material content ranges between 5 and 10%. The reaction is:



[3.16]

Polymeric alkylalkoxy siloxanes

When silanes or siloxanes are polymerized further, long chain molecules are developed. These polymer chains look very much like silicon resins, but are very sticky if no catalyst is applied. Polymeric alkylalkoxy siloxane are rarely used for concrete.

Silicon micro-emulsions

To overcome the environmental drawbacks of volatile organic solvents, micro-emulsions of silicon resins have been developed. Micro-balls of the silicon resin are dispersed in water and the emulsion particles are small enough to penetrate the pores.

3.8.5.2 Impregnation/filling up of pores

There are various impregnating agents available to fill up pores. Since concrete degradation often happens through the pores, this filling-up can make the concrete less vulnerable. Only filling up with organic compounds is discussed here.

The monomer, methacrylic-acid-methyl ester, easily penetrates dry concrete. Once it has penetrated, it can be polymerized to form polymethyl-methacrylate (PMMA). In Germany, this method is used to restore or preserve statues. The procedure for this preservation is as follows:

- Drying.
- Evacuating.
- Impregnating under pressure.
- Heating up for hardening.

This method may show good results, but is restricted in its application because the size of the objects is governed by maximum limits. For concrete structures that cannot be evacuated and impregnated under pressure, results are generally rather poor because the impregnation does not fully fill up the pores and the required drying can cause cracks in the concrete.

Another organic compound that can be applied to impregnation is a one-component polyurethane. This involves a prepolymer that polymerizes further by reacting with moisture from the air or from the concrete. Concrete must be relatively dry during application to prevent excessive foaming.

3.8.5.3 *Coatings*

In general, coatings comprise the following constituents:

- Binders.
- Fillers (sometimes).
- Pigments.
- Additives.
- Dispersing agents.
- Solvents/diluents.

Binding is achieved through polyaddition or polymerization reactions between the binding components (resins) or by drying as a result of evaporation of the solvents and dispersing agents. The most important resins for concrete structure protection are:

- Epoxy resins (EP): Epoxy resins usually provide good adhesion to concrete. Epoxy resins show minor polymerization shrinkage and are resistant to light chemical attack. However, they do not show a good resistance to UV radiation (see Section 3.8.4.2).

To reduce the emission of volatile organic solvents, new epoxy coatings have been developed, for instance the so-called high built epoxy coats. Some problems with these new types of coats are discussed in Section 2.5.3.1.

- Two component polyurethanes (PUR): These adhere well to dry concrete surfaces, are almost shrinkage-free and are resistant to light chemical attack. However, they are not very resistant in high alkaline environments and must therefore not be applied to surfaces that will be continuously wet, unless an alkaline resistant primer is used. PUR coatings usually show a good resistance to UV radiation and are often used as topcoats on epoxy resin base coats. PUR resins occasionally show discoloration (yellowing).
- Tar epoxy and tar PUR resins: These have some advantages, such as higher flexibility in comparison to EP and better alkaline resistance when compared with PUR. A disadvantage is that they do not have such a good resistance to UV radiation, oxidation and temperature extremes. Tar is banned in some countries (e.g. the Netherlands), because of the presence of aromatic compounds.
- Polymethylmethacrylate (PMMA): PMMA resins show very good adherence to concrete substrates. However, there is considerable shrinkage during polymerization of the polymer. Other features are their high mechanical strength and a good resistance to light chemical attack.

Drying types of binders are:

- *Acrylics*: in solution as well as in dispersion. Acrylics generally have a good adherence to concrete and a good resistance to alkalis, oxidation and weathering.

Further drying type agents are the following: chlorinated rubber, cyclo-rubber, vinylchloride copolymers, vinylfluoride copolymers, silicon resins and polymeric alkylalkoxy siloxanes. The latter are applied not only to coatings, but also to impregnation, as discussed previously. Combinations of drying binders are also available on the market, such as acrylics combined with polyvinyl(tetra)fluoride.

3.9 Cathodic protection systems

3.9.1 General

Cathodic protection (French: protection cathodique, German: kathodische Korrosionsschutz, Dutch: kathodische bescherming) is a well-established anti-corrosion method for protecting steel exposed to aggressive liquids (e.g. sea water) and soils. Over the last decades, cathodic protection of steel-reinforced concrete structures has also become popular.²⁸ Cathodic protection seems to provide a reliable way to stop the corrosion process. In the United States, the Federal Highway Administration (FHWA) has come to the following conclusion: "The only rehabilitation technique that has proven

to stop corrosion in salt contaminated bridge-decks regardless of the chloride content of concrete is cathodic protection.²⁹

The European Standard EN 12696 *Cathodic Protection of Steel in Concrete, Part 1, Atmospherically Exposed Concrete* specifies performance requirements both for existing and new structures. In practice, cathodic protection is usually applied to existing structures suffering from reinforcement corrosion. However, it is also used occasionally in new structures, e.g. by ensuring electrical continuity on the steel reinforcement so that later on cathodic protection systems can easily be applied if required (see Section 2.5.5).

3.9.2 Components

Cathodic protection systems with impressed current comprise the following components:

- A DC source.
- An anode system.
- A conductive electrolyte: concrete. The moisture content has a dominant influence on conductivity.
- A cathodic system: the steel reinforcement that must be electrically continuous.
- Electrical connecting cables that must be mechanically and chemically durable.
- Control and monitoring devices (reference electrodes, data compilers, control units, etc.).

3.9.3 Anodes

The anodic system serves to supply the protecting current homogeneously. It has to meet the following requirements:

- It must be able to resist the mechanical, physical and chemical influences of the environment.
- The lifetime must be longer than that of the covering layer, with a minimum of at least 20 years.
- The anode surface must be large enough to ensure that the current density is sufficiently low to prevent failure of the system due to degradation of the anode or its interface layer in the surrounding concrete.
- It must be economically feasible.

3.9.3.1 Types of anode system

Anode systems that can be distinguished are:

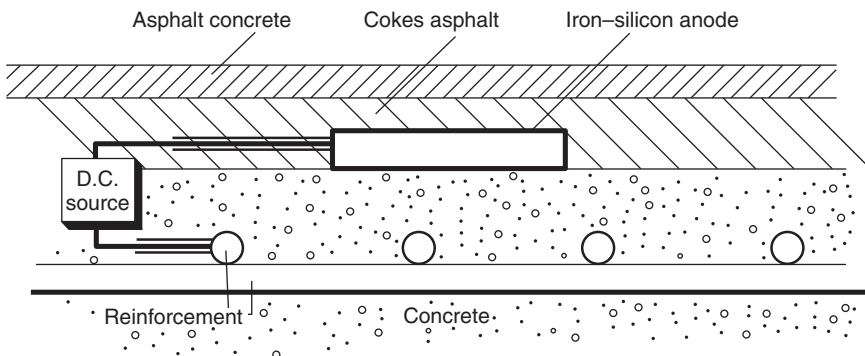
- Conductive overlays: a layer acting as an anode covering the entire concrete surface.
- Distributed anodes: discrete anodes that are spread evenly over the concrete surface. Only part of the concrete surface is covered with active anode material.
- Sacrificial metal layer: a covering metal layer that acts as a sacrificial anode.

Conductive overlays

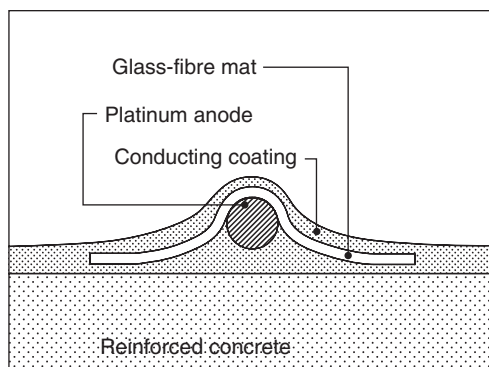
Anodes that cover the entire surface of the concrete usually consist of a combination of primary and secondary anodes. The primary anode consists of an inert metal. The secondary anode is an overlay consisting of a mortar, asphalt, or paint made conductive by adding carbon particles or consisting of a conductive polymer.

Examples of systems of this type are:

- Silicon iron anodes in combination with an overlay of conductive asphalt. This is the oldest cathodic protection system of concrete. The asphalt is made conductive by applying coke-breeze as a filler. The primary anodes are 300mm discs, 10mm thickness or $150 \times 230\text{mm}^2$ squares on which a conductive asphalt overlay is applied as a secondary anode. The system is shown in Fig. 3.39¹ and has been used on bridge decks in the United States since 1973. The system can only be applied on top of horizontal surfaces.
- Meshed AD-wire of titanium often applied with a mixed rare earth metal oxide film at the surface. The meshed AD-wires are covered with a conductive asphalt or conductive cement mortar.



3.39 Cathodic protection in a bridge deck. The iron-silicon anodes have been embedded in a layer of conductive coke breeze asphalt.¹



3.40 Principle of covering anode system with conductive paint.

- **Conductive paint (coating) systems.** A series of conductors that act as primary anodes on the concrete surface to dissipate the current. The conductors must be of a material that resists anodic reactions, e.g. platinum-coated or platinum-clad titanium or niobium with a copper core, or mixed metal oxide-coated titanium. They are fixed to the surface and sometimes protected by glass fibre mats over which a conductive paint is applied as a secondary anode, as shown in Fig. 3.40. The paint is a dense coating made conductive by using graphite as a filler. Conductive polymers are also used. For aesthetic reasons, the black paint is often covered with paint in the desired colour. This type of anode is appropriate for both ceilings and vertical and horizontal surfaces, but not for surfaces that are exposed to wear and tear. The durability of this paint system is estimated to be 10 years. After that period, the concrete must be cleaned and coated with a new paint system. Advantages of paint systems are the relative ease of application, the ease of repair of possible defects, and the favourable ratio of the anode surface to the concrete surface (one to one). Because of the latter factor, acid deterioration of the concrete below the paint coat will proceed relatively slowly due to the low current density at the anode. However, minor degradation at the interface can severely damage the effectiveness of the system. In addition, a combination of a dense paint system with a water-rich concrete layer underneath will show sensitivity to de-icing salts (see Section 3.8.4.3).

A practical case of a conductive paint system is discussed in Section 3.10.2.

Sacrificial anodes

An example of a sacrificial anode is the flame-sprayed zinc anode. A zinc coat of about 200µm is sprayed onto the concrete and connected electrically to the reinforcement. This is known as a sacrificial anode because it does not need an external potential source. Another type of sacrificial anode occurs where a zinc foil is adhered to the concrete surface.

3.9.3.2 Durability

For the sake of durability, all parts of a cathodic protection system must perform well throughout the design life. Some defects have appeared in a number of pioneering projects. These include: corroding of connecting cables on primary anodes, damage of the electrical control unit, erosion of conducting asphalt layers, debonding of a sprayed concrete cover and degradation of graphite-loaded polymer protection around copper wires. Not all of them are typical of cathodic protection systems. However, attention needs to be paid to a number of phenomena that occur in concrete fitted with an activated cathodic protection system.

Anode consumption

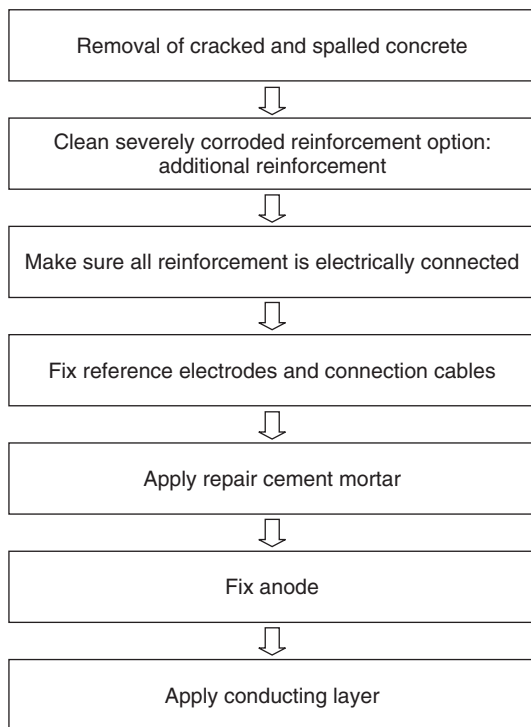
One of the possible degradation mechanisms takes place at the anode where various reactions occur as previously described in this chapter. If graphite is used for anode cables, conductive mortars etc., the following reactions can occur:



This means that the anode will slowly lose its thickness. The anodic current density should, therefore, be restricted and the maximum anodic value recommended by NACE is 0.1 A/m² current density. The titanium mesh anode appears to behave considerably better in this respect.

Acid production

Another durability aspect of cathodic protection is the production of acid at the anode. As a consequence of the reactions previously described in this chapter, alkalinity at the anode/concrete interface will decrease, thus increasing acidity. The anode materials may show very good resistance to acid. However, the surrounding cement mortar will not be resistant to acid formation. It is likely that a thin layer of concrete will be degraded.



3.41 Repair procedure with CP in the case of chloride-initiated corrosion.

Nevertheless, below the 0.1 A/m^2 current density, the effect appears to be negligible.

3.9.4 Hydrogen embrittlement

Prestressed or post-tensioned cables or bars are vulnerable to embrittlement when hydrogen is generated at the steel surface (see Section 2.3.2.11). It has been found that as long as the potential is more positive than -1100 mV in comparison to copper sulphate electrode (CSE), there is hardly a risk of hydrogen embrittlement.

3.9.5 Repair procedure with cathodic protection

Figure 3.41 shows a repair work procedure during which CP is applied.

3.9.6 Protection criteria

Various cathodic protection criteria for steel-reinforced concrete are advocated. The following criteria are given in EN 12696-1:

- An instant off potential (measured between 0.1 s and 1 s after switching on the d.c. circuit) that is more negative than -720 mV with respect to $\text{AgIAgCl}/0.5\text{ M KCl}$.
- A potential decay over a maximum 24 hour period of at least 100 mV from instant off.
- A potential decay over an extended period (typically 24 hours or longer) of at least 150 mV from the instant off subject to a continuing decay and the use of reference electrodes for measurements extended beyond a 24 hour period.

Overprotection must be avoided, not because of the increased cost due to elevated currents that do not usually exceed a 100 W per thousand m^2 of concrete surface, but because overprotection results in high current densities with a subsequent risk to the durability of the system.

3.9.7 Pulsed electro-osmosis

Electro-osmosis is a technology for drying concrete between electrodes embedded in the concrete and located at the concrete surface. The solid CSH-gel concrete surface is negatively charged and attracts positive ions from the pore water. When a voltage is applied to the positive anode and negative cathode electrodes, an electrical field is created between them. Consequently, the positive ions are attracted to the negative cathode and drag the water with them. Locating a positive electrode close to the reinforcement and a negative electrode at a distance from the reinforcement can reduce humidity at the reinforcement. Polarization can then be avoided by applying a pulsed voltage over the electrodes with a short rest period of no applied voltage.

It has been claimed that the humidity can be decreased to a level lower than 60 to 70%, which is a level below which corrosion is stopped. This corresponds to the Principle 8 'increasing resistivity' of Table 3.7. The patented method is offered in combination with a pulsed cathodic protection system of the reinforcement, in which case both cathodic protection and the increased resistivity of the drying concrete will contribute to decreased corrosion. The method is also recommended to counteract alkali silica expansion that corresponds to the Principle 2 'moisture control', Method 2.4 'electrochemical treatment' of Table 3.6.

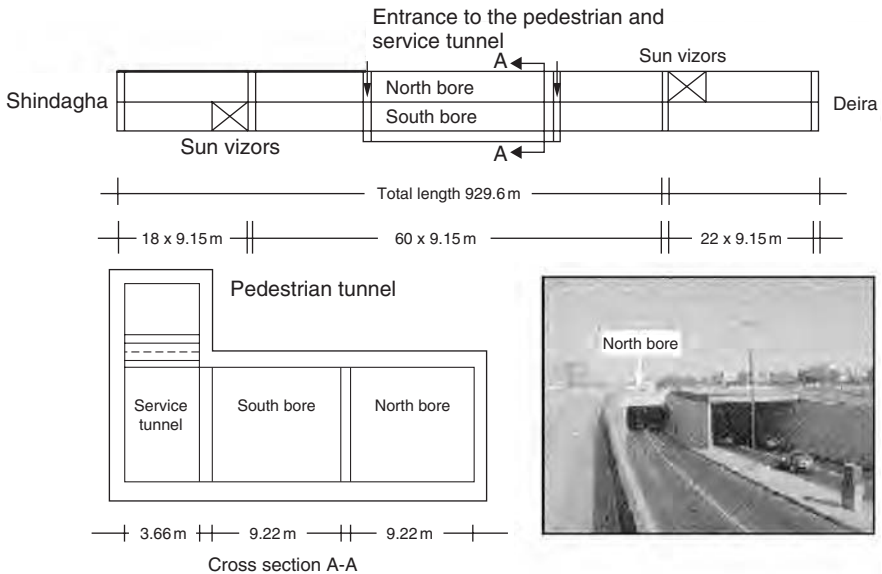
3.10 Practical cases of repair

3.10.1 Repair of the Deira–Shindagha tunnel in Dubai

3.10.1.1 Case description

A sea arm cuts off Dubai from the Arabian Gulf and, in 1975, a 561 m long tunnel crossing the Dubai Creek was completed. A cross-section of the tunnel construction is shown in Fig. 3.42. The concrete was cast in place and consisted of a sulphate-resistant Portland cement, porous limestone, coarse aggregate, beach sand with occasional chloride fractions, and tap water. The free water/cement ratio varied and could be as high as 0.6. The concrete structure was built in sections with a rubber water stop in the dilation joints and construction joints. The space in the dilations joints was filled up with bituminized cork and finished with a Neferma strip. The exterior of the tunnel was covered with Bitu-Thene sheets that were to act as a water and salt ingress barrier. A latex-cement (PC) coating with an aesthetic function was applied on the concrete inner-wall.

During construction, the cofferdam on top of the already constructed tunnel section slipped away under water pressure and damaged the water-impermeable Bitu-Thene layers. Although repairs were carried out, this may have caused permanent damage.



3.42 Schematic view of the Deira–Shindagha tunnel in Dubai.

3.10.1.2 Causes of damage

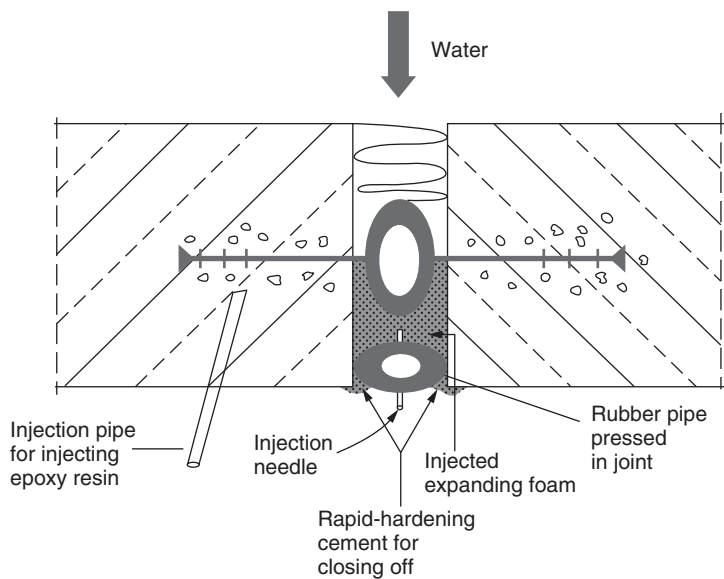
Soon after completion, water leakage of Creek water through the joints was observed and reinforcement corrosion was reported within a few years. Obviously, this was due to the local presence of chloride-contaminated beach sand and to the highly permeable character of the concrete applied with respect to chloride ingress (see Section 3.4).

In 1983, Nedeco (a Dutch joint venture of consulting engineers) was asked by the Municipality of Dubai to assess the damage and to advise on the possibilities of repair. Later on, Nedeco was also appointed resident engineer for the repair works. The author was the senior expert of the Nedeco team for material and corrosion aspects. On inspection, serious cracking and spalling of concrete was observed. Concrete adjacent to joints was often pushed away from the reinforcement for several centimetres and 80mm diameter rebars had completely corroded in some locations. Although the leakage rate was not substantially higher than leakage rates in similar tunnels in the Netherlands, the effect in this particular case proved to be highly detrimental. Obviously, this was due to the presence of the salty Creek water, the low resistance to chloride ingress of the concrete, and the high ambient temperatures. Previous repair work with epoxy repair mortars had failed. Rebars beneath the repaired sections had continued to corrode and corrosion had probably been aggravated where it was next to the repair work. A structural design check showed that there was no immediate structural safety problem due to substantial overdesign. Most of the concrete appeared to be in compression.

3.10.1.3 Repair principles

It was recommended to repair the tunnel by applying various repair principles. It was considered that the recommended combination of principles (of Tables 3.6 and 3.7) could substantially extend the service life of the tunnel.

- 1 Stop leakage (Principle 1).
Concrete around the rubber waterstop in the joints was injected with an epoxy injection agent, as schematically shown in Fig. 3.43. This appeared to stop effectively most of the leakage.
- 2 Removal of the affected concrete and areas severely contaminated with chloride (Principle 7, Method 7.2). This occurred up to a 50 mm distance behind the reinforcement but on locations that were in a critical structural area, this had to be restricted to the reinforcement level.
- 3 Cleaning or replacing reinforcing steel bars (Principle 4, Method 4.1).
- 4 Coating the cleaned reinforcement and replacing the rebars with an epoxy barrier coating (Principle 11, Method 11.2). Figure 3.44



3.43 Rubber waterstop injected in porous concrete using a low viscosity epoxy injection agent.



3.44 Cleaning and coating of concrete in ramp walls during repair of the Deira-Al Shindagha tunnel in Dubai.

shows the cleaning and coating operation of concrete walls in the ramps.

- 5 Replacing the removed concrete (Principle 7, Method 7.2). Initially, a polymer-modified shotcrete with blast furnace slag cement CEM III/B as the cementitious binder was applied (called SPCC, see Section 3.6.3). Later on, the polymer was replaced by silica fume for operational reasons.
- 6 The reinforcement cover was extended by 20mm (Principle 7, Method 7.1) with the same shotcrete.
- 7 An airtight coating was applied limiting oxygen flow to the reinforcement (Principle 9, Method 9.1). An oxygen-diffusion resistance of the coating system of 4000m was required. In general, cutting off oxygen ingress in concrete is difficult to achieve in repair works. In the actual case of the submerged tunnel, it was, however, considered feasible if taken in conjunction with other measures.

The coating system consisted of two epoxy coatings and a polyurethane topcoat. The topcoat showed better resistance to UV radiation, which was a specific requirement for the tunnel ramps, and also had a crack-bridging ability. Prior to applying the coating, the surface was smoothed with an epoxy-based equalization slurry.

Repair using cathodic protection had also been considered. There were, however, two reasons for rejecting this method. In the first place, experience with cathodic protection systems in concrete structures was limited in the 1980s. In addition, electro-continuity of the reinforcement was difficult to achieve. Rebars were placed in an irregular fashion and sometimes even absent.

3.10.1.4 Execution of the works

The repairs started with a trial repair, which led to modifications in the specifications. An extensive survey of the concrete was made, including crack mapping and chloride profiles. Repairs started in 1986 and were completed in 1988. During the whole operation, repairs were guided by calculations on the structural safety of the sections where concrete was removed and where rebars were replaced. Quality control was very strict and regarded as essential to repair work success.

Part of the contract consisted in drafting a maintenance manual for the tunnel and it was regarded essential to maintain the tunnel according to strict rules. Obviously, not all affected areas could be repaired due to structural reasons and renewed cracking in these areas could not be excluded. A thorough inspection regime would therefore take immediate action if defects were observed so as to avoid progressive deterioration. In 2002

when this book was drafted, the tunnel appeared to be in excellent condition. No major repairs have been necessary since the completion of the tunnel wall repairs in 1988.

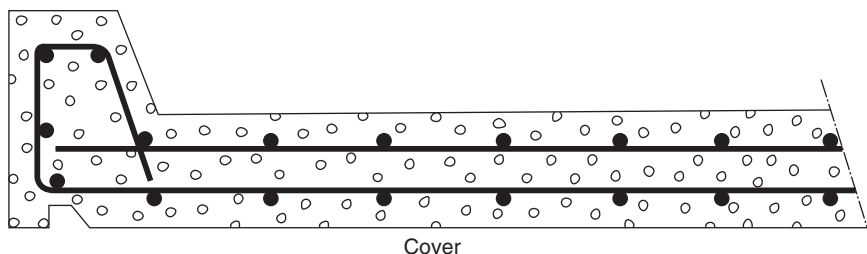
3.10.2 Cathodic protection repairs in multi-storey buildings on the Dutch coast

3.10.2.1 Case description

Balcony slabs of two multi-storey buildings in Zantvoort, a holiday resort in the Netherlands along the North Sea coast close to Amsterdam, showed severe cracking due to chloride-initiated corrosion. The edges of the balcony slabs were affected in particular. This was not only due to the protruding position of the edge rims, but also to the presence of a water-hole at the ceiling of the balcony slab. Reinforcement adjacent to this water-hole only had a cover of about 15 mm (see sketch in Fig. 3.45). The buildings were erected in the 1970s. In one of the buildings prefabricated slabs were applied. The slabs of the other buildings were cast on site. The specified cover was 25 mm and the slabs were produced from a Portland cement concrete. In the past, local repairs had been carried out several times, but appeared to have failed. It was observed that adjacent to the epoxy repair mortars, the initial concrete showed cracks and the reinforcement was often seriously affected (see Fig. 3.46). This was probably due to the fact that the repaired area had access to oxygen and functioned as a cathodic area (incipient anode effect, see Section 3.5.3). Rebars that had been coated in previous repair work showed renewed corrosion.

3.10.2.2 Measurements

Chloride profiles were determined on several occasions. They showed that the chloride content at the reinforcement had exceeded the critical corro-



3.45 Water-hole in balcony slab ceiling causing locally reduced reinforcement cover.



3.46 Local patch repair with adjacent new reinforcement corrosion.

sion concentration, as confirmed by the observed pit corrosion of the reinforcement. By means of half-cell measurements according to ASTM C867, the areas where the potential had dropped to form a corrosive area were mapped. This applied only to areas with no previous patch repair work.

3.10.2.3 Conventional repair or cathodic protection?

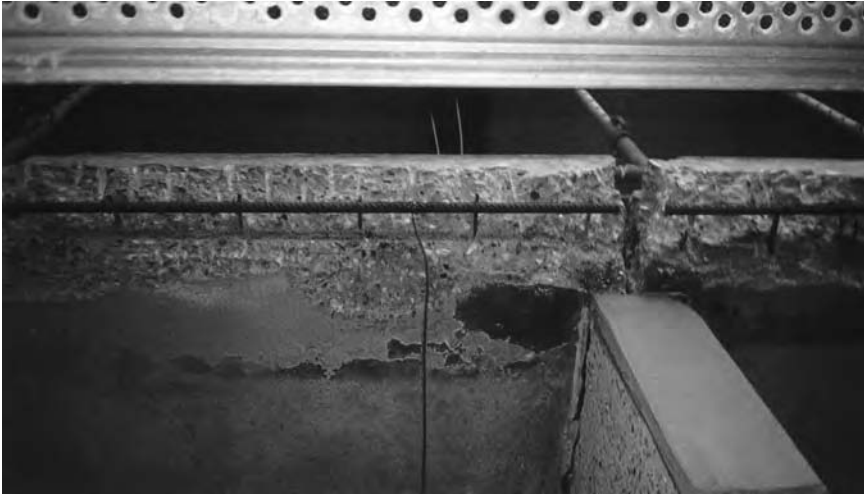
The tendering contractors were requested to table two proposals. One for a 'conventional repair' consisting of removing all concrete at the reinforcement that was contaminated beyond the critical level, cleaning of the reinforcement etc, and an alternative proposal using cathodic protection. The owner chose the cathodic protection proposal because:

- 1 The contractor to whom the contract was awarded was able to prove that the life time cost of cathodic protection repairs was lower than that of the conventional repair system, even though the initial repair cost was higher. In particular, the maintenance cost of the conventional repair system, including expected recurring small repairs, was estimated to be higher.
- 2 Cathodic protection causes less trouble to building users during repair work execution because noisy and dusty concrete removal could be avoided.
- 3 The owner had had a bad experience with the previous local patch repairs in which the principles applied were similar to the conventional repair work on offer.

3.10.2.4 Method applied

All cracked concrete was removed and the reinforcement cleaned up or replaced, as shown in Fig. 3.47. Most of the concrete on the edge rim was removed and the lower reinforcement, which was severely affected, was replaced. New concrete was cast. Connecting the reinforcement to safeguard electro-continuity turned out to be a substantial job. The rebars of the upper reinforcement of the edge rim had, for instance, been cut frequently due to coring of concrete to fix the balcony rail.

Cables for connecting the DC current were attached to the reinforcement and silver/silver chloride reference cells were fixed near the reinforcement. Locally removed concrete was replaced with a cement grout by hand trowelling. The ceiling of the balcony slab was grit-blasted. Subsequently, current dissipator wires were pinned to the ceiling surface of the slabs and beams. The wires were titanium-based with a mixed rare metal oxides coat. A conducting acrylic coating filled with carbon particles and small copper fibres was applied. The cables were connected to a rectifier supplying 1–2 V DC. The applied potential and current were adjusted regularly during the first month. A minimum 100 mV instant off potential shift over a 24 hour period was applied as the cathodic protection criterion. Finally, a topcoat was applied.



3.47 Removed concrete and replaced reinforcement at the edge of a balcony slab.

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4.1 Introduction

Wood, together with stone, is undoubtedly the oldest construction material in the world. Part of the wood market in civil engineering construction was lost when steel and reinforced concrete made their breakthrough, but it has made a comeback due to ecological preferences. Wood is a renewable material, while concrete and steel are not. Governments also promote the use of wood as a construction material in civil engineering structures.

The durability of wood depends largely on the environment to which the wood is exposed and its type. Service life has been known to exceed 500 years. The performance of wood structures can decrease with time due to natural weathering, biological attack, and chemical attack.

Biological decay can involve attack by any of the following: fungi, insects, marine borers or bacteria. If the selected wood is not durable in a specific environment, then almost any kind of wood can be protected so that it can be used in most circumstances. However, in a growing number of countries, the protective agents applied have been restricted due to environmental rules. In the Netherlands, for instance, most water-borne preservatives containing inorganic salts are not generally accepted due to the possibility of leaching out of the toxic inorganic salts during service life and afterwards when the wood is recycled, combusted or deposited.

Some wood types are durable in many natural environments, notably tropical hardwoods can be highly durable in western climates. Tropical hardwoods show great variety in colour, density and durability and are popular mainly for durability and aesthetic reasons. In order to protect the tropical forest it must be proven that the wood has been harvested in a sustainable way. Various organizations certify sustainably produced wood, e.g. the Forest Stewardship Council (FSC) supported by WWF.

These ecological requirements restrict the application of wood and this contradicts the policy of governments to extend the usage of wood as a construction material. However, these governments have also started

developing new processes to protect wood without the environmental drawbacks of the old methods, especially for wood from the temperate climate zone.

In this chapter, the following topics will be discussed:

- Deterioration mechanisms.
- Protection of wood.
- Hazard classes.
- Durability classes.
- Maintenance.
- Repair of affected structures.

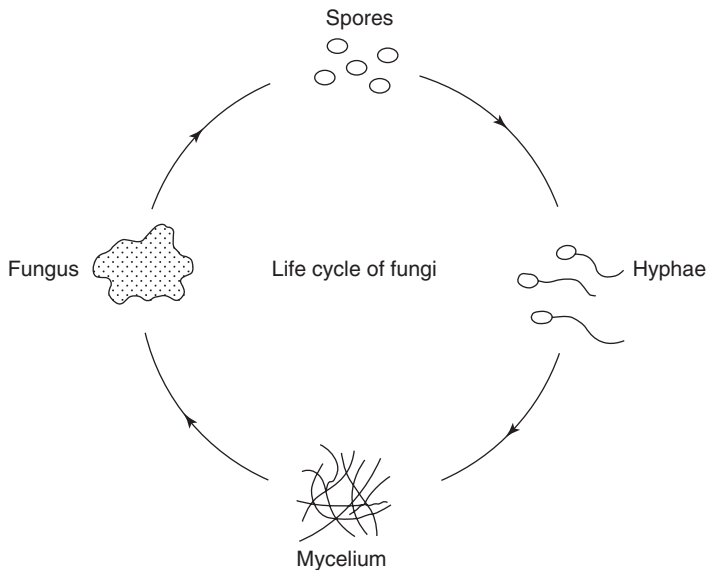
The terminology used is that of the European standards for wood.

4.2 Deterioration

4.2.1 Decay by fungi

4.2.1.1 Mechanisms

Fungi (French: mois, German: Pilze, Dutch: schimmels) cause decay of wood.¹ They pose the most serious threat to wooden structures that are not submerged in water. The life cycle of fungi is shown in Fig. 4.1.² The hyphae, fine tubes, grow into a dense network called a mycelium that affects the wood. The hyphae produce enzymes that disintegrate the cellulose or lignin,



4.1 Life cycle of fungi.²

two of the most important components of wood. The fungi play an important role in the forest. They return organic and inorganic matter to the soil and make nutrients available for new plants. They are, therefore, essential to the forests' survival.

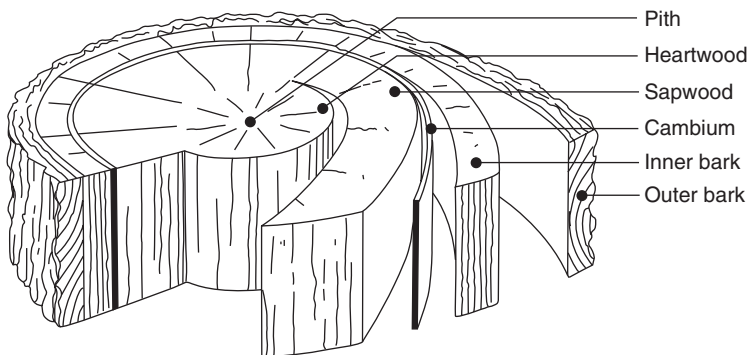
4.2.1.2 *Conditions for decay*

Decay fungi have four basic requirements for growth that must be fulfilled simultaneously: wood as a food source, oxygen (because fungi are aerobic organisms), water and appropriate temperatures. These basic requirements are all satisfied within forests, except during the winter period. Fortunately, these conditions are not usually met in building constructions. Sometimes removing that part of the wood that is most vulnerable to decay, the sapwood (French: aubier, German: Splintholz, Dutch: spinthout), see Fig. 4.2, is combined with controlling the basic requirements for durable performance.

In general, fungi do not affect wooden structures if the water content is lower than 22% m/m. Wood standards contain strict requirements for the water content of construction wood. The most favourable temperature range for wood decay is between 19 and 31 °C; a temperature of 25 °C is often optimal. Fungi survive freezing temperatures but die at temperatures exceeding 60 °C.

4.2.1.3 *Destructive fungi*

The following wood-destroying fungi can be distinguished:



4.2 Wood cross-sections showing wood parts.³



4.3 Brown rot in window frame edge.⁴

Brown rot and white rot fungi

The brown rot fungi decompose the cellulose and hemi-cellulose, while the lignin is not destroyed (see example in Fig. 4.3). The wood will stain dark brown and the degraded wood will have a cubic structure. Brown rot is therefore sometimes referred to as cubic rot. The white rot fungi particularly attack the lignin and give the wood a white colour while the wood is pulverized (see example in Fig. 4.4). White rot is predominantly associated with hardwoods.

Well-known fungi in Western Europe are the house sponge (French: *éponge de maison*, German: *Echte Hausschwamm*, Dutch: *huiszwam*), the cellar sponge (French: *éponge de cave*, German: *Kellerschwamm*, Dutch: *kelderzwam*) and the white pore sponge (French: *éponge blanche de pore*, German: *Weisse Porenschwamm*, Dutch: *witte poriënzam*).

Soft rot

Soft rot fungi need less oxygen to survive than do brown rot and white rot fungi. These fungi are particularly significant in wood exposed to soil or water. The effect of soft rot is the same as that of brown rot.



4.4 White rot predominantly affecting lignin.⁵

Wood-disfiguring fungi

The most important are the fungi causing mould or blue stain. These fungi are not a structural threat but do affect the aesthetic appearance. They can grow through decorative coatings and degrade them.

Moulds are not specific to wood and can grow on any material with a high moisture content.

4.2.2 Degradation by insects

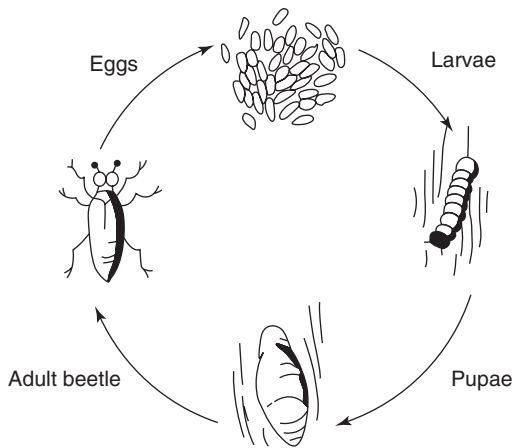
This can involve attack by beetles, termites and ants.

Beetles

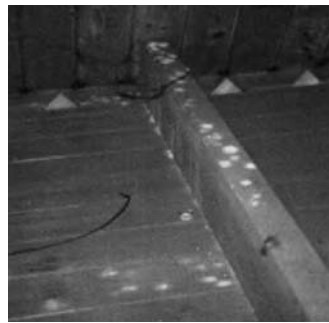
The wood-affecting beetles are insects capable of flying that lay their eggs in wood pores or cracks and have larvae that attack the wood. The life cycle of wood-boring beetles is shown in Fig. 4.5. They are present throughout Europe. Only those beetles that affect seasoned (dried) wood will be discussed. The most important ones are: the Death Watch beetle (*Xestobium*

rufovillosum), (French: grosse vrillette, German: Bunter/Gescheckter Nagekäfer, Dutch: bonte knaagkever of grote houtworm) and the Common Furniture beetle (*Anobium punctatum*), (French: petite vrillette, German: Gewöhnlicher oder Gemeiner Nagekäfer, Dutch: meubelkever). The House Longhorn beetle (*Hylotrupes bajalus*), (French: Capricorne des maisons, German: Hausbock, Dutch: huisboktor), and the Powder Post beetle (*Lyctus brunneus*), (French: lyctus, German: Brauner Splintholzkäfer, Dutch spinthout kever) are less important.

The Death Watch beetle and the Common Furniture beetle attack the sapwood of certain wood species. The damage can extend to the heartwood. Occasionally, damage is of structural significance. Figure 4.6 shows the Common Furniture beetle and affected wood. The House Longhorn beetle



4.5 Life cycle of wood-boring beetles.⁴



4.6 Common furniture beetle and affected wood.

is found throughout Europe, but less so in the north and north-west. It attacks mainly softwood species. The Powder Post beetle primarily attacks the sapwood of hardwoods, such as oak, ash and walnut. Powder Post beetles are usually discovered when small piles of fine bore dust are observed on or below the surface. The name 'powder post' refers to the phenomenon that, in heavily damaged wood, the sapwood is reduced to a mass of powdered wood held in place by a thin outer wood surface.

Termites

Termites (*Isoptera* or white ants) do by far the greatest amount of insect-related damage to wood. They have been confined to tropical and near-tropical areas but probably due to climatic changes, they are spreading northwards. In Europe, termites have been observed in Paris, Hamburg and the UK.

There are thousands of species of termites, but in Europe only four species are found. Most dangerous for buildings are the subterranean species, principally *Reticulitermes lucifugus* and *Reticulitermes santonensis*. Termites live in colonies; because of their large concentrations, damage proceeds quickly as soon as buildings become infested. Wood damaged by subterranean termites may show few, if any, signs of deterioration. By pounding, hollow infested areas can be detected and when broken up, the live termites are discovered.

4.2.3 Attack by marine borers

With the exception of a few resistant tropical woods, any untreated wood put to use or stored in salt or brackish water, throughout most of the world, is subject to damage by marine borers. The speed with which damage occurs depends upon climate, pollution, borer species, and other factors. Damage occurs most rapidly in warmer climates.

The most important marine borers with respect to wood damage are molluscan borers called shipworms (*Banksia* spp. and *Teredo* spp.) (French: taret, German: Schiffsbauwurm, Dutch: paalworm) and crustacean borers called gribble (*Limnori*). They use wood for food as well as for shelter. Shipworms are the most destructive. The immature forms enter the wood through pinhole-sized openings that never increase in size. The shipworms excavate tunnels; eventually they may honeycomb the interior of infested wood with little exterior evidence of attack. The ant-sized gribbles extend their small tunnels into the outer few centimetres of the wood. The surface of infested wood becomes sponge-like in appearance and erodes away, particularly at the waterline.

4.2.4 Bacterial attack

Bacteria degrade all wood in almost all environments. However, their enzyme production is slow and they therefore cause relatively little damage in comparison with other degraders. Bacteria are able to degrade preserved wood or highly durable species in conditions of low oxygen concentrations.

Bacteria mainly attack cellulose. Because the cellulose and hemicellulose are protected by lignin, this kind of attack is usually restricted. Where soft rot can cause deterioration within months, bacterial attack shows significant damage only after many years. Nevertheless, in the Netherlands where many houses and buildings are built on wooden foundation piles, serious problems have arisen with slowly deteriorating wooden piles under ground water level due to bacterial attack. The bacteria can function even under conditions of little aeration. A minor drain of oxygen from the top of the pile, which is beyond the water table, can suffice. Scotch fir (*Pinus sylvestris*) piles, also called Scots pine (French: sapin ecossaise, German: Scottische Tanne, Dutch: grenen) appears to be more sensitive to this attack than are other softwoods. However, knowledge of this type of deterioration is restricted.

4.2.5 Weathering

Natural outdoor weathering occurs due to combined exposure to sunlight and moisture. The absorption of UV radiation from the sun causes degradation of the lignin, which can subsequently be washed out by water. A silver-grey surface remains primarily consisting of cellulose components. Changes in moisture content, especially when exposed to rain, cause moisture gradients and consequently stresses in the wood that can aggravate the process. The completely weathered grey layer is only a few tenths of millimetres thick and, within two millimetres from the surface, the wood is not affected any more. The effect of weathering on strength properties is very limited. Surface abrasion resistance is reduced and toughness is somewhat diminished in a continuous process. This means that weathering as such is not a threat to structural applications. However, if the moisture content in use exceeds 20% of decay and degradation by insects, this will endanger wood durability, as discussed above.

4.2.6 Chemical attack

In general, wood shows good resistance to a variety of chemicals. Its resistance to acids is superior in comparison with that of steel and concrete. It is therefore used for tanks and other containers. Strong acids and strong alkaline solutions do, however, deteriorate wood.

4.3 Protection

4.3.1 General

The deterioration mechanisms described in the previous section mean that not every wood species will have an acceptable service life under all environmental conditions. If the durability of a certain wood species is insufficient for the specified design life, measures will have to be taken. Two types of measures can then be distinguished:

- Changing the environment of the wooden structure and making it less hostile to the wood, which can be achieved by protective design.
- Making the wood more durable by using a protective treatment.

Various options are available for the second measure. These include:

- Impregnating the wood with preservatives.
- Applying protective finishes and coatings.
- Modifying the wood chemically.

These measures will be discussed further in the following sections.

4.3.2 Protective design

The secret to a long service life for wood lies in the protective design of the structure. Fungi will not damage wood with a moisture content below 22% and wood-boring beetles will not be present at moisture levels below 8%. From the above sections it will be obvious that avoiding contact with water is a major step forward in achieving a long service life. An example of a protective design is the roofed bridge in Eggiwill, Switzerland.⁶ The wooden bridge deck has a membrane impermeable to water, covered with an asphalt layer. There are numerous roofed bridges in the world that have functioned for over a hundred years.

4.3.3 Wood treated with preservative

4.3.3.1 *Classification of preservatives*

Wood preservatives (French: produits preventives, German: Holzschutzmittel, Dutch: verduurzamingsmiddelen) are generally classified into three groups:

- Oily preservatives, mainly creosote, an oily liquid that has satisfactory viscosity for wood treatment.
- Oil-borne preservatives, where a hydrocarbon solvent is used to dissolve the preservative, such as penta (pentachlorophenol, PCP), coppernaphthenate and tributyltin oxide.

- Water-borne preservatives, where inorganic salts are dissolved in water. These include chromated copper arsenate (CCA), ammonial copper arsenate, acid copper chromate and bifluorides.

4.3.3.2 *Environmental restrictions*

The use of these preservatives is increasingly restricted in a growing number of countries. The preservatives are toxic in nature which they must be in order to be effective against biological attack. Generally speaking, the environmental drawbacks are the possible leaching out to the environment during usage and leaching out during disposal. In addition, there could be the emission of contaminations in flue gases at combustion or contamination of the remaining ashes after the end of service life.

According to EU legislation, PCP shall not be used in a concentration equal to or greater than 0.1% m/m of wood. National legislation may be more stringent. In Germany, for instance, products with PCP must not contain more than 5 mg/kg PCP. In the Netherlands, lindane (hexachlorocyclohexane), pentachlorophenol and coppernaphthenate are banned, the inorganic copper, chromate and arsenic salts are barely accepted and for creosote oil the amount of benzopyrene has been restricted to 5%. If creosote is applied, the vacuum–pressure method for impregnation must be used, which results in better fixation of the preservative to the wood. Sometimes preservation is applied locally, for instance, in window frame edges where the wood is preserved with ammonium bifluoride pills.

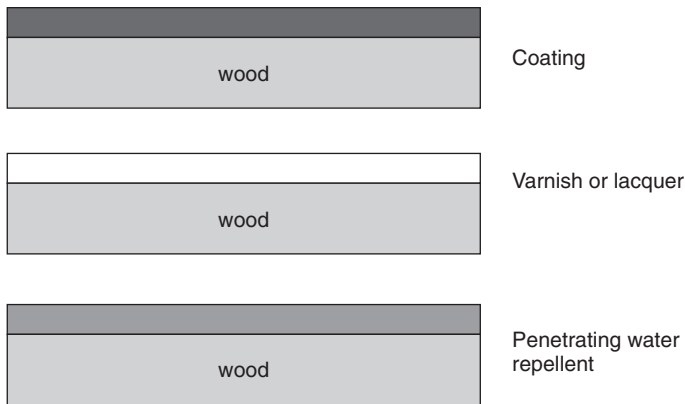
4.3.4 Protective finishes and coatings

The primary function of any wood finish is to protect the wood surface from natural weathering processes (sun and water) and to help maintain appearance. In principle, the complete protection of wood from the environment should mean that degradation mechanisms cannot reach it. Unfortunately, the durability of the protective finishes is limited. Exposure to natural weathering, especially in the behaviour of the wood itself, is of primary importance to the durability. The moisture content can change substantially, even under a coating. Furthermore, the resin and oil content of the wood, the growth pattern, the existence of knots, the reaction of the wood and other factors play an important role.

Two basic types of finishes or coatings can be distinguished:

- Those that form a film, layer or coating on the wood surface.
- Those that penetrate the wood and leave a distinct layer in the wood.

These finishes are shown schematically in Fig. 4.7.



4.7 Wood protection finishes.²

4.3.4.1 *Film-forming finishes*

Within the family of film-forming finishes, a distinction can be made between paints and varnishes, on the one hand, and lacquers, on the other. The essential difference between these two classes is that the first class contains pigments that will eliminate UV degradation by sunlight.

Paints are commonly divided into oil-based or solvent-borne systems and latex or water-borne systems. Latex paints are used more frequently than oil-based paints for finishing wood, particularly for exterior use. Because of occupational health concerns, the usage of solvent-borne paints is not allowed for the interior of dwellings in a number of countries. Paint is, however, not a preservative; it will not prevent decay if conditions are favourable to fungal growth. Clear varnishes or lacquers (French: laque ou vernis, German: Lack oder Firnis, Dutch: vernis of lak) give wood an attractive initial appearance. Other treatments either change the colour of the wood or cover it completely. Unfortunately, varnish finishes used on wood exposed to sun and rain require frequent maintenance so as to retain a satisfactory appearance. The addition of colourless UV light absorbers to clear finishes has had only moderate success in aiding retention of natural colour and original surface structure. It is generally accepted that opaque pigments found in paints and stains provide the most effective and long-lasting protection against light. Eventually, the varnish will flake and crack off, taking with it wood fibres that have been degraded photochemically.

4.3.4.2 Penetrating finishes

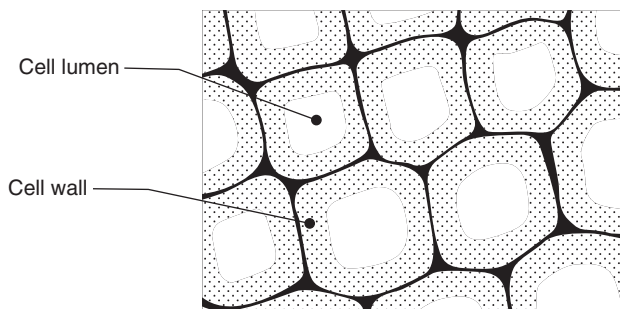
Penetrating water repellents (French: brou, German: Beize, Dutch: beits) reduce absorption of water and decrease the moisture content of the wood. The result is that the wood becomes more resistant to biological attack. Where pigments are added to the water-repellent products the wood is stained, while still permitting much of the wood grain to show through. Oily and oil-borne preservatives also have water-repellent properties. Part of the enhanced resistance of wood treated with these agents is due to this effect.

4.3.5 Wood modification

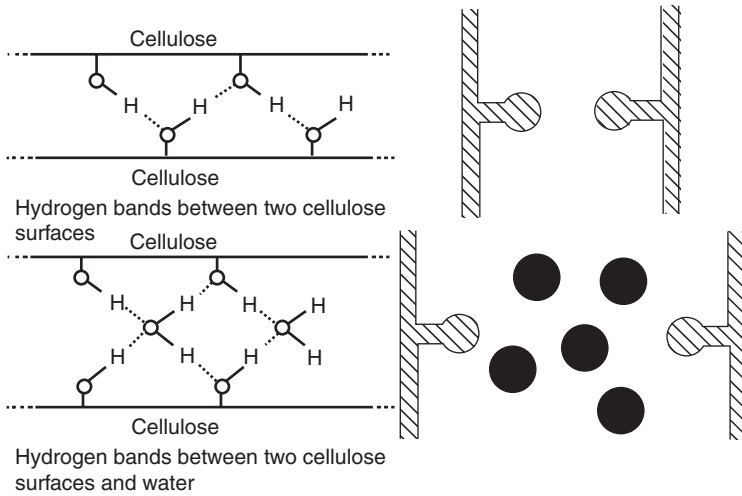
A distinction can be made between treatments where the pores are filled up and those where the wall cell is modified. The wall cells can be changed by chemical and thermal modification.

The wood microstructure is depicted in Fig. 4.8. Where the pores are filled up the wood is penetrated with a resin that fills up the lumina. Sometimes, the cell wall cavities can also be partly filled. In modified wood systems, the chemistry of the cell wall is changed. The most abundant functional group in wood is the hydroxyl group. The hydrophilic nature of this group is the main cause of the moisture-sensitive behaviour of wood, as shown in Fig. 4.9. In the modified wood treatment, the hydroxyl groups are substituted by hydrophobic groups or are cross-linked, as shown in Fig. 4.10.

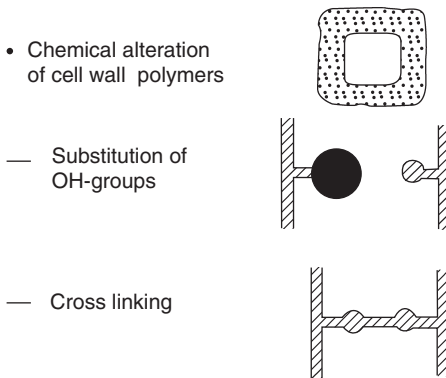
Chemical modification treatments, such as acetylation, esterification, etherification, treatments with urethane, oxidation and silanization of wood, look very promising. Most widely applied are acetylation and esterification. Figure 4.11 shows wood leaving an acetylating tank. The most developed cross-linking processes are thermal and hydrothermal treatments. Wood modification processes upgrade most woods so they become highly



4.8 Wood structure and distribution of chemical components within the cell wall.⁷



4.9 Mechanism of water uptake.⁷



4.10 Chemical modification of wood.⁸

resistant to biological attack. Poplar, for instance, which belongs to Durability Class 5 (see Section 4.4.2) is upgraded by a hydrothermal treatment to Class 2. Other important improvements are the reduced drying shrinkage and moisture uptake swelling, with reductions of up to 50%, and enhanced UV resistance for acetylated wood. On the other hand, the mechanical properties, especially of thermally modified wood, are reduced; for chemically modified wood this reduction is less pronounced.



4.11 Wood leaving an acetylating tank, AKBV plant, Arnhem, the Netherlands, 1998.

4.4 Hazard and durability classes

4.4.1 Hazard classes

The European standard EN 335 *Durability of wood and wood-based products – Definition of hazard classes of biological attack* identifies the hazard class (French: classe de risque, German: Gefährdungsklasse, Dutch: risicoklasse) of a given service environment and geographical location.

The definition of the hazard classes is given in Table 4.1.

4.4.2 Durability classes

The European standard EN 350 *Durability of wood and wood-based products – natural durability of solid wood* classifies natural durability in relation to the various agencies of biological attack. There is a five-grade scale for resistance to fungal attack (Table 4.2), a two-grade scale (susceptible

Table 4.1 Hazard classes and occurrence of biological agencies according to EN 335-1

Hazard class	Exposure	Biological attack	Examples
1	Above ground, covered (dry)	beetles, termites ^a	indoor applications, such as wooden frames of houses covered by roofs and cladding.
2	Above ground, covered (risk of occasional wetting)	fungi, beetles, termites ^a	carports, window frames, the roofed bridge discussed in Section 4.3.2
3	Above ground, not covered (risk of frequent wetting)	fungi, beetles, termites ^a	bridge beams and decks, sheds, cladding of buildings without protection
4	In contact with soil or fresh water (permanently)	fungi, beetles, termites ^a , bacteria	fences, piers, sheet pile walls, sluice doors
5	In salt water (permanently)	fungi, beetles, termites ^a , marine borers	mooring posts, piers, breakwaters, sea-defence walls

^atermites are only present locally in Southern Europe

Table 4.2 Classes of natural durability of wood exposed to fungal attack according to EN 350 and examples of wood species

Durability class	Description	Examples
1	Highly durable	teak, azobe, iroko, jarrah, bilinga
2	Durable	Balau/bangkirai, karri, merbau, western red cedar, European oak (French: <i>chêne rouvre</i> , German: <i>Eiche</i> , Dutch <i>eik</i>), robinia (French: <i>robinet faux acacia</i> , German: <i>Robinie</i> , Dutch: <i>robinia</i>)
3	Moderately durable	red merati, Douglas fir
4	Slightly durable	Norway spruce (French: <i>epicea</i> , German: <i>Fichte</i> , Dutch: <i>spar</i>)
5	Not durable	beech (French: <i>hêtre</i> , German: <i>Buche</i> , Dutch: <i>beuk</i>), ash (French: <i>frêne</i> , German: <i>Esche</i> , Dutch: <i>es</i>), poplar (French: <i>peuplier</i> , German: <i>Pappel</i> , Dutch: <i>populier</i>)

Table 4.3 Guidance in choosing wood of a specific durability class for a specific hazard class²

Hazard class	Durability class				
	1	2	3	4	5
1	S	S	S	S	S
2	S	S	S	(S)	(S)
3	S	S	(P)		(S)–(P)
4	S		(P)	P	P
5	S	P	(P)	P	P

S : Natural durability sufficient

(S) : Natural durability in principle sufficient, but under special service conditions the wood must be preserved

(S)–(P): Natural durability can be sufficient, but the choice of wood, the treatability of the wood by preservatives and the application determine whether or not the wood should be preserved

(P) : Treatment is applied, but under some service conditions, the natural durability can be sufficient

P : Treatment is required

and durable) for resistance to dry wood-destroying beetles and a three-grade scale (susceptible, moderately durable and durable) for resistance to termites and marine borers. Determination of the classification of a wood species requires the performance of the wood in standardized tests. In general, sapwood is not durable. The examples of wood given in Table 4.2 are all for heartwood.

4.4.3 Matching hazard class and durability class

For each application in a specific service environment and geographical location, wood of a specific minimum durability class must be chosen. Table 4.3 offers guidance.

4.5 Maintenance and restoration

4.5.1 Preventive maintenance

If wood performance declines as expected according to the design of the structure, but is still beyond an acceptable level, we can improve it so that the service life of the wooden product or structure is extended for another period, as described in Section 6.7. This is known as preventive

maintenance. If the wood was unprotected it can be protected according to one of the methods presented in Section 4.3. If the skin of the wood is weathered, it should be removed before a protective finish or coating is applied.

4.5.2 Curative restoration

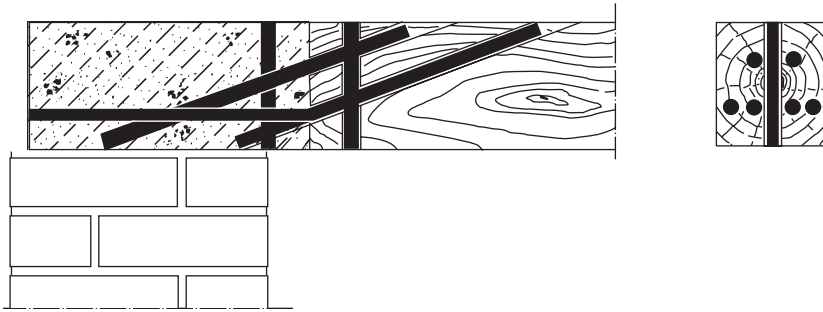
When the observed decline in wood performance has not been foreseen, but performance is still beyond a critical level, as shown in Fig. 6.1, curative measures are required to stop deterioration. In the case of wood affected by fungi or insects, destroying them should halt the erosion. This can be done either by increasing the temperature, by ultrasonic treatment (beetles only) or by applying fumigants. Increasing the temperature beyond 60 °C kills fungi (not the spores) and beetles. This is not applicable to every structure, but the remedy can usually be applied in houses.

Fumigants used to protect wood fall into two broad categories: those that rapidly destroy the insects or fungi, but do not remain in the wood for long, and agents that will stay in the wood for many years. The latter will, of course, retain their function, while in the former, new colonizing can occur unless other protective measures are taken. An example of a short-term fumigant is methyl bromide. All fumigants are toxic chemicals and are hazardous for human beings when misapplied. Long-term fumigants can evaporate slowly and can endanger the indoor climate. In many countries the use of these fumigants for curative wood restoration is banned or subject to strict rules.

4.6 Repair

When a wooden structure is seriously decayed, it is best replaced. However, the wood is often only locally affected and replacement of the whole wooden building component could be cumbersome and expensive. Occasionally, when it concerns monuments, replacement is considered undesirable because preservation of the cultural heritage must be considered. Instead there exists a well-known technique to remove the affected wood and replace it with a new, better decay-resistant wood and connect the old and the new wood by gluing. However, for structural applications this is often not safe enough. Frequently, reinforcement must be added to strengthen the connections, which, as shown in Fig. 4.12,⁹ enhances the load transfer between the old and new wood. The reinforcement can consist of carbon rebars, stainless steel rebars or other such items.

Sometimes, the affected wood is not completely replaced, but repairs are carried out using an epoxy mortar. Another possibility is to repair and apply



4.12 Example of repair of a deteriorated wooden beam section using new wood reinforced with glass fibre reinforced polyester rods.¹¹

an exterior reinforcement onto the wooden structural component. This is discussed further in Section 5.4.4. In any case, the causes of decay must be removed by protective measures to avoid further moisture entrance or by destroying the fungi, beetles and so forth with the techniques described above under maintenance. If no protective measures are taken to prevent further entrance of moisture, the old non-replaced parts in the vicinity of the replaced parts must be protected using preservatives; e.g. bifluoride pills are used to preserve the old wood of window frames adjacent to the new wood.

4.7 References

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5.1 Introduction

New materials for structural applications developed in other industrial sectors have been adopted in civil engineering practice, but in the construction industry itself there are also incentives to develop new materials. These incentives are:

- Failing durability of structures manufactured from traditional materials. Thus the US Federal Highway Administration estimates that nearly 30% of all bridges are either structurally deficient or functionally obsolete.
- Improved sustainability. Higher-strength materials allow for a leaner structural design with more durable materials resistant to various environments and with an extended service life, which generally decreases the environmental impact of building.
- Cost optimization over the entire life cycle (see Section 6.8). Thus low-maintenance structures are generally favoured because of the high cost of maintenance, repair or replacements, especially in countries with high labour costs.

Materials that perform better during the building process are more easily applied and are more resistant or have a higher strength than the traditional materials discussed in the previous chapter. They are called high-performance materials.

For the construction industry, the following materials for structural applications will be considered:

- High-performance concrete.
- High-performance steel and other metals.
- Fibre-reinforced polymers (FRP).

These materials will be discussed briefly in terms of their general properties and their durability will be described in greater depth.

5.2 High-performance concrete

5.2.1 Introduction

Any concrete that satisfies certain criteria aimed to overcome limitations of conventional concrete can be called high-performance concrete (HPC). This includes concrete that shows:

- A much improved resistance to environmental influences (durability in service): high-durability concrete.
- A substantially increased structural capacity while maintaining adequate durability: high-strength concrete.
- A significantly reduced construction time, e.g. to permit rapid opening or reopening of roads to traffic, without compromising long-term serviceability: high early-strength concrete.
- The ability of the fresh concrete to flow easily to fill form work and encapsulate reinforcing bars through grout action only while maintaining homogeneity: self-compacting or self-consolidating concrete (SCC).

It is, therefore, not possible to provide a unique definition of HPC without considering the performance requirements of the intended use of the concrete.

A definition of high-strength concrete according to the US Federal Highway Administration is given in Table 5.1. According to this table, it appears that an increase in strength coincides with improved durability parameters. This relationship will only be valid within a range of compositions made with the same raw materials; for instance, a change in cement type can cause a major change in the durability parameters, e.g. the chloride penetration rate, without a major change in strength.

An important topic covered in this book is design for durability. Design for durability of concrete structures is discussed in Chapter 3. As stated in Section 3.4.7, for natural weathering conditions, even for aggressive ones, highly durable concrete structures can be made that are essentially corrective-maintenance free (Section 6.7.4) for a very long time, with no other measures than a dense plain concrete, albeit with the application of cements or concretes with additions. An example is the concrete used in the Eastern Scheldt barrier in the Netherlands that was designed for a 200 year period with the cover to be replaced after 75 years². This concrete can be regarded as a high-performance concrete although it is, in fact, a normal-strength concrete. Its strength class is C35/45 according to EN 206-1. High resistance against chloride penetration is realized by applying a CEM III/B blast furnace slag cement according to EN 197-1 (see Section 3.4.7). This discussion is restricted to the durability aspects of

Table 5.1 Definition of HPC according to Federal Highway Administration¹

Performance Characteristics	Standard test method	FHWA HPC performance grade			
		1	2	3	4
Freeze-thaw durability (X = relative dynamic modulus of elasticity after 300 cycles)	AASHTO T 161 ASTM C 666 Procedure A	$60\% \leq X < 80\%$	$80\% \leq X$		
Scaling resistance (X = visual rating of the surface after 50 cycles)	ASTM C 672	X = 4, 5	X = 2, 3	X = 0, 1	
Abrasion resistance (X = average depth of wear in mm)	ASTM C 944	$2.0 > X \geq 1.0$	$1.0 > X \geq 0.5$	$0.5 > X$	
Chloride penetration (X = coulombs)	AASHTO T 277 ASTM C 1202	$3000 \geq X > 2000$	$2000 \geq X > 800$	$800 \geq X$	
Strength (X = compressive strength)	AASHTO T 2 ASTM C 39	$41 \leq X < 55 \text{ MPa}$ ($6 \leq X < 8 \text{ ksi}$)	$55 \leq X < 69 \text{ MPa}$ ($8 \leq X < 10 \text{ ksi}$)	$69 \leq X < 97 \text{ MPa}$ ($10 \leq X < 14 \text{ ksi}$)	$97 \text{ MPa} \leq X$ ($14 \text{ ksi} \leq X$)
Elasticity (X = modulus)	ASTM C 469	$28 \leq X < 40 \text{ GPa}$ ($4 \leq X < 6 \times 10^6 \text{ psi}$)	$40 \leq X < 50 \text{ GPa}$ ($6 \leq X < 7.5 \times 10^6 \text{ psi}$)	$50 \text{ GPa} \leq X$ ($7.5 \times 10^6 \text{ psi} \leq X$)	
Shrinkage (X = microstrain)	ASTM C 157	$800 > X \geq 600$	$600 > X \geq 400$	$400 > X$	
Specific creep (X = microstrain per MPa)	ASTM C 512	$75 \geq X > 60 \text{ MPa}$ ($0.52 \geq X > 0.41 \text{ psi}$)	$60 \geq X > 45 \text{ MPa}$ ($0.41 \geq X > 0.31 \text{ psi}$)	$45 \geq X > 30 \text{ MPa}$ ($0.31 \geq X > 0.21 \text{ psi}$)	$30 \text{ MPa} \geq X$ ($0.21 \text{ psi} \geq X$)

high-strength concrete, high early-strength concrete and self-compacting concrete.

Since high strength and high durability can coincide, it is possible to use high-performance concrete for applications that normally are the exclusive domain of steel. For instance, high-strength concrete is used in steel bridge decks sensitive to fatigue corrosion in the Netherlands, as discussed in Section 2.3.2.12. The high-strength concrete dissipates the stresses due to wheel loads. Another example is the intention of the Dutch Ministry of Transport, Public Works and Water Management to develop a lift-bridge in high-strength reinforced concrete because it requires less maintenance than does a steel bridge.³

5.2.2 High-strength concrete

5.2.2.1 *Composition and microstructure*

High-strength concretes are obtained by the following measures:

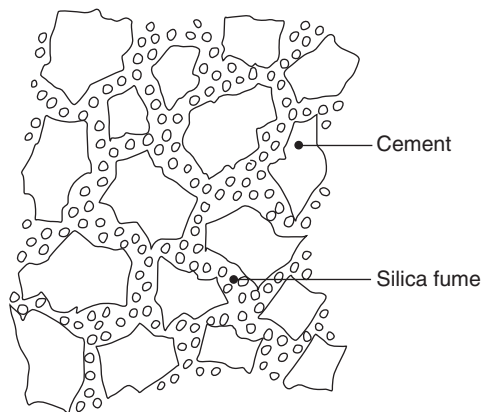
- Low water/cement ratio (< 0.4).
- Use of high-range water reducers, sometimes referred to as superplasticizers.

The effect of these measures on the microstructure is a decrease in the average pore size, especially a decrease in the capillary pores ($0.03\text{--}1\text{ }\mu\text{m}$), and a more homogeneous cement paste. However, it has to be noticed that not every high-range water reducer has a positive effect. Mohammed et al.⁴ report that concrete with a polycarboxyl type of high-range water reducer has shown relatively high ingress of chloride ions in test samples.

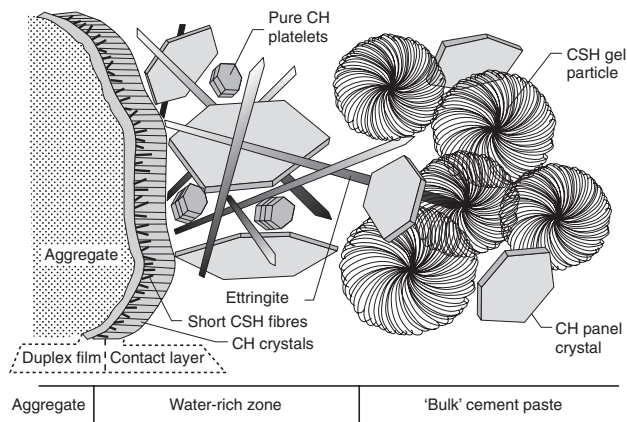
It is optional whether the following are applied:

- Highly reactive pozzolans, such as silica fume and metakaoline. Their reactivity is partly due to their very fine nature (particles smaller than a few micrometres).
- Dense particle packing in the micron and sub-micron area. The very fine reactive pozzolanic particles are able to fill up the open spaces between cement particles, as shown in Fig. 5.1.

The effects of these applications are: a decrease in the average pore size, a substantial decrease in the thickness of the porous interfacial zone around aggregates (see Fig. 5.2), a decrease of the free lime content ($\text{Ca}(\text{OH})_2$) and an increase of the CSH-gel of the concrete. The use of highly reactive pozzolans, such as silica fume, decreases pore water alkalinity. The effects on the microstructure and on the chemical composition in terms of durability are discussed in the following section.



5.1 Packing of particles of cement and silica fume.



5.2 Transition zone between aggregate and cement paste.

5.2.2.2 *Durability parameters*

Sensitivity to curing

In general, high-strength concrete is more sensitive to curing than is normal-strength concrete due to the decreased bleeding capacity of the concrete. In ordinary concrete, segregation occurs and a layer of water is formed at the surface. This layer protects the young concrete against rapid dry-out. In high-strength concrete with better cohesion due to better particle packing, this layer is thinner or even absent. If young concrete dries out prematurely, it is likely to crack (plastic shrinkage) and cement hydration will virtually come to a halt, leaving a porous, permeable concrete.

Autogeneous shrinkage

High-strength concrete shows a strong increase in autogeneous shrinkage, which can lead to cracking, especially when highly reactive pozzolans, such as silica fume, are applied. This shrinkage is due to the decrease of solid material volume during the reaction with water. It causes self-desiccation with increased sensitivity to cracking. Cracks in concrete have a negative effect on the penetration rate, although in general the effect appears to be small as long as the cracks are less than 0.3 mm wide, as required in most standards.

Chloride-initiated corrosion

An important parameter for the rate of chloride ingress is the chloride diffusion coefficient (see Section 3.4.3.1). The effect of decreasing the water/cement ratio on the resistance to chloride penetration is minor; possibly by a factor of 2 over the range of interest. However, reactive pozzolans can have a much greater effect. Examples of the effect of various measures to decrease the chloride diffusion coefficient are given in Table 5.2.⁵ The use of highly reactive pozzolans, like silica fume, decreases pore water alkalinity, which in turn decreases the threshold chloride concentration required for the onset of corrosion (see Section 3.4.5). Concurrently, the percentage of chloride bound to the cement can be diminished and consequently the free chloride concentration is increased. This effect is very noticeable with silica fume. A positive effect of the changes in microstructure is the increase in the electrical resistivity of concrete, as shown in Fig. 5.3.⁶

Frost–thaw de-icing salt resistance

In general, it can be stated that both the frost–thaw resistance and the de-icing salt scaling resistance increase at greater concrete strengths. However, results and conclusions are somewhat conflicting.⁷ Lack of correlation between existing laboratory performance tests and field experience makes a definite assessment difficult. Some researchers are of the opinion that air entrainment is not required for high-strength concrete, but others contradict this. In ordinary concrete, air-entrainment is used to enhance the frost–thaw resistance (see Section 3.3.3). However, air entrainment decreases strength, which is in conflict with the aim of producing a high-strength concrete. If air-entrainment is applied, the required spacing factor appears to be smaller because of the reduced permeability of the concrete, which hinders water access to the air bubbles.

Table 5.2 Values of the chloride diffusion coefficient for various concrete mixtures according to Bamforth and Price⁵

Mix type	Curing	Diffusion coefficient $\times 10^{-12} \text{ m}^2/\text{sec}$
CEM I (OPC)	W	2.31
	E	1.43
	M	2.38
	average	2.04
30% fly ash (pfa)	W	0.04
	E	0.04
	M	0.04
	average	0.04
CEM I OPC + water reducer	W	1.47
	E	2.01
	M	1.09
	average	1.52
70% slag (GGBS)		0.02
		0.02
		0.02
	average	0.02
waterproofed		1.62
		1.83
		1.85
	average	1.77
8% SF		0.31
		2.05
		0.27
	average	0.88

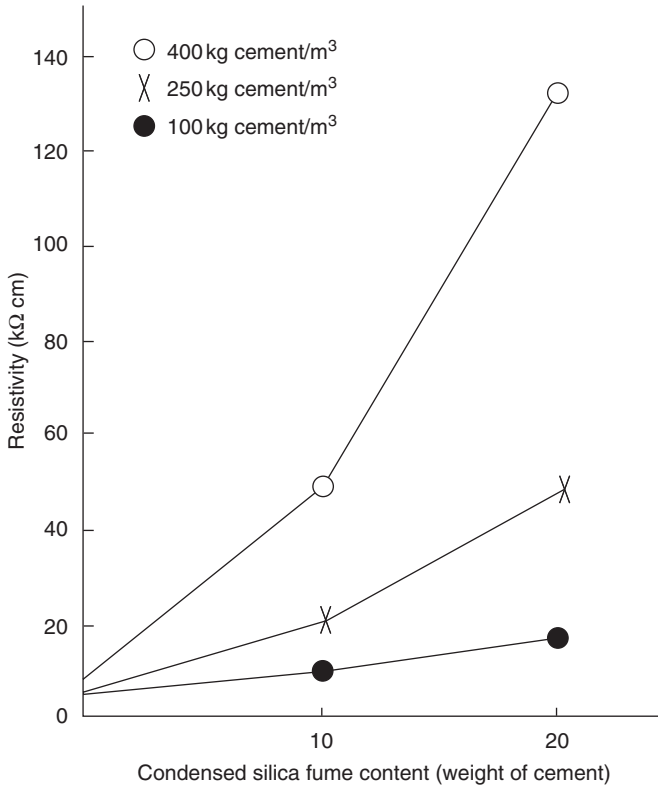
Key

GGBS	Ground granulated blast furnace slag
waterproofed	Hydrophobic agent
SF	Silica fume
W	Wet cured
E, M	Dry cured

Chemical resistance

In general, when using reactive pozzolans, the resistance to chemical degradation mechanisms, as discussed in Section 3.2, will increase. This is due to:

- The decrease of the free lime present. Free lime plays an important role in the formation of expansive salts, such as ettringite and also in the alkali aggregate reaction expansion.
- The decrease of the permeability and diffusion coefficients. These coefficients play a part in all chemical degradation mechanisms.
- The decrease in pore water alkalinity. Pore water alkalinity can contribute to alkali aggregate attack.



5.3 Effect of silica fume on the electrical resistivity of cement.⁶

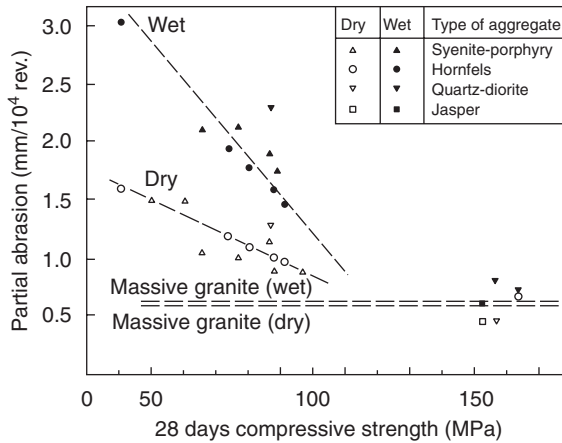
- A reduction in the thickness of the water-rich, porous interfacial zone (see Fig. 5.2). This zone plays an important part in both expansive ettringite formation and alkali-silica expansion.

Abrasion-erosion resistance

High abrasion or erosion resistance can be required for various concrete structures in situations such as:

- Fast flowing water at dams that generate hydroelectricity.
- Tidal movements along dams and bridge piers where abrasion occurs by sand particles.
- Drifting ice along concrete piles.
- Pavements and bridge decks subject to studded tyres.

There appears to be a linear relationship between compressive strength and abrasion as shown in Fig. 5.4.⁸



5.4 Relationship between abrasion resistance and compressive strength.⁸

5.2.3 Self-compacting concrete

Self-compacting concrete was initially developed in Japan and needs no compaction by vibration. It fills up the mould easily and surrounds the reinforcement without bleeding or any other form of segregation, at the same time expelling entrapped air in the absence of honeycombing. Very good cohesion is due to the optimization of the fine particle packing in the micron and sub-micron range, while ability to flow is achieved by the use of high-range water reducers. When using these concrete mixtures that flow easily and do not need compaction to consolidate, some practical problems with large pours and congested reinforcement can be overcome. Applying viscosity-modifying admixtures, such as Welan Gum, which have made it possible to control and stabilize highly fluid mixtures has made further improvements.

Another development is the application of a special clay, attapulgite, as an admixture, which has an effect similar to that of viscosity-modifying agents. It is claimed these clays can be applied without further adjustments to the concrete mix design and that large quantities of fines can thus be avoided⁹ and are then referred to as 'tolerant concretes'. Reactive pozzolans are often applied in self-compacting concrete so as to provide the required fineness. In such cases, the concrete is not only self-compacting, but is also of high-strength and the durability aspects as discussed in the previous section on high strength are applicable. A positive aspect of self-compacting is that the concrete fills up the mould completely and fully embeds the reinforcement. Homogeneity of the concrete on the meso-level

is, therefore, improved. In Japan, the development of self-compacting concrete in the early 1980s was pervaded with problems of durability caused by insufficient compaction.¹⁰ This coincided with a trend to reduce the number of skilled workers.

The literature on the durability aspects of self-compacting concrete is limited. Den Uijl¹¹ makes a comparison between self-compacting concrete and ordinary concrete for two compressive strength grades of concrete, of which the composition and the carbonation rate are shown in Table 5.3. The carbonation rate for self-compacting concrete is higher than that of the ordinary concrete of the same strength grade. For the B35 grade, chloride penetration of concrete cubes immersed in a sodium chloride solution appears to be higher in the self-compacting concrete. For the B65 grade, the difference is small. It should be noted that the main cement applied in these tests is blast furnace slag cement, CEM III/B 42.5, that already has a high chloride penetration resistance (see Section 3.4.7.1). There are, as would be expected, many mix designs for self-compacting concrete with a variety of

Table 5.3 Composition and carbonation depth of self-compacting concrete (SCC) and reference concrete (Ref) for two compressive strength classes B35 (C30/37 EN 206) and B65 (C55/67 EN206)¹¹

Mixture	B35-SCC	B65-SCC	B35-Ref	B65-Ref
w/c	0.65	0.49	0.45	0.40
w/p ^a	0.61	0.44	–	–
Materials	kg/m³	kg/m³	kg/m³	kg/m³
CEM II/B 42.5	270	184	350	300
CEM I 52.5 R	–	185	–	100
Powder coal fly ash	273	185	–	–
Water	176	180	157	159
Fine aggregate (river sand) 0–4 mm	670	662	1040	830
Coarse aggregate round (river gravel) 4–16 mm	870	–	851	–
Coarse aggregate crushed 4–16 mm	–	900	–	975
Superplasticizer (polycarboxylate-ether) Cugla LR	1.62	2.24	–	–
Superplasticizer (polycarboxylate-ether) Cugla HR	0.65	0.90	–	–
Superplasticizer (lignine sulphonate) Addiment BV1	–	–	–	1.6
Superplasticizer (lignine sulphonate) Addiment FM 951	–	–	–	4.8
Carbonation depth after 9 months	mm	mm	mm	mm
50% RH 20 °C	8	1	4	1
50% RH 20 °C 3% CO ₂	50	16	29	15

^aw/p = water/ (cement + k × fly ash) ratio according to EN 206

raw materials. Nevertheless, the results indicate that the durability of self-compacting concrete is not superior to that of ordinary concrete compositions.

5.3 High-performance steel

High-strength steels have existed for many years but have not been very successful in the construction market because of poor weldability and low toughness. The steel industry has put a considerable amount of effort into the development of new high-performance steels with the following characteristics in relation to conventional steel:

- Higher strength and toughness.
- Improved weldability and decreased sensitivity to hydrogen-induced cracking.
- The same or improved corrosion resistance.

This has been achieved by reducing substantially the carbon and sulphur contents and by applying the techniques of ‘thermo-mechanical rolling controlled processing’ (TMCP), and ‘quench and temper’ (QT).^{12,13} Steel grades such as S355 and S460 (EN10113) that are made using these techniques show good weldability in comparison with conventional steels, notwithstanding their high yield stress. Even higher strength grade steels appear to be possible without losing the characteristics targeted.

As discussed in Section 2.3.2.12 and Section 6.7.6 fatigue resistance and fracture toughness are of special interest for bridges and more particularly for bridge decks. These high-performance steels show a much better fracture toughness than conventional steels, while the fatigue resistance is similar or better. The corrosion resistance appears to be similar or slightly better than that of conventional steel, although long-term atmospheric corrosion tests are not available at present (durability, mainly focussing on the corrosion resistance of steel and steel alloys, is discussed in Chapter 2). High-performance steels can therefore be used to increase the service life of structures, especially where dynamic loading is applied, such as in bridge decks, as well as for a leaner construction where it is likely that the higher costs of this steel are easily compensated by a lower quantity of material used.

5.4 Fibre-reinforced polymers

5.4.1 General

Fibre-reinforced polymers (FRP) are composed of fibres and a polymer matrix. The fibres provide the primary strength and the matrix holds the reinforcement in its proper orientation for optimum properties. The fibres

used in civil engineering structures are made from glass (GFRP), carbon (CFRP) and poly-aramid (AFRP). The polymer resins are either thermoplastic or thermosetting resins. They are extensively used in diverse areas ranging from boats, consumer goods, aircraft, automotive components and industrial applications. Their use in civil engineering applications is relatively modest but growing.

FRP composites offer the following properties when compared with other materials used for structural applications: Their advantages are:

- Good resistance against natural weathering conditions.
- Low to no maintenance.
- Light-weight.
- High strength to weight ratio.
- Flexibility in design.
- High electrical resistance, but adjustable.
- Easy to tool.
- Colour and finish moulded-in.

Their disadvantages are:

- Low fire resistance.
- Lower stiffness.
- Higher initial production cost.

With respect to the fire resistance, it should be noticed that due to the low heat conductivity of FRPs, failure in a fire could take more time than it would in a comparable steel structure.

When considering the durability of engineering structures, the following uses of FRP are considered important:

- As the main composite construction material in structures with functions similar to those traditionally made of steel, concrete or wood. The composite shows some similarities to reinforced concrete. Higher concentrations of reinforcement will be applied where there is a requirement for tensile strength, whereas higher polymer content will be applied where there is no such requirement. In general, the average fibre content will not exceed 20%.
- As a composite material so as to reinforce existing structures, either to restore the minimum performance requirements or to extend the function of the structure. The fibre percentage will be in the order of 50–70% for strips and 25–35% for sheets.
- As bars and tendons to replace steel cables, tendons and bars embedded in concrete structures. The fibre percentage can then be as high as 80%.

Fibres can be manufactured in a continuous or discontinuous (chopped) form. Continuous fibres are used for structural applications, occasionally in

combination with chopped fibres. The fibres are often produced as strands consisting of hundreds of individual fibres, each individual fibre having a diameter ranging from 5 to 20 μm . Various matrices can be applied but fibre polyester for glass, vinyl ester for aramid and epoxy for carbon fibres are used most frequently. It is these three composites that will be discussed. The first issue under consideration is their durability and subsequently, their practical application will be described.

5.4.2 Durability aspects of FRP composites

5.4.2.1 *Manufacturing process*

Apart from the constituent material, FRP properties, notably durability, depend on the manufacturing process and quality control. Manufacturing processes can vary considerably and determine durability to a large extent. Sensitivity to deteriorating mechanisms, as discussed in the following sections, can be substantially higher for FRP manufactured on site when repairing structures using processes like wet lay-up with low-temperature cure epoxies than for FRP products manufactured using autoclave moulding in a factory with high-temperature epoxies. Differences can result from the level of bonding between fibre and resin, the void content, the degree of cure and the level of process-induced stresses.¹⁴

5.4.2.2 *Strength*

Glass, carbon and polyaramid fibres are very strong and stiff. Table 5.4 compares the properties of these fibres with those of mild steel. Figure 5.5 shows the uniaxial tensile stress strain curve for these materials.¹⁵ Fibres are stronger than steel, less stiff than or as stiff as steel and show a more brittle behaviour; i.e. there is linear elastic behaviour until failure, while steel yields. The variation in strength and stiffness depends on the type of carbon fibres used. These fibres are normally based on either pitch or polyacrylonitrile (PAN) used as a raw material. Pitch fibres are fabricated from refined petroleum or coal pitch. PAN fibres are made of polyacrylonitrile that is carbonized through burning. The PAN fibres offer the highest strength and/or E-modulus.

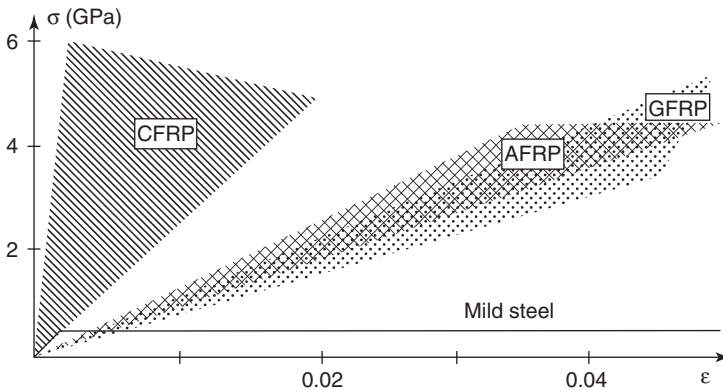
5.4.2.3 *Moisture sensitivity of polymer*

FRP composites have a long history of use in marine vessels, piping, corrosion equipment and underground storage tanks. Experience has shown that they can be successfully engineered to have long service lives in contact with moisture and aqueous solutions. However, it is a misconception that

Table 5.4 Typical properties of fibres for FRP¹⁴

Material	Modulus of elasticity (GPa)	Tensile strength (MPa)	Ultimate tensile strain (%)
Carbon fibre			
High strength	215–235	3500–4800	1.4–2.0
Ultra-high strength	215–235	4500–6000	1.5–2.3
High modulus	350–500	2500–3100	0.5–0.9
Ultra-high modulus	500–700	2100–2400	0.2–0.4
Glass fibre			
E	70	1900–3000	3.0–4.5
S	85–90	3500–4800	4.5–5.5
Aramid fibre			
Low modulus	70–80	3500–4100	4.3–5.0
High modulus	115–130	3500–4000	2.5–3.5
Mild steel	200	400	25*

* Yield strain = 0.2%

5.5 Uniaxial tension stress–strain diagrams for different unidirectional FRPs and mild steel.¹⁵ CFRP = carbon FRP, AFRP = aramid FRP, GFRP = glass FRP.

FRP composites and polymers are ‘water-proof’ since moisture diffuses into all organic polymers, leading to changes in properties. The primary effect of absorption is on the resin itself through hydrolysis and saponification that cause both reversible and irreversible changes in the polymer structure. Apart from a change in the mechanical properties, osmotic effects in the case of a poorly cured resin can cause surface blistering, as discussed in

Section 3.8.4.4. The absorption of moisture can also cause deleterious effects to the fibre-matrix bond resulting in loss of integrity. Furthermore, it causes a depression in the glass transition temperature¹⁴ (see Section 5.4.2.7).

5.4.2.4 *Alkaline and acid attack*

Glass fibres used for continuous fibre reinforcement can be classified into three types: E-glass fibres, S-glass fibres and alkali-resistant AR-glass fibres. E-glass fibres have a low alkali resistance. S-glass fibres are stronger and stiffer than E-glass, but still not resistant to alkali. To prevent glass fibre from being eroded by cement alkali, a considerable amount of zircon is added to AR-glass fibres.

The alkaline resistance of glass fibres is important when such fibres are in contact with an alkaline material, such as concrete. Where stagnant water comes into contact with glass alkalis, constituents can leach out of the glass and build up a highly alkaline environment in the water that will in turn affect the silicate structure of the glass. This phenomenon is known to have occurred in glass fibre communication cables. Carbon fibres do not deteriorate in a cementitious highly alkaline environment, but aramid fibres embedded in a vinyl ester alkaline medium may be susceptible.

All three composites resist acids well.

5.4.2.5 *Stress corrosion, stress rupture, creep, fatigue*

The effect of alkaline attack, as discussed in the previous section, is aggravated if the glass fibres are under stress and suffer stress corrosion. Stress corrosion depends on the fibre, the matrix, the stress level, the time and the environment. GFRP and AFRP can exhibit tensile rupture under sustained stress, a phenomenon known as stress rupture. The tensile strength of GFRP can drop to low values when the material is under permanent tension.

Carbon fibres are relatively unaffected by stress corrosion up to high stress levels. GFRP and AFRP are both sensitive to creep. Fatigue behaviour of the three FRPs discussed in this chapter is, in general, superior to that of steel.

5.4.2.6 *UV radiation*

Polymeric materials degrade when exposed to UV radiation, which can cause dissociation of chemical bonds. The radicals formed then react with oxygen and chains in the polymeric materials become broken, cross-linking can occur and side-chains split off. The polymer matrix deteriorates first

because it surrounds the fibres. Of the fibres discussed here, glass and carbon fibres are largely unaffected by UV radiation, while aramid fibres are only slightly affected. In general, FRP deterioration is only minor and determined by the resin matrix applied. A polyester matrix is more sensitive to UV radiation than is an epoxy one.

If FRP composites are exposed to the sun, it is recommended that the FRP composites are protected against UV radiation by applying paint. Acrylic-based and polyurethane paints generally show good UV resistance. The colour of the paint is also important; white paint reflects most of the sun's radiation, while black paint absorbs most of it. On a sunny day, this can mean a temperature difference of 20 degrees centigrade between a black- and white-coated FRP, which can have a major effect on the performance of the reinforcement of existing structures with FRP strips or sheets fixed to the surface with an adhesive (see Section 5.4.4).

5.4.2.7 *Glass transition temperature*

As previously stated (in Section 3.8.4.3), thermo-resins have a glass-rubber transition temperature in the usage temperature range of structures. Above the glass transition temperature, the modulus of elasticity decreases dramatically and this results in reduced stress transfer. The ultimate load-carrying capacity can decrease by 30–40%.¹³ It is therefore important to ensure that the glass transition temperature is higher than the maximum service temperature of the structure or that part of the structure that contains the FRP reinforcement. One also has to be aware that water absorption can cause a substantial decrease of the transition temperature. Hence it is critical that the FRP composite has a transition temperature that is at least 30 degrees above the maximum service temperature so as to allow for this potential depression.¹⁴ If the FRP is exposed to the sun, applying a white coat of paint can reduce its maximum temperature.

5.4.3 Applications in structures with functions similar to those of traditional materials

A growing number of mostly small span bridges are built partly or completely using FRP. Especially in the USA, where the failure of bridges made from reinforced concrete is widespread, this relatively new material has been welcomed as a solution to the problems of corrosion and de-icing salt deterioration. Thus the 11 m long and 8 m wide Tech-21 bridge that consists fully of FRP composites is being built in Butler County, Ohio, a state where de-icing salt damage is severe. The different sections all consist of glass fibres embedded in various matrices. The total bridge weight is only about 12 tons.

Another example is an FRP sluice door in the Netherlands. The door consists of a horizontal corrugated sheet manufactured by hand lay-up and fixed in a U-frame. In general, stiffness is not very important for sluice doors so the relatively low stiffness of FRP is therefore not a disadvantage in this application. Here again, the better corrosion resistance of FRP is a deciding factor, although initial costs are substantially higher. The Ramspol *balgstuwkering* (bellows water barrier) in the 'Zwarte Meer' ('Black Lake') in the Netherlands is intended to protect the land against flooding. At times of a high risk of flooding, a bellow made of fibre-reinforced polymer canvas is inflated with air. The inflated bellow floats on the water and forms a barrier against water ingress. The bellow is mostly under the water level and only in times of incipient flooding will it be exposed to the air. The bellow is made of a fibre-reinforced rubber, which is, in fact, not strictly speaking, an FRP.

In general, structural use of FRP is subject to highly conservative safety factors, thus a strength safety factor of 4 to 5 is applied based on the combined action of stress rupture and fatigue, whereas much lower factors are the rule for steel. This is a very careful approach, considering that, under sustained loading, glass fibre-reinforced composites can be used at a stress level of 25–30% of the ultimate stress.

In most of the applications of FRP described, long-term durability depends strongly on the use of an appropriate gel-coat on the FRP surface that can effectively protect the underlying FRP against the deterioration mechanisms discussed.

5.4.4 FRP as reinforcement in existing structures

5.4.4.1 General

The most extensive use of FRP in civil engineering is as reinforcement in existing structures where, usually, the FRP is externally bonded. Extensive information on the design and use of externally bonded fibre-reinforced polymer reinforcement (FRP EBR) in reinforced concrete structures is given in Reference 15. FRP can also be used for reinforcing steel and wood structures. Carbon fibre reinforced CFPR strips are the most frequently applied for reasons already given (Section 5.4.2.2).

Systems used to connect the FRP reinforcement to the existing structure are:

- Wet lay-up.
- Adhesion of prefabricated elements.
- Specials systems such as wrapping, prestressing.

Adhesion of prefabricated elements is the most frequently applied technique and is similar to externally bonded steel reinforcement. However, the

use of FRP solves some of the problems associated with steel plates, such as difficulties in positioning the heavy steel plates, the need for scaffolding due to weight, limited length, the threat of corrosion and the need to protect the steel plates.

Another application of FRP is the reinforcement of concrete columns by wrapping FRP sheets around them.¹⁶ An alternative approach that has been adopted for column strengthening is the use of prefabricated composite shells, also called jackets. The annulus between the FRP shell and the existing column is filled up with a cementitious grout or an epoxy material. If these techniques are used on columns that are subject to chloride-initiated reinforcement corrosion, it is essential that the corrosion be stopped prior to applying the FRP reinforcement. Although laboratory tests showed a decrease in the rate of corrosion after wrapping,¹⁷ practice has shown that corrosion continues.¹⁸ Corrosion could be stopped by applying a cathodic protection system, for instance, by applying a zinc mesh as a sacrificial anode system inside the jacket, as discussed by Davison et al.¹⁹

5.4.4.2 *Durability*

The adhesive has the function of transferring the load from the structure to the FRP, so that full composite action can develop. Generally, the adhesive is epoxy resin-based. An epoxy resin is then mixed with a hardener and, depending on the application, other constituents, such as fillers or toughening or softening agents. The craftsmanship of surface preparation and the application of adhesive and reinforcing material are of great importance to the quality of the external reinforcement.

Durability aspects of the FRP composites were discussed in Section 5.4.2 where the adhesive and the interface between the structure and the FRP were investigated. The effects discussed are also applicable to wet lay-up where the fibre/polymer composite is manufactured by applying both fibre and polymer onto the surface; hardening of the matrix occurs after application.

Attention should be paid to a number of aspects.

Glass transition temperature

As for the FRP itself, the adhesive must also have a glass transition temperature beyond the maximum service temperature. The effect of moisture absorption by the resin at the transition temperature, as discussed in Section 5.4.2.4, should be taken into account. This may present difficulties, particularly if the strengthened member of the structure is subjected to high temperatures.

Temperature dependence

The properties of the adhesive as well as the matrix of the FRP are temperature dependent. This should be taken into account by assessing the performance of a plate-bonded composite beam. In general, load tests have shown that failure occurs in the adhesive at higher temperatures in the service range, while interlaminar plate failure in the FRP has been observed at low temperatures.

Fire protection

In case of fire, the FRP external reinforcement will lose its function quickly. The adhesive will weaken and the polymer matrix will also be affected. When fire protection is provided, the FRP will stay in function longer. The length of the functional period has to be determined empirically.

Alkaline attack of adhesive

Since the adhesive of prefabricated FRP reinforcement is in direct contact with the concrete substrate, the adhesive must be alkaline resistant. In general, epoxy resins show good resistance to alkaline attack.

Frost–thaw

Most tests show that the performance of externally bonded FRP is little affected in freeze–thaw cycles. Problems may arise if the quality of the concrete is poor.¹²

5.4.5 FRP rods and tendons for reinforcing concrete

Pultruded FRP rods and tendons can reinforce concrete replacing steel rebars or prestressed concrete which in turn replace prestressing or post-tensioning cables or tendons. Pultrusion is possible in combination with fibre mats at an angle of 45 degrees. The use of FRP rods or tendons is mainly advocated for durability reasons, notably for corrosion resistance. Occasionally there can be other reasons such as their use in rooms where steel in walls and floors is not allowed because of installed magnetic resonance devices.

There has been much attention paid in the literature to the bond strength between the FRP reinforcement and concrete. Stress transfer between concrete and reinforcement is, however, warranted by friction. In the case of steel also, there is no chemical bond and the pull-out strength is greatly enhanced by proper geometry of the rebar. ‘Bonding’ can thus be greatly improved, for instance, by having ribbons on the rebars by winding part of

the fibres around the rod and by sanding. In the case of concrete where cracking occur due to tension loading, the cracks in FRP-reinforced concrete would be substantially larger than in steel reinforced concrete because of the lower Young's modulus of the FRP. This also depends on the FRP applied, as shown in Fig. 5.5, which means that the modulus of elasticity does not have to be lower.

In general, the relevant deterioration mechanisms are discussed in Section 5.4.4.2. Of special interest is the resistance of the FRP to alkalinity because the cementitious binder in concrete considerably increases alkalinity. UV resistance is, however, of no consequence for reinforcement embedded in concrete.

5.5 References

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6.1 Introduction

Engineering Structures are designed to perform over a stated period of time which means that their functions must remain intact during the design service life. Essential functions, such as structural safety, should be met with a specified reliability, not only on completion of the work but during the whole service life. This may look obvious, but in reality it is one of the most cumbersome aspects of the designing process. It is the rule rather than the exception that structures show unforeseen defects in one or more functions during the design service life. As far as the design is concerned, this is due to the lack of prompt knowledge about the performance of structures and materials, which is reflected in the standards where guidance to determine the service life of products is generally very limited. Another factor is that a long service life is beyond the horizon of designers and builders, who are often not liable for more than ten years, while many defects, such as reinforcement corrosion in concrete, manifest themselves only after this period. For these reasons the designers do not really have the incentive to attend to such matters.

Several durability problems are not due to the designer but to low-quality job execution. However, sometimes the design is less amenable to variability in the construction process. For instance, if very dense reinforcement is applied in concrete, it will be difficult to avoid segregation and honeycombing of the concrete, and if the thickness of a steel coating must fall within narrow boundaries, e.g. 300–100 μm to avoid cracking, this cannot be achieved at the edges and welds and cracking is then likely to occur regardless of the quality of the application.

Design for durability should include the structure's maintenance programme during service life. Often parts of the structure will have to be replaced or repaired to maintain performance at an acceptable level during the design service life. This also includes inspection programmes to monitor the structure's performance. For instance, if steel with an organic coating is applied to a structure that is exposed to the exterior, we know

this coating has to undergo regular maintenance during the service life of the structure.

In actuality most structures are designed without specifying the service life and the functions to be fulfilled and the structure is completed without offering the client a maintenance programme. In theory, this could mean that there is no maintenance required during the service life, but reality is different, particularly in cases where durability is not intentionally taken into account in the design process. In such cases, maintenance is a question of an *ad hoc* reaction whenever a defect manifests itself. This reaction can come too late in the case of a sudden decline in performance, e.g. in the case of stress corrosion of steel tendons. Often, the moment unexpected defects appear, maintenance activities are triggered off, including the drafting of a maintenance programme for the future.

Design for durability is a matter of preventing deterioration of performance below a specified level of reliability and there are many ways of doing this. A variety of building products is often available to perform a certain function in the structure, e.g. steel, reinforced concrete or timber beams in a bridge. Within a generic type of material or product, a variety of qualities is available, e.g. high-performance concrete or plain concrete. The choice sometimes depends on the preference of the designer or client, but is more often a matter of cost. Costs have to be considered over the whole service life; life-cycle costing is, therefore, attracting more and more attention. An optimum has to be found while still meeting the minimum values of performance requirements. The environmental impact of structures is becoming another aspect of optimization. Environmental life-cycle assessment of structures will gain importance in the near future. Sometimes the environmental loading is capitalized and is made part of the costs.

In this chapter attention is paid to the following.

- Definitions.
- Listing the threats endangering the structure and their associated effects.
- Deem-to-satisfy approach and performance-based design.
- Probabilistic approach to service life design.
- Maintenance strategies.
- Life-cycle costing.
- Environmental life-cycle assessment.

6.2 Definitions

6.2.1 Functions and performance

Function: a function is a task that a product (building material, building element or structure) has to comply with, e.g. a concrete pile has to bear the load of a bridge.

Functional requirement: a requirement related to the function that the product has to meet.

Performance: behaviour related to use.

Performance requirement: a statement of the performance required. Performance requirements include structural safety, fire safety, weather tightness, energy demand and environmental burdens.

Often several features (characteristic properties) of the product are required so as to comply with a function. For instance, to carry loads a bridge needs strength and stiffness. In this case one functional requirement is translated into two performance requirements.

Failure: A product fails if a performance requirement is no longer met.

Performance indicator: an indicator covers a specific aspect of the structure, e.g. bending of a beam.

6.2.2 Quality and reliability

Quality: Conformance to specification.¹

Reliability: The probability that a product (building material, building element or structure) will perform a required function, under stated conditions, for a stated period of time. Reliability is therefore the extension of quality into the time domain and may be paraphrased as ‘the probability of non-failure in a given period’.¹ Reliability = $[1 - \text{probability of failure } (P\{F_i\})]$, see Section 6.5.

Reliability index: limit value given in codes for the minimum reliability to be met.

6.2.3 Redundancy

Redundancy: the provision of more than one means of achieving a function.

Active redundancy: all items remain operational prior to failure. *Standby redundancy:* replicated items do not operate until needed.¹

6.2.4 Service life and durability

Service life: the period of time during which the product performs the functions for which it is designed and used. This period can be restricted due to technical reasons e.g. the product no longer meets the specified technical functional requirements (technical service life) but it can also be that the non-technical functional requirements of the product are no longer met,

e.g. economic requirements (economic service life) or that the functional requirements themselves are changed in time.

Technical service life: the period of time during which all essential technical properties meet or exceed minimum acceptable values when routinely maintained. The technical service life must be in accordance with the required service life.

Required service life: the service life specified to meet users' requirements.

Design service life: the period of use intended by the designer. This design service life should be no shorter than the required service life.

Serviceability: a product's capability to perform the functions for which it is designed and used.

Serviceability limit state (SLS): if this limit state is exceeded the functioning of the structure must be restricted. In structural codes this is, for instance, the case if the deflection of a reinforced concrete beam exceeds a certain limit, e.g. 20 mm. This principle can also be used for non-structural items, such as watertightness of a tunnel where the serviceability limit state then is a certain amount of leakage water allowed.

Ultimate limit state (ULS): if this limit state is exceeded, the structure does not function. In structural engineering this means the structure has collapsed, is fractured, etc.

Durability: the capability of a building, assembly, component, product or structure to maintain its functions over at least a specified period of time.

Explicit design for durability: a design in which each relevant deterioration mechanism is taken into account with the result that the probability of failure as a result of a decline in performance will not exceed a defined limit value within the design life.²

Implicit design for durability: a design where an exposure or environmental class given in standard specifications for the specific application is chosen with the prescriptive product requirements specific to that class. In a deem-to-satisfy approach, it is assumed that if these requirements are met, the required design service life will be achieved.

Life-cycle cost: the total discounted cost of owning, operating, maintaining, and disposing of or recycling a building or a building system over a period of time.

6.2.5 Maintenance and repair

Maintenance: refers to all activities aimed at maintaining and/or restoring a specified performance of a product (building materials, building elements

or structures). Such activities comprise not only execution of activities, but also preparation and design. Maintenance activities can be divided into corrective, preventive and inspective maintenance.

Corrective maintenance (also called *essential maintenance*): concerns repair to restore functioning of the product (building materials, building elements or structures) where the performance is unduly affected.

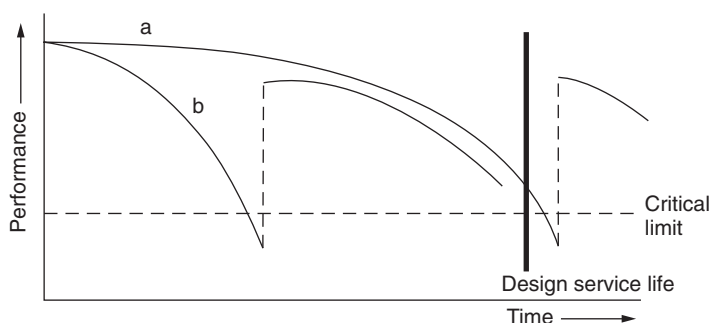
Preventive maintenance (prescribed): maintenance that is carried out periodically from the as new state, even in the absence of any sign of deterioration.

Preventive maintenance (reactive): regards preventive measures when significant loss of performance is found through condition monitoring and/or assessment, but on a level at which its performance is still not unduly affected.

Inspective maintenance: involves activities performed to assess the present performance of the product (building materials, building elements or structures). It can be regarded as part of preventive maintenance.

Repair: restoring the functioning of products (building materials, building elements or structures).

Damage: in the case of damage, the performance of products (building materials, building elements or structures) is affected, the occurrence of which was not predicted in the service life design. Figure 6.1. illustrates this. Curve a presents the predicted performance in time. Curve b is the actual performance versus the time function. The maintenance required to keep performance above the critical level has not been predicted with the result that damage has occurred. For instance, a concrete structure suffering from reinforcement corrosion is damaged. The defect was not foreseen in the design and was not expected. A steel structure that needs a new topcoat for



6.1 Performance of building material, building component or structure in time:

- (a) no corrective or preventive maintenance required, no damage
- (b) corrective maintenance was necessary, damage.

the protection system does not face damage because this was foreseen and the topcoat is part of the preventive maintenance.

Maintainability: The extent to which it is feasible to restore product performance to the intended original minimum level within a given period of time.¹

6.3 Listing of deterioration mechanisms and effects

Any systematic approach to determine the service life, such as described in the CIB-RILEM publication, *Prediction of Service Life of Building Materials and Components*,³ starts with a problem definition of performance requirements and performance criteria. Subsequently, there is the identification of threats, degradation mechanisms and the associated effects of the specific environment to which the structure is exposed. For a specific material, environmental classes are distinguished (see EN 206 'Concrete-Part 1' for concrete as given in Table 6.1, and, for timber, Table 4.1, Chapter 4). A list of degradation factors is given in Table 6.2, taken partly from Reference

Table 6.1 Exposure classes from EN 206 – 1, *Concrete, Specification, Performance and Conformity*

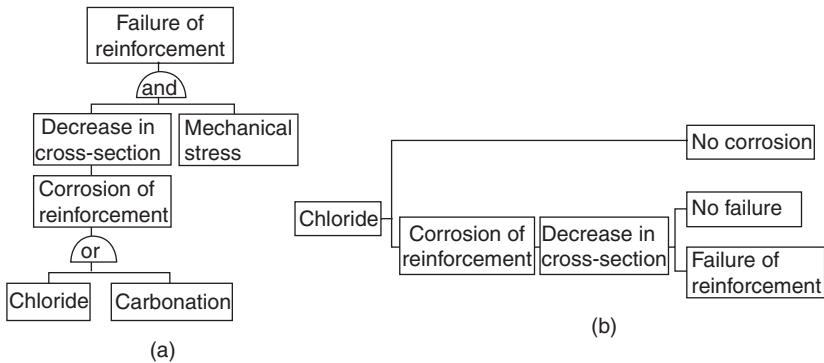
Class	Characteristics
1 XO	No risk of corrosion or attack (inside buildings with very low air humidity)
2 XC	Corrosion induced by carbonation
XC1	Dry or permanently wet
XC2	Wet, rarely dry
XC3	Moderate humidity
XC4	Cyclic wet and dry
3 XD	Corrosion induced by chloride other than from sea water
XD1	Moderate humidity
XD2	Wet, rarely dry
XD3	Cyclic wet and dry
4 XS	Corrosion induced by chlorides from sea water
XS1	Exposed to airborne salt but no direct contact with sea water
XS2	Permanently submerged
XS3	Tidal, splash and spray zone
5 XF	Freeze/thaw attack with or without de-icing agents
XF1	Moderate water saturation, without de-icing agent
XF2	Moderate water saturation, with de-icing agent
XF3	High water saturation, without de-icing agent
XF4	High water saturation, with de-icing agent
6 XA	Chemical attack
XA1	Slightly aggressive
XA2	Moderately aggressive
XA3	Highly aggressive

Table 6.2 Degradation factors affecting the service life of building materials and components³

Type of factor	Example
Weathering factors	Radiation
	Solar
	Nuclear
	Thermal
	Temperature
	Elevated
	Depressed
	Cycles
	Water
	Solid (such as snow, ice)
	Liquid (such as rain, condensation, standing or running water)
	Vapour (such as high relative humidity)
	Normal air constituents
	Oxygen and ozone
	Carbon dioxide
	Air contaminants
	Gases (such as oxides of nitrogen and of sulphur)
	Mists (such as aerosols, salts, acids, and alkalis dissolved in water)
	Particulates (such as sand, dust, dirt)
	Freeze–thaw
	Freeze–thaw de-icing salts
Biological factors	Wind
	Fungi
	Insects
	Marine borers
Stress factors	Bacteria
	Stress, sustained
	Stress, periodic
	Physical action of water (as rain, hail, sleet, and snow)
	Physical action of wind
	Combination of physical action of water and wind
Incompatibility factors	Imposed deformations such as settlements, temperature or shrinkage deformations
	Chemical
Use factors	Physical
	Design of system
	Installation and maintenance procedures
	Normal wear and tear
	Abuse by the user

Table 6.3 Example of a Failure Mode and Effect Analysis (FMEA) with respect to the durability of concrete

Hazard	Mechanism	Effect
Alternating load	Fatigue	Cracking, failure
Flowing water	Erosion	Surface deterioration
Frost	Expansion	Cracking
Carbon dioxide	Carbonation	Corrosion of reinforcement
Chloride ions	Depassivation	Pitting corrosion

**6.2** Fault tree (a) and event tree (b) relating to the corrosion mechanism.

3. In a 'Failure Mode and Effect Analysis' (FMEA), the threats, mechanisms and effects are coupled. An example is given in Table 6.3.

Only those threats that may have a negative influence on performance during the design service life require further consideration. Mechanisms of greater complexity, where several threats play a part and/or the effect of one mechanism constitutes a threat to another, can be represented with the aid of fault trees and event trees, as shown in Fig. 6.2. After selection of the relevant degradation mechanisms and effects on the structure or the structural component, the effects must be quantified.

6.4 Approaches to design for service life

6.4.1 Implicit and explicit design for durability

With respect to service life, most specifications refer to such standards as those that prescribe specific raw materials, metal alloys, generic types of coating, coating thickness, concrete composition, cover thickness, curing

conditions, allowable crack width and type of wood in specific environments. It is assumed that meeting these specifications will ensure a long service life.

Often, service life length is not specified. If the service life is specified or imposed by building regulations (e.g. reliability indices for structural safety safeguarded over 50 years), it is then implicitly assumed in a deem-to-satisfy approach that if these prescriptive requirements are met while in use, the specified design service life will be achieved. In the case of a long term proven record in similar structures with similar materials and in similar environments, this can be an effective approach. Initial testing and either deterministic or probabilistic calculations of service life are then not required. If no such similarity exists, this approach can easily lead to failure.

There are a number of reasons why there is a tendency to design explicitly for service life by means of performance-based design instead of implicitly by means of prescriptive specifications. These reasons are:

- Often the product standards do not sufficiently warrant the durability required to meet the service life requirements, e.g. applying the European Standard EN 206 to concrete is no guarantee that a reinforced concrete structure in a marine environment will have, for instance, a 75 year service life without reinforcement corrosion.
- Competition between different kinds of materials on the basis of durability cannot be done objectively with the present deem-to-satisfy approaches, as these are not based on the same starting points.
- Adoption of western standard prescriptive specifications for harsher climates in, for instance, the Gulf and other coastal areas with a hot climate, has led to major economic damage.
- The variety of materials available for structural applications and their protection systems has grown rapidly over the last decade. As a result of environmental pressure, new protective coating systems have been developed for metals. In concrete, additions, admixtures, and secondary aggregates, such as concrete granulates and new compositions such as self-levelling concrete have properties that are insufficiently covered or not covered at all by the old prescriptive requirements. Modified woods are being developed due to a ban on preservatives for wood. In all these cases it is not self-evident that the old prescriptive standards still apply.
- The market favours tailor-made materials and products; prescriptive requirements can be a barrier to such developments.
- Responsibility for product performance is increasingly shifted to the manufacturer who can no longer hide behind standard prescriptive specifications.

6.4.2 Performance-based design

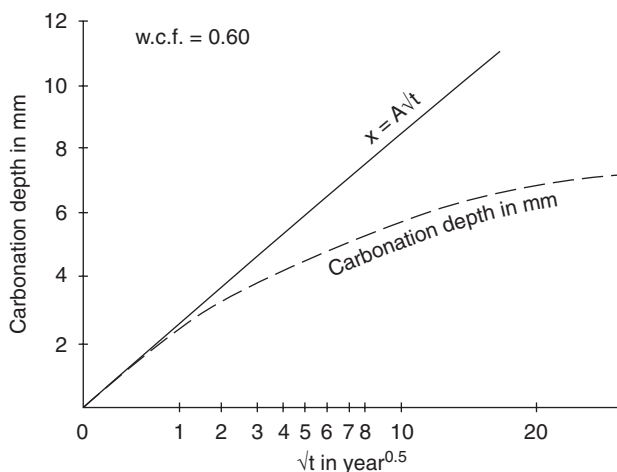
6.4.2.1 *Practice, test or model*

Performance-based design for durability can be based on:

- 1 Monitoring the performance of structures or testing structures under in-use conditions.
- 2 Determination of the resistance in a standardized artificial environment which accelerates the deterioration process, and which is selected after consideration of the relevant degradation mechanisms, e.g. a Weather-O-meter test for steel coating systems, frost–thaw de-icing salt tests for concrete or for wood, ENV 12038, a test method for determining the resistance against wood-destroying Basidiomycetes (fungi, see Section 4.2.1). This can also concern a phase in the deterioration process, e.g. an accelerated carbonation test for concrete, where only the initiation phase of the corrosion process is considered.
- 3 A performance indicator that is different from the actual one, which is regarded as representative for the resistance against a threat, e.g. compressive strength as an indicator of concrete durability.
- 4 A model that describes performance over time, as previously discussed. Input parameters in the model are determined in prescribed tests.
- 5 Combination of two or more of the above approaches.

Further comments can be made on the methods listed above.

- Re 1: Method, monitoring the performance of the structure when in use over time offers the most reliable results. However, it is not feasible for application to new products. Another problem is that the history of the structures is often not or poorly documented. Nevertheless, it is the good performance of steel and concrete structures in use that have resulted in market confidence in the reliability of these materials.
Systematic monitoring of the performance of test structures and of materials under practical conditions is done on exposure farms.
- Re 2: In accelerated deterioration testing, the key feature is to ensure that the accelerated or short-term tests bear a safe and economic relation to the structure's long-term performance.
Especially in new materials and new protection systems, it appears that existing accelerated test methods often do not relate well to practice. For instance, frost–thaw de-icing tests such as CDF, SS 137244 and ASTM C672 methods are well suited to predicting the resistance of concrete with dense aggregates, e.g. river gravel, yet when concrete granulate (crushed concrete coarse aggregate) is applied, concrete fails in the frost–thaw de-icing salt test, while in practice it does not.⁴
The frequently applied salt-spray test (ASTM B 117) for metal and



6.3 Carbonation depth as a function of $\sqrt{t}$.

coated metals appears to show a bad correlation with actual behaviour in practice.

In accelerated carbonation tests for concrete, one phase of the deterioration process is tested. Concrete is subjected to an environment with an enhanced CO_2 content and constant temperature and humidity. This results in a too pessimistic view of the carbonation process, as shown in Fig. 6.3. In the accelerated tests, a linear relationship between carbonation depth and the root of time is found in accordance with Fick's second law of diffusion. However, in practice carbonation moves asymptotically to a limit value. This is due to the fact that in conditions of outside exposure, only a zone to a certain depth from the surface is subject to fluctuating moisture conditions. Beyond that zone, moisture conditions appear to be fairly constant.

More importantly, in covers exceeding 20 mm and subject to Western European conditions, there will be no significant corrosion, even if the carbonation front advances beyond the reinforcement corrosion (see Section 3.4.2). The application of accelerated carbonation tests is, therefore, likely to result in a much too conservative assessment of the probability of failure due to carbonation-induced corrosion.

- Re 3: If another product property is used as an indicator for durability, it must be realized that such a relation is only valid within strict boundaries. For instance, a higher compressive strength generally means a better resistance in concrete when using the same raw materials. Yet between concretes of the same strength, but made with different raw materials, there can be a very large difference in resistance against

degradation mechanisms. Strength does not appear to be a dominant factor for durability!

- Re 4: An example of a test that determines durability model parameters is discussed in Section 6.5.5.2. The input parameters can be the chloride diffusion coefficient and the cover thickness but a number of constants must also be determined at some stage. The key question here is how well the model relates to actual chloride penetration when in use. The model does not take direct account of the convection of chloride into concrete due to water absorption, which, in Western Europe, is the most important penetration mechanism in the first 15 mm from the surface. Where the service lifetime is calculated by means of a model, there is always the question of how representative the model is in practice.
- Re 5: A combination of methods in most cases offers the best approach (see e.g. the next section).

6.4.2.2 *Practical case: protection of steel structures*

There is serious cracking of newly developed high solids epoxy-coatings on sea defence structures along the Dutch coast; this is discussed in Sections 2.5.3.1 and 2.5.3.3.⁵ The specifications for these projects were mainly prescriptive and copied from previously applied epoxy-tar coatings, yet appeared to be unsuited to the new more environmentally-friendly coating systems applied. This prompted the Dutch government to change its specification policy. A performance-based method was drafted consisting of four steps:

- 1 The first step is problem definition. It describes the structure in relation to the protection to be applied. Threats, degradation mechanisms and the associated effects are identified while taking into account the specific environment to which the structure is exposed.
- 2 The functional requirements are defined and translated into performance requirements.
- 3 Actual data gathering and testing of protection systems is carried out.
- 4 The data gathered is analysed in relation to the performance requirements and conclusions about conformance are reached.

In step 2, a distinction is made into three classes of performance requirements:

- Proven good performance in practice.
The protective system should have at least five years of proven experience in similar applications. Criteria with regard to frequency and size of defects, such as blistering, cracking etc, are given.

- Accelerated degradation tests
Currently used accelerated weathering tests are:
 - TNO corrosion cabinet;
 - Weather-O-Meter (WOM) test;
 - salt water spray test ASTM B 117;
 - fresh water immersion test ISO 2812-2;
 - salt water immersion test ISO 2812-1.
- Modelling of cracking behaviour

A preliminary model is as follows:

$$1/(1-\nu) \cdot E_{\text{coating}} ([\alpha_{\text{coating}} - \alpha_{\text{steel}}] \cdot \Delta T + C \cdot \epsilon_{\text{shrinkage}}) \leq \sigma_{\text{tensile strength}} \quad [6.1]$$

where

ν	= Poisson coefficient of the coating
E_{coating}	= mean E-modulus of the coating over the usage temperature interval
C	= mean relaxation coefficient over the usage temperature interval
α_{coating}	= mean temperature expansion coefficient of coating system over the usage temperature interval
α_{steel}	= temperature expansion coefficient of steel
ΔT	= temperature change
$\epsilon_{\text{shrinkage}}$	= shrinkage of coating system
$\sigma_{\text{tensile strength}}$	= mean tensile strength of the coating over the usage temperature interval

If there has been no practical experience over a minimum period of five years and there are no alternatives, conformance to specifications has to be based on accelerated deterioration tests and modelling. In such cases, the opinion of the expert must weigh heavily in the decision-making process. In generic new protection systems, the above performance criteria for accelerated weathering and modelling must be considered thoroughly with regard to their representativeness in practice.

6.4.2.3 Responsibility

Performance requirements with respect to durability at the structural level are mostly general in character. For instance, if a reinforced concrete bridge is to have a maintenance-free service life of 75 years, such a requirement means that the designer and the contractor have the responsibility to design and construct a concrete structure with a service life of 75 years. For instance, the contractor of the King Fahad Causeway between Saudi Arabia and Bahrain was asked to show that the durability of the concrete structure was adequate for a service life of 75 years when routinely maintained.

This prompted the contractor to conduct an in-depth investigation into durability in this harsh environment. As a result, the contractor proposed major changes to the design, most of which were accepted by the client.⁶

If performance requirements by the designer are set at the material level, e.g. strength of concrete must be grade 40 or the diffusivity of chloride ions must be $10^{-12} \text{ m}^2/\text{s}$, then the designer is responsible for ensuring this specification warrants a maintenance-free service life of 75 years. The contractor does not need to consider the 75 years but must only ensure that the requirements at the material level are met. The legal possibility to claim indemnity from either designer or contractor differs greatly between countries.

6.4.2.4 *Deterministic and probabilistic service life predictions*

Service life predictions can be carried out in a deterministic or probabilistic way. Deterministic service life predictions take into account mean values. The probabilistic approach takes mean values and uncertainties into consideration and calculates the probability of failure or reliability. This is discussed in the next section. Sometimes it will be clear that a design is highly reliable. The structure is, for example, protected against an aggressive environment by such items as tanking, membranes and coatings or corrosion is inhibited by cathodic protection. In such cases, reliability calculations can be omitted.

6.5 Probabilistic approach to service life design

6.5.1 Analysis

In a probabilistic approach to service life design, the probability of failure and the extent of damage in case of failure must be assessed. The combination of probability and damage, expressed as the mathematical product of the two, is called the risk. A condition for structural design is that the probability of failure of a function will be lower than a limit value within the design service life. Limit values for the probability of failure in standards for structural safety are very low. Table 6.4 gives an example of limit values for the probability of failure of buildings subject to wind loading according to Dutch standards NEN 6700 and 6702. For other functions where there is no direct danger of loss of human life or the structure can remain operational while being repaired, e.g. rain tightness of building envelopes, limit values can be much higher.

The durability design guide follows the same principles (reliability and performances) as a structural design code.⁷ This means that durability design will be based upon:

- Realistic and sufficiently accurate definitions of environmental actions (different micro-environmental aggressiveness classes) depending on the ensuing type of degradation.

Table 6.4 Limit values for probability of failure during reference time, compared with design service life in the case of predominant wind loading (according to Dutch standards 6700 and 6702)

Safety class	1	2	3
R = 15 years	$6.9 \cdot 10^{-4}$	$3.4 \cdot 10^{-4}$	
R = 50 years		$3.4 \cdot 10^{-4}$	$1.6 \cdot 10^{-4}$
R = 100 years			$1.6 \cdot 10^{-4}$

- Material parameters.
- Mathematical models for degradation processes.
- Performances expressed as limit states.
- Reliability.

6.5.2 Structural design

In modern codes, such as the Eurocode, the limit state function expresses the basis of the conventional design procedure for structural safety and serviceability. This limit state defines the boundary between an adverse state (such as collapse, buckling, deflection, vibration) and the desired state. The limit state can, in principle, be formulated as:

$$R - S = R(X_1, X_2, \dots, X_n) - S(X_{n+1}, X_{n+2}, \dots, X_m) = 0 \quad [6.2]$$

where:

R = a function that describes the load bearing capacity of the structure

S = a function that describes the influence of the load on the structure

X_i = a basic variable for the functions R or S

The structural design procedure is elaborated in such a way that the probability of failure is restricted:

$$P\{\text{failure}\} = P_f = P\{R - S < 0\} < P_{\text{target}} = \Phi(-\beta) \quad [6.3]$$

where:

$P\{\text{failure}\}$ or P_f = the probability of failure of the structure

P_{target} = the accepted maximum value of the probability of failure

Φ = standard normal distribution function (mean = 0 and standard deviation = 1)

β = reliability index (parameter normally used instead of the probability of failure)

This probability of failure can be calculated with the aid of probabilistic techniques. In practice, however, the design has been simplified to a semi-probabilistic level with characteristic values and partial factors γ calibrated in such a way that the target reliability will be achieved:

$$R_c / \gamma_R - S_c \cdot \gamma_S = R_d - S_d > 0 \quad [6.4]$$

where

R_c = load-bearing capacity of the structure based on characteristic values

γ_R = material factor

S_c = characteristic value of the influence of loading

γ_S = load factor.

R_d = design value of the load-bearing capacity

S_d = design value of the load

Calculation example for structural design

A relatively simple calculation example of such a limit state function can be derived from Fig. 6.4. The beam has two supports and a rectangular cross-section. A concentrated load F loads the beam in the middle section. The span is l , the width b and the height h .

The maximum bending moment M_{middle} occurs in the middle section:

$$S = M_{middle} = F/2 \times l/2 = 1/4 Fl \quad [6.5]$$

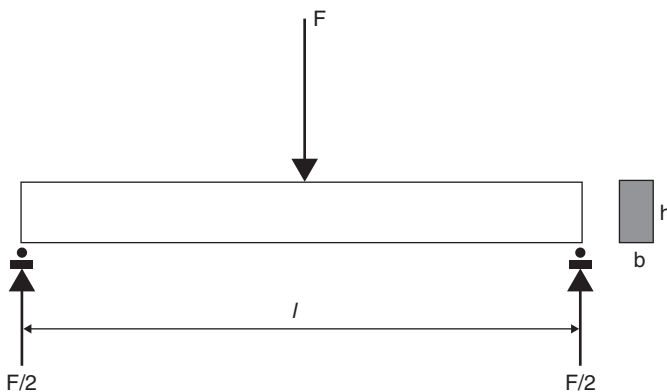
in case of linear elastic material behaviour, capacity M_{max} of the middle section is:

$$R = M_{max} = Wf = 1/6 bh^2 f \quad [6.6]$$

where:

W = the section modulus

f = the material strength (in this example either tensile or compressive strength)



6.4 Beam on two supports.

Table 6.5 Parameters in the calculation example

Basis variable	Description	Distribution	Mean	Standard deviation
b	Beam width	Deterministic	150 mm	–
h	Beam height	Deterministic	400 mm	–
F	Concentrated load	Normal	250 kN	75 kN
l	Span	Deterministic	5000 mm	–
f	Material strength	Normal	200 N/mm ²	20 N/mm ²

Equilibrium is possible as long as relationship [6.7] applies:

$$R - S > 0 \quad \text{or} \quad 1/6bh^2f - 1/4Fl > 0 \quad [6.7]$$

An overview of the parameters, including their stochastic characteristics, is given by way of example in Table 6.5. On the basis of this information, a reliability index $\beta = 4.0$ can be calculated. For an ultimate limit state, Eurocode 1 has defined a reliability index of at least $\beta = 3.8$. The conclusion therefore is that this design is safe.

6.5.3 Time-dependent design

In performance-based structural design, both resistance R and load S are considered to be time independent. In many situations this is not realistic. The load can be time dependent or the capacity can change in time due to degradation. Relationship [6.2] should then be rewritten as a time-dependent limit state function⁸ taking such effects into consideration:

$$R(t) - S(t) > 0 \quad [6.8]$$

A special case for this limit state function occurs if either R or S is not time dependent. These relationships do, in principle, not differ from those in [6.2]. Relationship [6.8] applies to all values of t in the time interval $(0, T)$. T is the intended service period (i.e. reference period). From a mathematical point of view, it can be stated that relationship [6.8] can be used for durability design. The service life concept can be expressed in a design formula, similar to [6.2]:

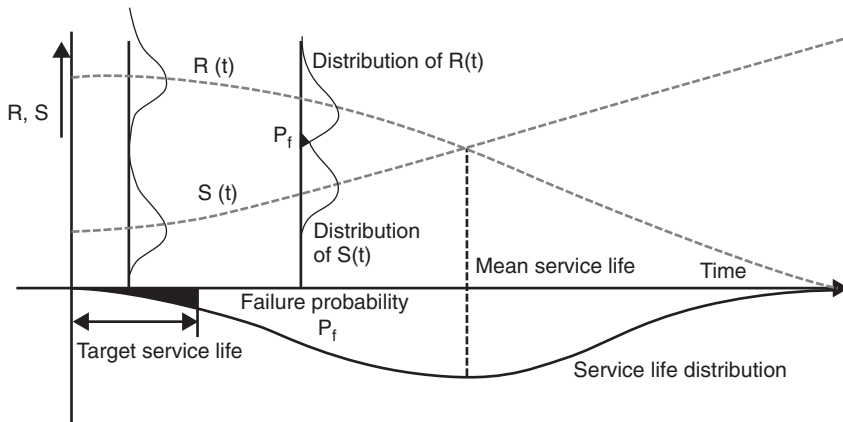
$$P_f(T) = P\{R - S < 0\}_T < P_{\text{target}} = \phi(-\beta) \quad [6.9]$$

where:

$P_f(T)$ = the probability of failure of the structure within T

T = intended service period.

It will probably be possible to simplify relationship [6.9] at a later stage to one similar to that used in the conventional design procedure [6.4].



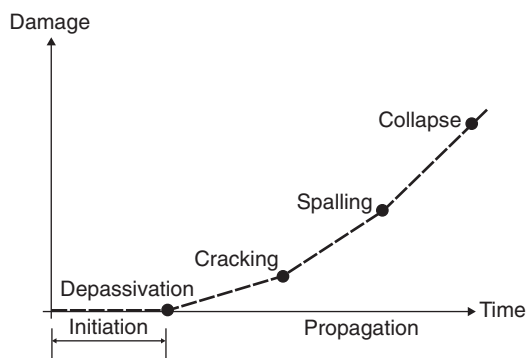
6.5 Failure probability and target service life (illustrative presentation).

The mathematical model for describing the event ‘failure’, i.e. passing a durability limit state, comprises a load variable S and a resistance variable R (see Fig. 6.5). Failure occurs if the resistance is smaller than the load. The probability of failure within the period of time $[0; T]$ $P_f(T)$ is defined as the probability that the load does not exceed the resistance within the given period T .

$$P_f(T) = 1 - P\{R(t) > S(t) \quad \forall t \in [0; T]\} \quad [6.10]$$

The first step of this design will be the definition of the desired/required performance(s) of the structure. The client or the owner of the structure is asked to define their requirements for quality and target service life. Further requirements can be given in building codes. The definition of performance criteria will be related to a limit state criterion.

Figure 6.6. shows an example of the performance (damage function) of a concrete structure with respect to reinforcement corrosion and related limit states. This example is taken from Reference 9, where a description is given of the Western Scheldt Tunnel in the Netherlands (see also Section 6.5.5.2). The tunnel lining is loaded by salt (sea) water from the outside and salt leakage water and de-icing salts from the inside. Due to the ingress of chloride ions, depassivation of the reinforcement will occur and corrosion can be initiated. If enough corrosion products have been formed, the concrete cover will crack. Finally, spalling of the concrete cover will occur. In the last stage, the reduction of the cross-section of the reinforcing bars involved will end in the collapse of the structure. Depassivation and cracking in principle represent serviceability limit states related to durability.



6.6 Determination of service life and limit states with respect to reinforcement corrosion (after Reference 9).

Collapse represents an ultimate limit state, whereas spalling can involve both serviceability and safety related to durability.

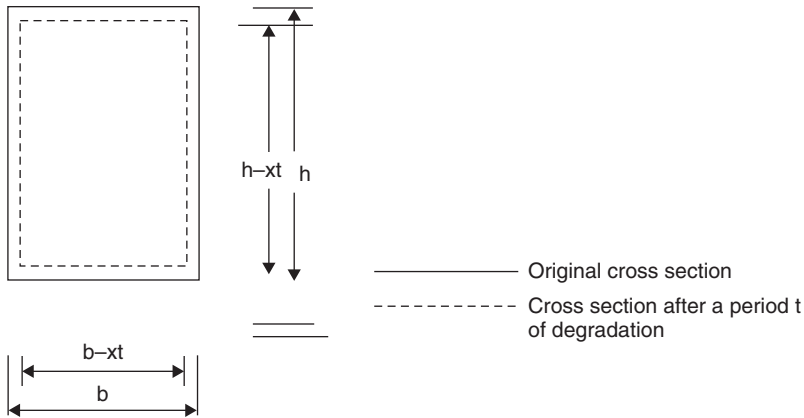
Calculation example including a degradation model

The second step of the durability design is to analyse the environmental actions and to identify the relevant degradation mechanisms. Mathematical models describing the time-dependent degradation processes and the resistance of the material to it are needed. The big step forward in performance-related durability design is that these models enable the designer to evaluate the time-related changes in materials and structures, depending on specific material and environmental conditions. In this calculation example, we will simplify the approach by assuming only degradation of the material.

Further to the example given in Fig. 6.4, we assume that the dimensions of the cross-section are reduced by x mm per year (see Fig. 6.7). This means that after a period t , the width is reduced to $(b - xt)$ and the height to $(h - xt)$. The original relationship [6.7] changes due to this degradation to:

$$R(t) - S > 0 \quad \text{or} \quad 1/6(b - xt)(h - xt)^2 f - 1/4 Fl > 0 \quad [6.11]$$

In Table 6.6 a new set of parameters is defined, based on the parameters listed in Table 6.5 and including the new parameter x . A probabilistic calculation based on these parameters results in the reliability indices given in Fig. 6.8. For $t = 0$ we again come up with the original result $\beta = 4.0$. This value decreases in the course of time. After about 3.3 years, the value $\beta = 3.8$ is exceeded. After about 65 years, the remaining reliability $\beta = 0.0$; meaning a 50% probability of failure.



6.7 Calculation example with a degraded cross-section.

Table 6.6 Parameters of the calculation example, including the rate of degradation

Basis variable	Description	Distribution	Mean	Standard deviation
b	Beam width	Deterministic	150 mm	–
h	Beam height	Deterministic	400 mm	–
F	Concentrated load	Normal	250 kN	75 kN
l	Span	Deterministic	5000 mm	–
f	Material strength	Normal	200 N/mm ²	20 N/mm ²
x	Rate of degradation	Lognormal	1 mm/year	0.05 mm/year

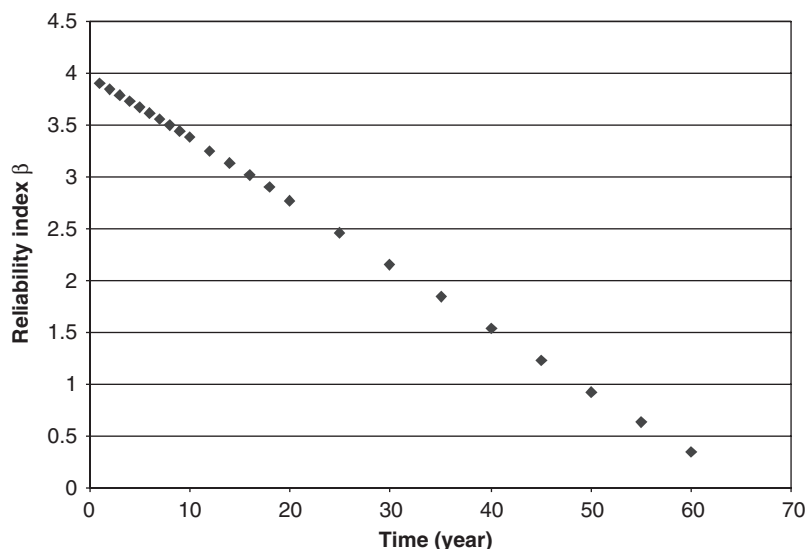
6.5.4 Case 1: Durability of a gallery slab

6.5.4.1 Introduction

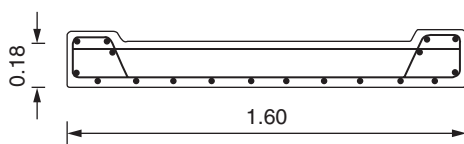
A reinforced concrete outdoor gallery slab will be investigated so as to illustrate optimization analysis in relation to durability. The slab is shown in Fig. 6.9. In order to keep the problem within conveniently manageable limits, only the threat due to carbon dioxide will be considered. The mechanism concerned is the carbonation of free lime and the effect is corrosion of the main reinforcement at the underside, manifesting itself in spalling of the concrete cover.

Four design situations are to be distinguished:

- 1 Cover of 15 mm, no coating.
- 2 Cover of 30 mm, no coating.



6.8 Reliability indices resulting from probabilistic calculation.



6.9 Data for the gallery slab of Case 1.

- 3 Cover of 15 mm, coating, maintenance every 20 years.
- 4 Cover of 15 mm, coating, maintenance every 10 years.

In principle, both the cover and the maintenance period are continuous variables. A much greater number of alternatives would therefore have to be investigated. However, for a preliminary examination of the problem, analysis of the four above-mentioned choices will suffice. As soon as the deteriorative damage visibly manifests itself, remedial action will have to be taken. The following possibilities can be considered:

- Complete replacement.
- Removal of the entire carbonated zone and application of sprayed concrete.
- Local repair of the visible damage with polymer mortar.

Which of these methods will be optimal in any particular case will depend on time when repairs are effected in relation to the lifetime of the

structure. Radical repair work is meaningful only if the structure still has a sufficiently long unexpired service life to complete. The planned service life for the building as a whole is 60 years.

6.5.4.2 Mathematical models for carbonation

If concrete is not provided with a protective coating, the depth of carbonation (d in mm) can be calculated with the aid of the following equation:

$$d = \left(\frac{46w - 17.6}{2.7} \right) RK \sqrt{t} \quad [6.12]$$

For the case under present consideration (see Reference 10) we have:

$$w = 0.50$$

$$R = 2.0$$

$$K = 0.7$$

If the carbonation depth is equal to the concrete cover c , corrosion of the reinforcement can start. The depth of carbonation is not the same at every point of the slab, however. Equation [6.12] gives the average depth over the slab (see Fig. 6.10). Corrosion initiation is governed not by the average depth of carbonation, but by the advanced peaks of the carbonation front. According to Reference 10, this can make a difference (Δ) of between 5 and 10 mm. On the other hand, not every peak encounters a reinforcing bar, and it moreover takes some time for the corrosion to manifest itself in an externally visible manner (spalling of the concrete cover). For this the following formula is given.¹¹

$$t_c = \frac{0.08c}{\phi v_c} \quad [6.13]$$

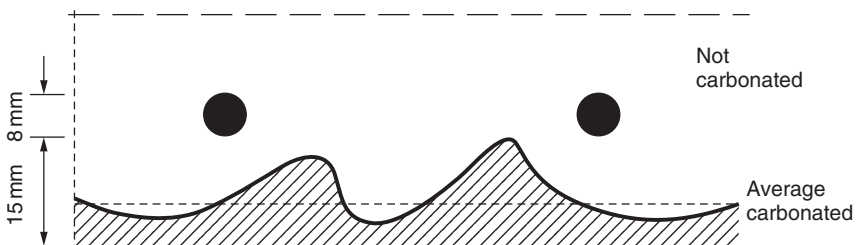
where

t_c = time for corrosion to manifest

c = the cover

ϕ = the bar diameter

v_c = the rate of corrosion



6.10 Irregular carbonation front in concrete.

In general, the time that elapses before corrosion damage becomes visible can be calculated from the following formula, with replacement of d in equation [6.12] by $(d - \Delta)$ or $(c - \Delta)$, because c is the cover thickness:

$$t_L = \frac{(c - \Delta)^2}{R^2 K^2} \left(\frac{2.7}{46w - 17.6} \right)^2 + \frac{0.08c}{\phi v_c} \quad [6.14]$$

The influence of fluctuations in carbonation depth is taken into account by means of Δ for which an average value of about 5 mm can be adopted. The corrosion rate v_c is affected by considerable scatter. In the literature (Tuutti¹² and Gonzales et al.¹³), values ranging from 0.015 to 0.09 mm/year are found. The constant is 0.08 mm. Its purpose is to make equation [6.14] non-dimensional.

The question as to which is the appropriate model to be adopted for describing the carbonation process in concrete is still a subject of much discussion. By presenting equation [6.14], it is not the intention of the present report to take a position in this discussion. The only purpose is to show that reliability analysis can be used for solving durability problems. The formula in question has, in this context, been chosen more or less arbitrarily.

In the slab considered in this example, the nominal cover is 15 mm. The actual cover can be assumed to be somewhat greater: let it be 20 mm. The reinforcement consists of 8 mm diameter bars ($\phi = 8$ mm) spaced at 150 mm from centre to centre. If the previously stated values are adopted for w , R , K and E and if A and v_c are taken as 5 mm and 0.04 mm/year, respectively, the following result is obtained for the service life:

$$t_L = \frac{(20 - 5)^2}{2.0^2 \times 0.7^2} \left(\frac{2.7}{46 \times 0.5 - 17.6} \right)^2 + \frac{0.08 \times 20}{8 \times 0.04} = 29 + 5 = 34 \text{ years}$$

The main contribution comes from the first term, the so-called initiation time. If, in accordance with the second design alternative, the cover is taken as 30 mm nominally (35 mm in practice), then:

$$t_L = 115 + 7 = 122 \text{ years}$$

In that case, there is a marked increase in durability.

If the underside of the concrete slab is provided with a coating that is highly impermeable to carbon dioxide (chlorinated rubber or epoxy paint), the time period up to initiation of reinforcement attack can be considerably increased. It can be derived from the formulae given in Reference 10 that this increase is expressed by:

$$\Delta t_L = \frac{(d - \Delta)s}{180f} \quad [6.15]$$

where s is the thickness of the coating and f is the fraction of the surface not covered by the coating. As an example, an epoxy paint with $s = 0.18 \text{ mm}$ and $f = 10^{-5}$ is considered. Then:

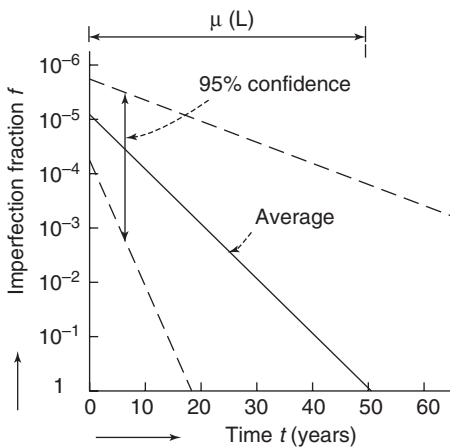
$$\Delta t_L = \frac{(20-5)0.18}{180 \times 10^{-5}} = 1500 \text{ years} \quad [6.16]$$

The protective action of the coating is thus clearly demonstrated. This does presuppose, however, that the values of s and f do not vary in the course of time and also that the coating remains intact. This (ideal) situation can be approximated if frequent inspection and maintenance are carried out.

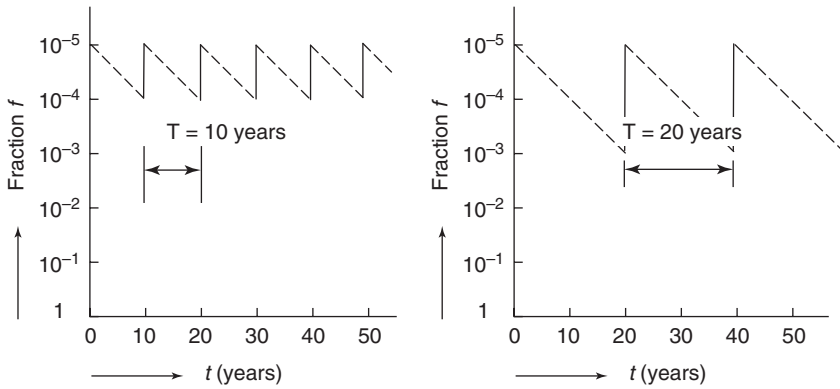
As regards the thickness s of the coating, a rate of surface disintegration of $3 \mu\text{m}/\text{year}$ is mentioned by de Sitter¹⁰ based on the work of Klopfer.¹⁴ For a normal maintenance cycle of 8–12 years, this rate is virtually negligible i.e. the thickness of the coating would decrease for example from 0.18 mm to 0.15 mm in 10 years. Only when longer intervals between maintenance operations are considered does this aspect become important.

Of greater significance is the time-dependent behaviour of f , at least if phenomena such as damage and scaling-off are taken into account. There are, however, at present no data available on this. Meanwhile, pending further information on the subject, the behaviour of f as a function of time will be assumed to be as presented in Fig. 6.11. Based on this assumption, the logarithm of f increases linearly with time and the coating will have entirely disappeared after about 50 years. This is expressed by the formula:

$$f(t) = f_0 e^{-at} \quad [6.17]$$



6.11 Assumed behaviour of the fraction f as a function of time t if no maintenance is carried out.



6.12 Behaviour of f as a function of time t for maintenance every 10 and 20 years respectively (saw-tooth model).

If maintenance is carried out periodically, $f(t)$ will show the familiar saw-tooth behaviour as presented in Fig. 6.12.

When calculating the corresponding service life, it will be assumed that the contribution made by the concrete itself is negligible in comparison with the extra time gained by the coating. Since $f(t)$ is time dependent, it is not possible to make direct use of equation [6.17]. Some differential calculus will have to be applied. During a very short period dt , the carbonation front will advance a very short distance dy . In analogy with equation [6.15], we then have:

$$dt = \frac{sdy}{180f(t)}$$

or:

$$f(t)dt = \frac{sdy}{180}$$

Integration of both sides of the equation, from carbonation initiation ($t = 0, y = 0$) to the point in time it reaches the reinforcement ($t = t_L, y = d - \Delta$), yields:

$$\int_0^{t_L} f(t)dt = \int_0^{d-\Delta} \frac{sdy}{180} \quad [6.18]$$

The right-hand integral is equal to $s(d - \Delta)/180$. For the integral on the left it is necessary to take account of the logarithmic saw-tooth pattern displayed by $f(t)$ according to Fig. 6.12. If the maintenance period is T , the number of maintenance cycles during service life is t_L/T , so that the following approximation can be made for this integral:

$$\int_0^{t_L} f(t)dt = \frac{t_L}{T} \int_0^T f_0 e^{-\alpha t} dt = \frac{t_L}{T} \frac{f_0}{\alpha} (1 - e^{-\alpha T}) \quad [6.19]$$

By equating the two expressions [6.18] and [6.19], we obtain the following formula for the service life:

$$t_L = \frac{(d - \Delta)s}{180 f_0} \left(\frac{\alpha T}{1 - e^{-\alpha T}} \right) \quad [6.20]$$

For $T = 10$ years: $\alpha T = (T/T_0) \ln f_0 = (10/50) (-11.15) = -2.3$, whence we obtain:

$$t_L = \frac{(d - \Delta)s}{180 f_0} 0.26$$

The earlier estimate of 1500 years for the service life is thus reduced to:

$$t_L = \frac{(20 - 5) \times 0.18}{180 \times 10^{-5}} 0.26 = 1500 \times 0.26 = 390 \text{ years}$$

For $T = 20$ years: $\alpha T = (T/T_0) \ln f_0 = (20/50) \times (-11.5) = -4.6$

$$t_L = \frac{(20 - 5) \times 0.18}{180 \times 10^{-5}} = 1500 \times 0.047 = 70 \text{ years}$$

For this 20 year maintenance period, the resistance that the concrete itself offers to the advance of carbonation, previously neglected, would certainly appear worth taking into account. The carbonation times are allowed to be added up, thus:

for $T = 10$ years: $t_L = 390 + 40 = 40$ years

for $T = 20$ years: $t_L = 70 + 40 = 110$ years

6.5.4.3 Quantification of uncertainties

In probabilistic calculations, a variable is described with the aid of a probability density function. Such a function is in many cases characterized by a particular type of distribution (e.g. a normal distribution or a Weibull distribution), a mean value μ and a standard deviation σ . Instead of the standard deviation, the coefficient of variation ($V = \sigma/\mu$) is often used. In the ideal case, there are statistical data available for all the variables, enabling the type of distribution, the mean value and the standard deviation to be unambiguously determined. In most cases, however, as in this example, the opposite applies, namely, there are no or, at best, scanty data available. The statistical properties will then have to be estimated. It is possible for many variables, based on information published in the literature, on experience or on intuition, to specify values within which the variable in question is likely to fall. It is well known that, for the normal distribution, there is a

95% probability that a variable will have a value ranging $\mu - 2\sigma$ to $\mu + 2\sigma$. It is thus possible to estimate the mean value and the standard deviation:

$$\mu(x) = \frac{1}{2}(x_{high} + x_{low}) \quad [6.21]$$

$$\sigma(x) = \frac{1}{4}(x_{high} - x_{low}) \quad [6.22]$$

In a log-normal distribution, the corresponding formulae, if $x_{high}/x_{low} < 10$, are:

$$\mu(x) = \sqrt{x_{high} - x_{low}} \quad [6.23]$$

$$V(x) = \frac{1}{4} \ln \left(\frac{x_{high}}{x_{low}} \right) \quad [6.24]$$

The choice between using the normal or the log-normal distribution will depend on the physical nature of the stochastic variable. Many variables are by nature unable to take on negative values, and in such cases the log-normal distribution is to be preferred. For small values of the coefficient of variation ($V < 0.10$), the difference between normal and log-normal is negligible for practical purposes. Where it is necessary to decide between normal or log-normal, on the one hand, and other potentially available distributions (e.g., extreme value distributions), on the other, the present example hardly seems to provide relevant arguments. The consequences of choosing a particular type of distribution for the stochastic variables will be considered later.

In Table 6.7, a choice has been made for the type of distribution, the mean value and the standard deviation for the various problem variables. Some of these variables have been taken as deterministic because the scatter affecting them is considered to be very small. The mean values adopted are those values rated as providing the best estimates in the preceding section. The coefficients of variation are largely based on estimates of the type indicated above. Only in the case of the concrete cover is it possible to use statistical information obtained from measurements of cover depths in existing structures.¹¹

The least scatter is presumed to occur in the water/cement ratio. The coefficient of variation $V(w) = 0.05$ indicates with a 95% probability that the value of w is between 0.45 and 0.55. The scatter in, for example, R and K , is considered to be much greater. For the rate of corrosion v_c the values 0.015 and 0.09 have already been mentioned as the upper and lower limit. When applying equations [6.23] and [6.24], we obtain:

$$\mu(v_c) = \sqrt{0.015 \times 0.090} = 0.036 \text{ mm/year}$$

$$V(v_c) = \frac{1}{4} \ln 6 = 0.45$$

Table 6.7 Review of the carbonation variables and their stochastic properties

	Description	Distribution	Mean	Standard deviation
d	Nominal concrete cover of 15 mm	Log-normal	20	5
d	Nominal concrete cover of 30 mm	Log-normal	35	3.85
Δ	Distance maximum–mean carb. depth in mm	Log-normal	5	1.0
R	Influence factor for the type of cement	Log-normal	2.0	0.3
K	Climate factor	Log-normal	0.7	0.14
w	Water–cement ratio	Log-normal	0.5	0.025
E	Constant mm	Deterministic	0.08	–
ϕ	Diameter of reinforcement bar in mm	Deterministic	8	–
v_c	Rate of corrosion in mm/year	Log-normal	0.0–1	0.50*mean
s	Thickness of coating in mm	Deterministic	0.18	–
f_0	Damage coefficient for coating in mm/year	Log-normal	0.00001	0.00001
T_0	Durability parameter for coating in year	Log-normal	50	25
T	Maintenance period year	Deterministic	10/20	–

Note: A more exact calculation yields $\mu(v_c) = 0.040$ mm/year and $V(v_c) = 0.4$. These values have been adopted in Table 6.7, where $V(v_c)$ has been rounded off to 0.50. The high coefficient of variation $V(f_0) = 1$ for the imperfection parameter f_0 indicates a ratio $f_{0(\text{high})}/f_{0(\text{low})} = 50$. The estimate for $V(f_0)$ is based on the information that for any particular paint system it is difficult to estimate f_0 to an accuracy within a factor of 10. Finally, $V(T_0) = 0.50$ indicates that this durability value is in all probability between 20 and 100 years.

6.5.4.4 Probabilistic analysis

On the basis of the mathematical model according to equation [6.20] and the stochastic properties listed in Table 6.7, it is possible to determine a probability density function of the service life. This analysis will be confined to an approximate calculation for the mean service life and the coefficient of variation. The procedure adopted for the purpose is called the ‘level II/mean value approximation’ or the ‘first order, second moment approximation’ (FOSM) in the literature.¹⁵ According to this procedure, the mean service life is obtained by calculating the life on the basis of mean values

Table 6.8 Calculation of $\sigma^2(t_L) = \sum_{j=1}^{t_G} \left\{ \frac{\partial t_L}{\partial X_j} \sigma(X_j) \right\}^2$

Variable		Alternative			
X_j	Description	1	2	3	4
d	Cover	415	1561	1905	21994
Δ	Distance between max. and mean carbonation depth	15	59	72	864
R	Influence factor for type of cement	74	1187	74	74
K	Climate factor	132	2111	132	132
W	Water/cement ratio	150	2395	150	150
ϕ	Bar diameter	—	—	—	—
v_c	Rate of corrosion	6	19	6	6
s	Coating thickness	—	—	—	—
f_0	Imperfection coefficient for coating	—	—	2304	112084
T_0	Durability value for coating	—	—	16178	89366
T	Maintenance period	—	—	—	—
	$\sigma^2(t_L)$	792	7333	20820	224672

for all the variables. The standard deviation is determined by linearizing the function t with the aid of a Taylor series. We then obtain:

$$\sigma^2(t_L) = \sum_{j=1}^{t_G} \left\{ \frac{\partial t_L}{\partial X_j} \sigma(X_j) \right\}^2 \quad [6.25]$$

in which t_G = the target service life. The partial derivatives $\partial t_L / \partial X_j$ are calculated for the mean values of the stochastic variables. This calculation has been performed for all four alternatives and the results are given in Table 6.8.

The scatter in service life is found to be very great. Table 6.9 shows which variables are most responsible for this scatter. Thus it appears that with alternative (1) particularly, the depth of cover is of major influence. If the cover is increased, criteria such as the water/cement ratio and the climate factor become more important. If a coating is applied, the durability value T_0 predominates if maintenance is carried out every 20 years, whereas the initial imperfection coefficient f_0 predominates if maintenance is carried out every 10 years. It is particularly these variables that are affected by a high degree of scatter that should receive most attention in further research. Table 6.9 reveals that a probability analysis is essentially a sensitivity analysis where the sensitivity $\partial t / \partial X_j$ of the variable X_j is weighed with a measure for the variation, the standard deviation $\sigma(X_j)$, in relation to the scatter in the service life.

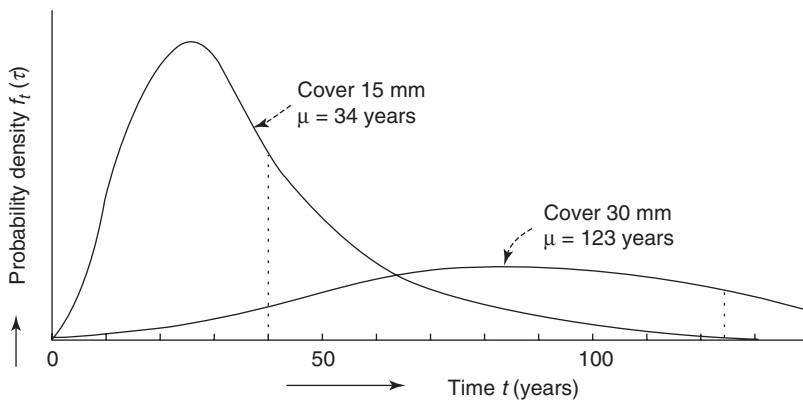
Table 6.9 Relative contributions of the stochastic variables to the service life variance for the four design alternatives

Variable		Alternative			
X_i	Description	1	2	3	4
d	Cover	62	21	9	10
Δ	Distance between max. and mean carbonation depth	2	1	1	1
R	Influence factor for type of cement	9	16	0	0
K	Climate factor	17	29	1	0
w	Water/cement ratio	19	33	1	0
ϕ	Bar diameter	—	—	—	—
v_c	Rate of corrosion	1	0	0	0
s	Coating thickness	—	—	—	—
f_0	Imperfection coefficient for coating	—	—	11	50
T_0	Durability value for coating	—	—	78	40
T	Maintenance period	—	—	—	—
	Total	110%	100%	100%	100%

By means of the calculations in previous sections, the mean value and the standard deviation of the service life t_L have now been determined. The distribution of t_L is as yet not known. The log-normal distribution should firstly be considered as a possible choice: the service life definitely appears to be positive and the scatter is large. On the basis of theoretical considerations, it can be shown that the distribution cannot be exactly log-normal. According to equation [6.20], the service life is composed of three terms. For each individual term the log-normal distribution seems appropriate, multiplication or division of variables conforming to this distribution will again yield a log-normally distributed variable. However, the sum of log-normally distributed variables will certainly not follow a log-normal distribution but for the present purpose what is exactly correct is less important than what will provide a serviceable model. To this end, the probability that the four alternative solutions will fail within the intended service life of 60 years has been calculated both on the basis of the log-normal model and with the aid of the ‘first order, second moment method’ (FOSM).¹⁵ In problems of this type, the FOSM method provides an almost exact calculation of the probability of failure. The results are presented in Table 6.10. From these results it can be inferred that the log-normal distribution yields good results for determining the probability of failure and given a mean value and standard deviation. Finally, the service life distributions obtained for the first two alternative design models are presented in Fig. 6.13 by way of illustration.

Table 6.10 Comparison of results on the basis of the log-normal distribution with results obtained by the FOSM method

Design alternative	$\mu(t_L)$ (year)	$\sigma(t_L)$ (year)	$P(t_L < 60)$ (log-normal distribution)	$P(t_L < 60)$ (FOSM)
1	34	28	0.86	0.76
2	123	86	0.20	0.13
3	103	144	0.50	0.39
4	417	474	0.05	0.05

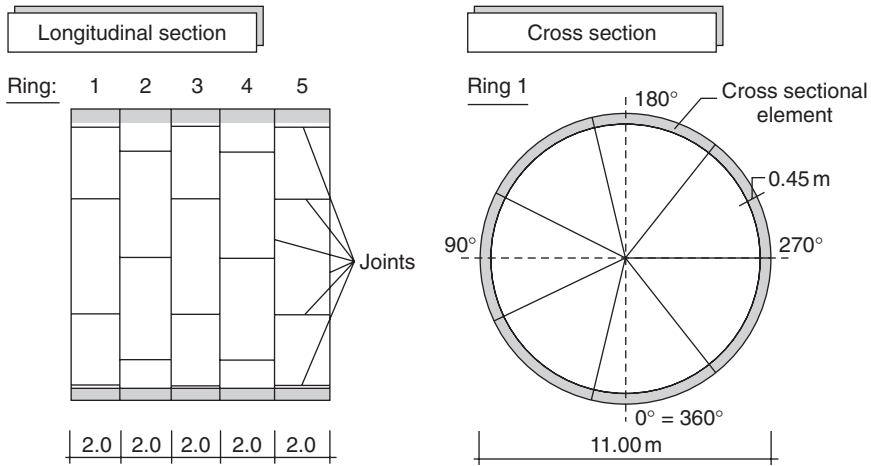


6.13 Service life distribution of two alternatives.

6.5.5 Case 2: Western Scheldt tunnel, service life design

6.5.5.1 Introduction

In the previous case described in Section 6.5.4, it was concluded that it is possible to use the same consistent design for safety, serviceability and durability of structures with the aid of probabilistic analysis. The framework and the design method for such a design were elaborated in the late 1990s in a EU research project named DuraCrete.¹⁶ The basis for this research was the result of previous work done by international organizations such as RILEM,⁸ IABSE¹⁷ and CEB.¹⁸ The first structure where the service life was based on the principles of this design method is the Western Scheldt Tunnel in the southern delta area of the Netherlands. The tunnel is fundamentally a bore tunnel made of prefabricated segments and two ramps made *in situ*, both in reinforced concrete. The tunnel offers a north-south road traffic connection between peninsulas in the Dutch province of Zeeland, without



6.14 Longitudinal and cross section of the Western Scheldt tunnel.

obstructing shipping traffic on the Western Scheldt between the North Sea and the Antwerp harbour in Belgium.

6.5.5.2 Western Scheldt bored tunnel

The DuraCrete model has been applied to the service life design of the Western Scheldt Tunnel in the Netherlands.⁹ The tunnel is bored and consists of two tubes with a diameter of 10.3 m and a length of 6.5 km. Figure 6.14 shows two sections and an overview of one of the lining segments. The construction of the tunnel started in 1999 and is scheduled to open to traffic in 2003. The contract for this tunnel is a so-called 'design and construct' contract.

The design requirement with respect to durability was a service life of at least 100 years. However, the contract supplied no design method, no performance specifications and no requirements for minimum reliability. After deliberation between the tunnel owner and the contractor, it was decided to base the design on the DuraCrete design methodology. The limit states that had to be considered were the structural limit states listed in the Dutch Building Decree but combined with the effects of carbon dioxide, chloride ions and corrosion. Furthermore, a limit state for reducing the probability of repair was added by defining the prevention of corrosion propagation. In accordance with the Dutch Building Decree, the reliability index for the serviceability limit state was defined as $\beta = 1.8$ and for the ultimate limit state as $\beta = 3.6$. These values are almost equivalent to the Eurocode requirements of 1.5 and 3.8, respectively (Table 6.11). The limit state for reducing the probability of repair was considered as a serviceability limit state.

Table 6.11 Limit values for the reliability index of different consequence classes, related to the consequences of failure, during a reference period, compared with design service life, of 50 years according to the Eurocode

Consequence Class	CC1 'Low'	CC2 'Medium'	CC3 'High'
Reliability Class	RC1	RC2	RC3
Reliability Index	3.3	3.8	4.3

The different models used for this durability design consist of design parameters, such as structural dimensions, environmental parameters and material properties that correspond to the load and resistance variables of the structural design procedure. In the case of chloride-induced corrosion, the following model describing corrosion initiation has been identified:

$$x(t) = 2 \cdot C_{(Crit)} \cdot \sqrt{k_t \cdot D_{RCM,0} \cdot k_e \cdot k_c \cdot \left(\frac{t_0}{t}\right)^n} \cdot t \quad [6.26]$$

where:

$$k_t \cdot D_{RCM,0} = D_0 \quad [6.27]$$

$$C_{(Crit)} = erf^{-1} \left(1 - \frac{C_{Crit}}{C_{SN}} \right) \quad [6.28]$$

with:

erf^{-1} inverse error function

t exposure time [year]

The other variables and their parameters are given in Table 6.12.

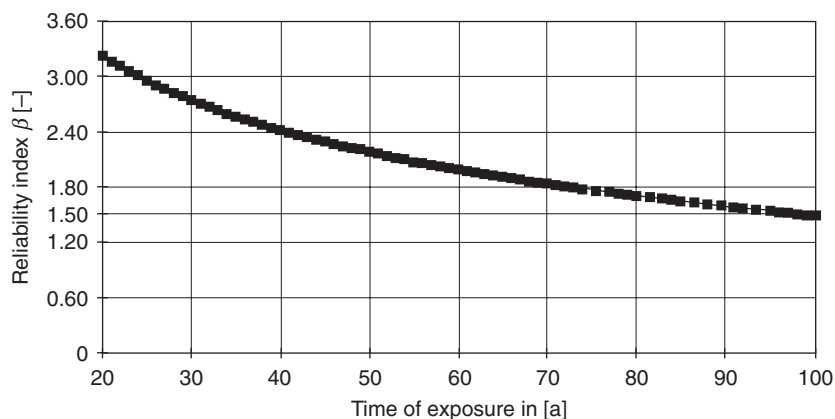
Taking the example of the onset of chloride-induced corrosion, the durability inputs are as follows:

- The serviceability limit state is based on the requirement that the chloride concentration at the reinforcement surface must not reach the critical chloride concentration.
- Resistance R is given by the critical chloride concentration and the quality and thickness of the concrete cover.
- Load S is represented by the actual chloride concentration at the reinforcement level. This depends on material parameters (chloride diffusion coefficient) and environmental effects.

It is possible to make a probabilistic calculation based on this information. This has been done with the Strudel software package. For each successive year, the probability of failure has been calculated and added to the

Table 6.12 Overview of the basis variables

Variable	Parameter	Description	Dimension	Mean	Standard Deviation	Distribution
1	x_c	Concrete cover	[mm]	37	2	Exponential
2	$D_{RCM/0}$	Chloride migration coefficient	$[10^{-12} \text{m}^2/\text{s}]$	4.75	0.71	Normal
3	C_{crit}	Critical chloride content	$[\text{m}/\text{m}\%]$	0.70	0.10	Normal
4	n	Age exponent	[-]	0.60	0.07	Normal
5	k_t	Test model factor	[-]	0.85	0.20	Normal
6	k_e	Climate factor	[-]	1.00	0.10	Normal
7	k_c	Curing constant	[-]	1.00	0.10	Normal
8	C_{SN}	Surface chloride content	$[\text{m}/\text{m} \ %]$	4.00	0.50	Normal
9	t_0	Basic testing time for $D_{RCM/0}$	[year]	0.0767	–	Deterministic



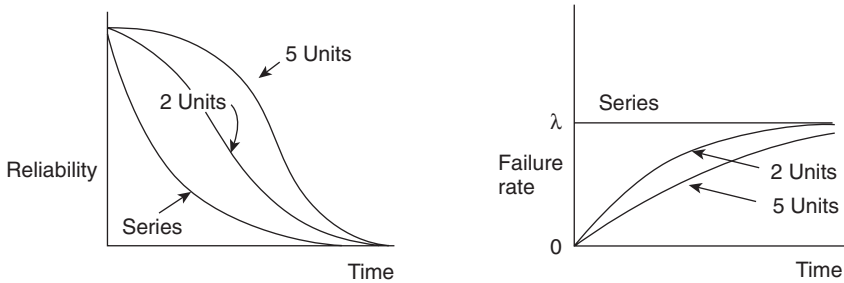
6.15 Reliability Index versus Time of Exposure.

probabilities of failure of previous years. Furthermore, this value has been transferred to the reliability index β . The result is presented in Fig. 6.15. The figure shows that the reliability index $\beta = 1.5$ after a period of 100 years. The design requirement was, however, $\beta = 1.8$ which meant that additional durability measures had to be applied.

6.6 Redundancy and over-design

Structures sometimes contain sections that are essential to safety, but give no warning of imminent failure; they simply fail. Stress corrosion of the stainless steel hanging strips discussed in Section 2.4.3 caused sudden collapse of the swimming pool ceiling. When the first hangers failed, the other strips were not able to bear the load and failed too. Collapse of the southern part of the roof of the Berlin Congress Hall in 1980 happened without any warning and was due to stress corrosion of the prestressed cables.¹⁹

In maintenance strategies for manufacturing processes, the principle of redundancy is widely accepted for those process steps for which failure would have dramatic effects. The redundancy principle means that parallel to the critical section of the structure, another similar or different type of section with the same function is installed. This can take over the whole function as soon as the first section fails (standby redundancy). Alternatively, the two structural elements can both function (active redundancy), but if one of the two fails the other one can take over the full function. Figure 6.16 shows the effect of redundancy on reliability and failure rate.¹ This principle is applied in civil engineering, but its application does not make sense in terms of topmost durability. An example of the application of the redundancy principle is in the tensile piles of the retaining walls in



6.16 Effect of redundancy on reliability and failure rate.¹

the foundation of the new Alexandria library in Egypt. Double reinforcement is applied allowing one of the reinforcements to corrode away, while still leaving the redundant reinforcement to function in this harsh marine environment.²⁰

If a structure design is inadequate and no deterioration processes have been taken into account, any decrease in the structural properties of structural materials, such as corrosion in steel or fungal attack in wood, will lead to a situation where the safety requirements are no longer met. However, in most instances there is some overdesign present; for instance finite element calculations on the Deira–Shindagah tunnel in Dubai (see Section 3.10) showed that the safety was still safeguarded, although some rebars 80mm in diameter were fully corroded. Owing to the (non-intentional) overdesign, the tunnel does not have to be taken out of operation immediately.

6.7 Maintenance strategies

6.7.1 Maintenance as part of the design

Service life design should include maintenance of the structure. During the design stage a choice can be made between minimizing maintenance, optimizing the life-cycle costs, or optimizing the environmental effects of the structure. In reality, maintenance is often not part of the design and a maintenance programme at a later date is only begun as defects become obvious.

6.7.2 Objectives

In practice, maintenance of engineering structures has various objectives:

- Prevention of corrective maintenance and ensuring that the structure always performs above a specified minimum level during its design service life.

- Repairing the structure in the best possible way.
- Repairing the structure as quickly as possible.
- Repairing the structure at the lowest cost.
- Repairs should cause the least inconvenience to the users of the structure.
- Those parts that are affected should be replaced, preferably with standard spare parts rather than allowing repairs to take place.
- Repairs should be made in such a way that further corrective or preventive maintenance is no longer required during the remaining service life.
- Repairing the structure and at the same time extending or adding new functions to the structure.
- Optimizing the maintenance cost at a specified state of performance.
- Optimizing the cost for the time the structure is non-operational because of repairs.
- Optimizing the cost of maintenance and non-operation.
- Optimizing the life-cycle costs of the structure.

Cost-optimizing strategies will be discussed in Section 6.8.

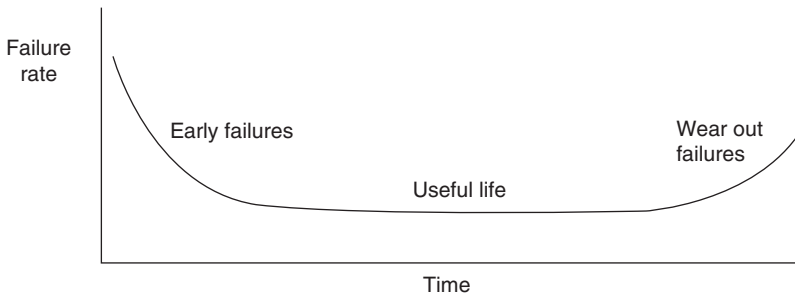
6.7.3 Factors influencing the choice of strategy

The maintenance strategy chosen depends largely on the structure, the degradation mechanism(s) and the situation in question. If the structure is an office building, nuisance to the users will be an important factor. During the repairs on the Deira-Shindagh tunnel, discussed in Section 3.10, one of the tunnel bores had to remain in use because there were no nearby alternatives for traffic to pass the Dubai Creek. Replacement of the steel barrier plates in the Eastern Scheldt storm-surge barriers with fibre-reinforced polymer (FRP) sheets or thin high-strength reinforced concrete slabs has been considered, yet the consequence of having to change the whole hydraulic lifting system is problematic in terms of dimensions and weight.

If the degradation mechanisms are such that the structure shows signs that something is wrong, for instance the appearance of cracks over the reinforcement indicating corrosion or blisters on the steel structure coating, it is time to develop an optimum maintenance programme. If a sudden collapse of the structure is possible, however, for instance if stress corrosion is detected in the steel tendons of a suspension bridge, immediate action is required.

6.7.4 Inspective, preventive and corrective maintenance

Inspection is part of any maintenance management system. The type of inspection to be carried out depends on the structure in question, the



6.17 Bath-tub curve.

inspection techniques available and the maintenance strategy objectives, as discussed in Section 6.4.2. Visual inspection is still the most advanced method of inspection, but it can be assisted by other methods, such as half-cell measurements for reinforcement corrosion, rebound hammering to detect any deterioration of wood or concrete and built-in corrosion cells enabling continuous monitoring of chloride penetration in concrete structures.

Inspection frequency can take account of the high occurrence or so-called bath-tub curve of the failure rate, shown in Fig. 6.17. This concerns all kinds of failures in various parts of a structure. Directly after completion of the structure, the failure rate is high, i.e. so-called early failures or infant mortality failures. In engineering structures, this concerns incidents such as failing joints and electrical devices that do not function. Thus the failing high-solids epoxy coating on the Eastern Scheldt barrier, as discussed in Section 2.5.3.1, can be regarded as an early failure. The failure rate subsequently decreases to a lower level, where it remains until the end of the service life, when the failure rate increases again. Failures at the end of the service life are different in nature to those at the beginning. They are strongly related to ageing, e.g. reinforcement corrosion will manifest itself at those locations where the cover was underspecified or the concrete quality less. The bath-tub curve can also occur at intermediate time-intervals, e.g. when the technical service life of a protective coating system on steel comes close to its end.

To avoid declining performance and the exceeding of a critical performance level limit, an inspective maintenance programme can be carried out that monitors the structure against specific performance indicators. When declining performance is observed and defects appear that indicate failure in the near future but performance is still below a critical level, preventive measures (reactive) can be taken to bring performance to a higher level so the structure can continue to function during the remaining service life. It can also be the strategy to prescribe a preventive maintenance programme after completion of the structure or after repairing the structure.

That means that maintenance is carried out regardless of actual performance, e.g. regular painting of steel structures. Action must be taken once performance falls below the acceptable level. This is called corrective maintenance or essential maintenance (see Figure 6.1). Inspective maintenance and prescribed preventive maintenance are part of a maintenance management strategy. An optimum strategy depends on the objectives of maintenance, as discussed in Section 6.7.2.

6.7.5 Maintainability and repairability

The definition of maintainability is: 'The extent to which it is feasible that the performance of a product will be restored to the intended original minimum level within a given period of time'. Repairability can be defined in a similar manner and is part of maintainability.

There are two factors determining maintainability:

- Accessibility of the structure to inspection so as to allow for proper diagnosis.
- Ease of remedial action to restore structural performance.

Diagnosis is promoted by:

- A simple and transparent design concept and execution.
- Built-in or easy-to-apply test devices.
- As-built drawings and maintenance manuals adjusted to the needs of maintenance personnel.
- Availability of expert maintenance personnel and foreign expertise.

Remedial actions are made easier:

- If vulnerable parts can be easily repaired, removed and replaced. Examples are the following:
 - sliding bearing plates of steel, rubber and Teflon on bridge pier heads show decreasing performance in time. If there is sufficient space and bearing capacity on the pier heads, the bridge can be jacketed and replacement can be carried out easily;
 - easy replacement of bridge deck joints that have a service life of possibly 15 years;
 - the possibility of easy instalment of a cathodic protection system by ensuring electrical continuity of the reinforcement (e.g. in the Great Belt Tunnel in Denmark) when reinforcement corrosion becomes an (unexpected) threat;
- If maintenance personnel are continuously available and spare parts are in stock. In the case of the King Fahad Causeway (see Section 3.4.3.2), for instance, a very strict maintenance programme is applied, maintenance materials are in stock and immediate action is taken when a defect is observed.

6.7.6 Maintenance and performance models

In the case of performance-based design for durability, models are occasionally used to predict the service life, e.g. the diffusion model for chloride-initiated corrosion discussed in Section 6.5.5.2. Practice can be different from theory. Moreover, many durability problems with structures are not due to failing designs, but to the inadequate quality of job execution. It is, therefore, wise to monitor performance in structures against these indicators, even if a maintenance-free service life was designed for. By monitoring chloride penetration, for instance, the design predictions can be checked and adjusted if required.

In steel structures under dynamic loading, such as heavy traffic steel bridge decks, service life prediction is subject to considerable uncertainties.²¹ The models applied are unreliable. By monitoring stresses and strains in practice, performance can be followed properly and predictions about behaviour can be highly reliable.²¹ This obviously means that there is no real durability design, but adequate maintenance is possible.

6.8 Life-cycle costing

6.8.1 General

Within the framework of specified requirements of structural performance over the design service life, cost will be an important item to optimize. It is quite common when designing structures to consider only the investment costs of the building process, yet there is a growing awareness that this is only part of the process. Low investments can mean high operational costs and maintenance costs where the total can exceed high investments with low operational and maintenance costs. Considering the cost of structures over the whole life cycle is becoming increasingly popular.

Life-cycle costing is defined as the total discounted cost of owning, operating, maintaining, and disposing or recycling a structure or a building system over a period of time.

In this section we will restrict ourselves to owning, operating and maintenance costs, while elaborating on the probabilistic approach given in Section 6.5.

6.8.2 Principles of cost analysis

6.8.2.1 Capitalizing costs

The cost analysis of a building can be based on annual cost or on capitalized cost. In the former case, the investment cost of the building is trans-

lated into an annual amount comprising interest and redemption. In the latter case, i.e. involving capitalization of cost, every future expenditure is converted back to an amount that can be added to the direct investments. In principle, both approaches lead to the same conclusions.

In some cases, termination of the service life of a part, member or component of a structure is determined solely by its ability to perform its function: as soon as it fails to satisfy the functional requirements (and assuming that repair is too expensive), it is replaced (window-pane, electric bulb, motor). External factors can also determine the service life, however. For example, an offshore drilling platform need not have a service life of more than 30 years because the oil well in question will be exhausted by the end of that period. All that is required of the structure and its members is that they will not give rise to problems within the service life. A similar approach may be adopted with regard to housing or industrial building construction, based on an intended service life of, for example, 50 years. If a member of the structure fails in one way or another within that period, it means that damage will occur, necessitating replacement or repair.

In the present section on capitalized cost, only the case of intended 'target' service life will be considered. For many purposes, it will suffice to conceive the anticipated capitalized cost as being composed of three terms: the direct investment (initial expenditure), the cost of maintenance and the risk (anticipated loss, being the cost incurred when remedying deterioration or damage). Both maintenance cost and loss due to deterioration or damage must be calculated for each year of planned service life and then be capitalized and added up:

$$E\{C_{cap}\} = S + \sum_{j=1}^{t_G} \frac{V_j}{(1+r')^j} + \sum_{j=1}^{t_G} \frac{P\{F_j\}D_j}{(1+r')^j} \quad [6.29]$$

where

$E\{C_{cap}\}$ = Expectation of the capitalized cost C_{cap}

S = Direct investment

V_j = Maintenance cost and administration expenses for year j

$P\{F_j\}$ = Probability of failure in year j

D_j = Loss due to failure in year j

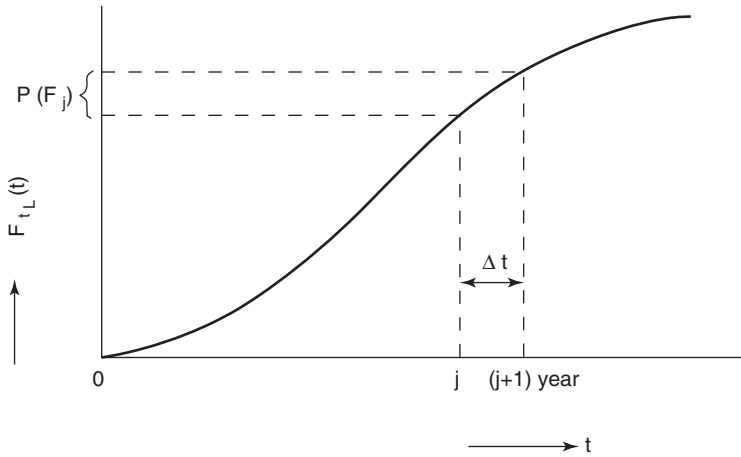
r' = Real rate of interest (nominal rate minus inflation)

t_G = Target service life (in years)

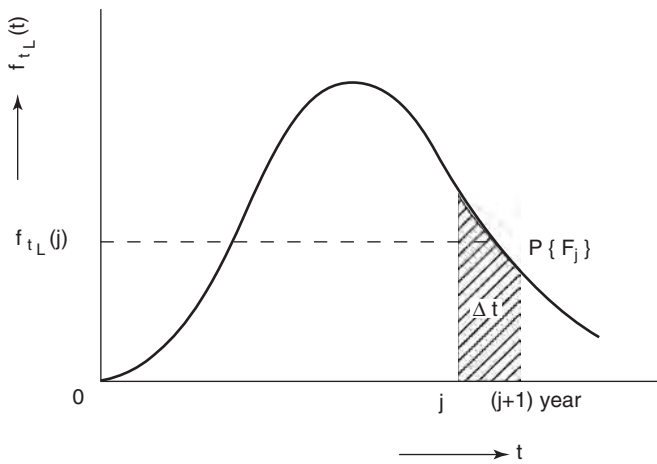
The probability of failure $P\{F\}$ in year j follows from the probability distribution for service life t_L (in years) (see Figs 6.18 and 6.19).

$$P\{F_j\} = P\{(j-1) \leq t_L < j\} = F_{t_L}(j) - F_{t_L}(j-1) \quad [6.30]$$

Alternatively, we can start from the probability density function $f_{t_L}(t)$ as follows:



6.18 Distribution of t_L .



6.19 Probability density of t_L .

$$P\{F_j\} = [F_{t_L}(j) - f_{t_L}(j+1)]\Delta t \quad [6.31]$$

$$P\{F_j\} = F_{t_L}(t)\Delta t \quad [6.32]$$

where

$t = j$ years

$\Delta t = 1$ year

The real interest rate, i.e., the nominal interest rate corrected for inflation is adopted in equation [6.29]. When determining D ; and V , it is therefore

not necessary to take account of a rise in costs due to inflation. The term $(1 + r')^j$ indicates that the value V and D ; in year j correspond to $V/(1 + r')^j$ and $D(1 + r')^j$ in the initial year.

Example

Suppose that the annual cost of maintenance V is 1.2% of the direct investment S , that the probability of loss (due to damage or deterioration) is 10^{-4} for each year and that the loss D_j is equal to $2S$. Such a constant probability of loss occurs in the case of wear. The sum of the following geometric progression is used for the calculation:

$$\sum_{i=1}^{t_G} \frac{1}{(1 + r')^i} = \frac{1}{r'} \left\{ 1 - \left(\frac{1}{1 + r'} \right)^{t_G} \right\} \quad [6.33]$$

For the present example:

$$E\{C_{cap}\} = S + \frac{0.012S}{r'} \left\{ 1 - \left(\frac{1}{1 + r'} \right)^{t_G} \right\} + \frac{0.001 \times 2S}{r'} \left\{ 1 - \left(\frac{1}{1 + r'} \right)^{t_G} \right\}$$

With $r' = 0.02$ and $t_G = 50$ this expression yields:

$$E\{C_{cap}\} = S + 0.38S + 0.06S = 1.44S$$

It thus appears that, in this example, maintenance and risk amount to 44% of the direct investment.

6.8.2.2 Cost calculation on an annual basis

Firstly we will consider the case where there is only direct investment and where every year a certain constant value X (annuity) is paid throughout the entire service life of t_G years. The amount of X paid in year j corresponds to a direct redemption equal to $X/(1 + r')^j$. Since the total redemption must be equal to S , it is thus possible to calculate X from:

$$S = \sum_{j=1}^{t_G} \frac{1}{(1 + r')^j} = \frac{X}{r'} \left\{ 1 - \frac{1}{(1 + r')^{t_G}} \right\} \quad [6.34]$$

or:

$$X = Sr' \left\{ \frac{(1 + r')^{t_G}}{(1 + r')^{t_G} - 1} \right\} \quad [6.35]$$

Example: If $r' = 0.02$ and the intended service life $t_G = 50$ years, it then follows that $X = 0.0318S$ (if no interests were payable, then $X = S/50 = 0.02S$).

If r' in equation [6.34] denotes the nominal interest, this represents a normal annuity (i.e., nominally the same amount every year). However, for

the purpose of this book, it is preferable to work with the real interest because the annual increase of an amount then keeps pace with the price level.

The cost of maintenance and risk can, if they are the same for each year, be directly added on. But if these costs are not constant, they must be capitalized and added to S , after which the value of X can be determined with the aid of equation [6.35]. The capitalized cost is, therefore, often a more suitable criterion for the optimization problems considered in this book.

In the foregoing, the real interest has always been incorporated and cost increases due to inflation have been left out. The drawback of this procedure is that the calculated amounts of money do not directly correspond to the amounts actually payable.

Example

By way of illustration, an optimization problem will be dealt with in this section. Here a simple carbonation problem is discussed where, for a given service life, the optimum depth of concrete cover is determined (see also Section 6.4.2).

Carbonation refers to the combining of free lime in concrete with carbon dioxide from the atmosphere, as a result of which the pH in concrete is lowered. If concrete thus ceases to have sufficient alkalinity, it loses its protection and the reinforcing steel can corrode, which can be regarded as constituting the end of the service life. The following approximation (simplified model with respect to equation 6.12) for carbonation progress is used:

$$d^2 = At \quad [6.36]$$

where

d = the depth of carbonation (mm)

A = a constant (mm²/year)

t = the exposure time (years)

The service life t_L is attained when carbonation depth d becomes equal to the depth of concrete cover c :

$$t_L = \frac{c^2}{A} \quad [6.37]$$

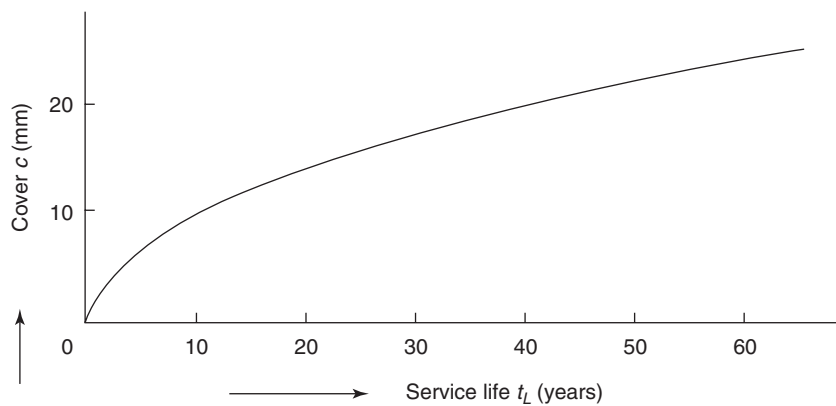
For $A = 10 \text{ mm}^2/\text{year}$, the service life t_L of the concrete is given as a function of c in Fig. 6.20. It appears from this diagram that a 24 mm cover corresponds to a service life of 60 years. Conversely, to obtain a service life of 60 years, it will be necessary to provide a 24 mm cover. If the cost of the cover is 0.20 euro per m² of slab surface and per mm of cover thickness

(which is a reasonable figure for a concrete gallery slab), the required cover will cost:

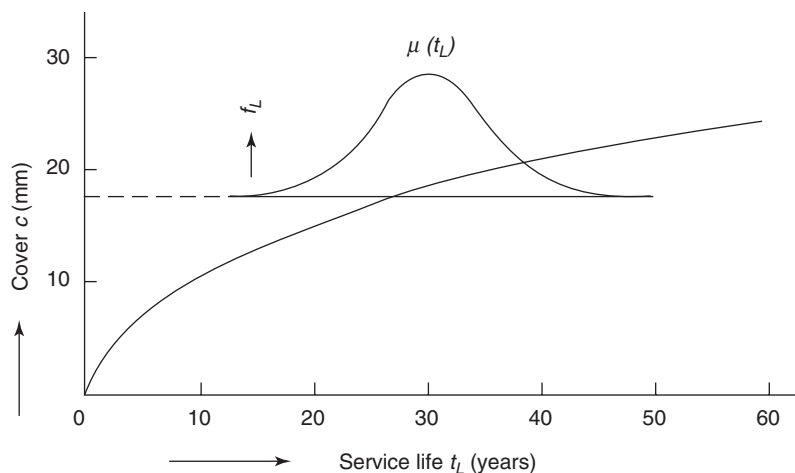
$$C_{cap} = 24 \times 0.20 = \text{€}4.80 \text{ per m}^2 \text{ of slab}$$

It is, however, not very realistic to conceive t service life as a deterministic quantity, for carbonation does not proceed everywhere at the same rate, and the depth of cover itself will never exactly conform to the nominal value indicated on the drawing.

Assuming that Fig. 6.20 represents mean service life and that the actual values display a scatter in relation to this with a coefficient of variation of 0.33, we obtain Fig. 6.21.



6.20 Service life t_L as a function of the cover c .



6.21 Service life of the cover as a stochastic quantity.

Now, for a given depth of concrete cover, it is possible to calculate with what probability the entire cover will become carbonated within a certain period. The total capitalized cost according to equation [6.29] then is:

$$E\{C_{cap}\} = S + \sum_{j=1}^{t_G} \frac{V_j}{(1+r')^j} + \sum_{j=1}^{t_G} \frac{P\{F_j\}D_j}{(1+r')^j}$$

With $S = 0.2c$; $V_j = 0$; $D_j = D$ and an intended service life $t_G = 60$ years this reduces to:

$$E\{C_{cap}\} = 0.2c + D \sum_{j=1}^{60} \frac{P\{F_j\}}{(1+r')^j}$$

Because $P\{F_j\}$ is not constant, summation is difficult to carry out. The following approximation can, therefore, be suitably adopted for equation [6.29.]:

$$E\{C_{cap}\} = 0.2c + \frac{D}{(1+r')^{60}} P(t_L < 60) \quad [6.38]$$

What this approximation comes down to technically, i.e. from the financial point of view, is that every 60 years, failure is assumed to occur in the sixtieth year. The associated financial loss is somewhat underestimated.

The sum of the probabilities $P\{F_j\}$ for the years 1 to 60 is equal to the probability of the service life being less than 60 years, so that:

$$E\{C_{cap}\} = 0.2c + \frac{D}{(1+r')^{60}} P(t_L < 60) \quad [6.39]$$

If a normal distribution is adopted for t_L , the probability that t_L is less than 60 years can be determined from:

$$P(t_L < 60) = \Phi(-\beta) \quad [6.40]$$

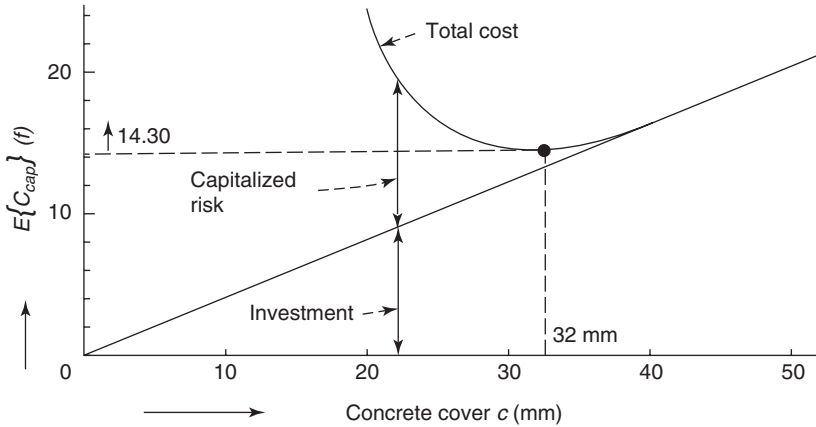
where:

Φ = standard normal distribution ($\mu = 0$ and $\sigma = 1$)

For example, assuming $c = 24$ mm (as found earlier on), we obtain $\mu(t_L) = 60$ years, so that $\beta = 0$. The probability of failure then is 50%. For loss amounting to $\text{€}25/\text{m}^2$ and a real interest rate $r' = 0.02$, it follows that:

$$E\{C_{cap}\} = 4.80 + \frac{25}{1.02^{60}} 0.50 = 4.80 + 3.80 = \text{€}8.60 / \text{m}^2$$

If a 30 mm depth of concrete cover is chosen, the following results are obtained:



6.22 Optimization of the depth of the cover with respect to carbonation.

$$\mu(t_L) = \frac{c^2}{a} = \frac{900}{10} = 90 \text{ years}$$

$$\sigma(t_L) = 0.33 \times 90 = 30 \text{ years}$$

$$\beta = \frac{\mu(t_L) - 60}{\sigma(t_L)} = \frac{90 - 60}{30} = 1$$

$$P\{t_L < 60 \text{ year}\} = \Phi(-\beta) = \Phi(-1) = 0.16$$

$$E\{C_{cap}\} = 0.2 \times 30 + \frac{25}{1.02^{60}} 0.16 = 6 + 1.2 = 7.2 \text{ €/m}^2$$

Increasing the cover results in an increase in direct cost and risk reduction, while the total cost expectation decreases. The optimum cover can be found by repeating this calculation for a number of values of c , as indicated in Fig. 6.22. The optimum is obtained for $c = 32 \text{ mm}$, the probability of failure then being 10%, in the present example.

6.8.3 Example of gallery slab maintenance costs

The lifetime costs for the various durability design alternatives of the balcony slab discussed in Section 6.5.4.1 are calculated by way of example.

For the purpose of economic optimization, it is necessary to obtain insight into the cost of the increased concrete cover, the application and maintenance of protective paint coatings and repairs. The cost figures adopted in this example are given in Table 6.13. Uncertainties in these costs, and also

Table 6.13 Review of cost data (in €/m²)

Operation	Cost
Manufacturing and installing concrete slabs in a new building (h = depth of slab in mm)	20 + 0.2h
Replacement at h = 150 mm	137.5
Removing concrete cover, applying sprayed concrete	112.5
Removing concrete cover, repairing with polymer mortar	125
Painting new concrete (epoxy)	25
Painting existing concrete	40
Maintenance of coating	20
Repair of coating	40

Table 6.14 Expected capitalized costs (in €)

Cost item	Design alternative			
	1	2	3	4
Extra concrete cover	–	30	–	–
Coating	–	–	250	250
Maintenance of coating	–	–	335	565
Repair expectation	430	85	130	20
E(C _{cr})	430	115	715	835

the uncertainty in the interest rate to be applied, have been ignored for the sake of simplicity.

The capitalized cost expectation according to equation [6.29] will be adopted as the basis of comparison for optimization and is written as:

$$E\{C_{cap}\} = S + \sum_{j=1}^{t_G} \frac{V_j}{(1+r')^j} + \sum_{j=1}^{t_G} \frac{P\{F_j\}D_j}{(1+r')^j}$$

where r' (= real rate of interest) is taken as 0.02, while V_j and D_j are the cost of maintenance and the loss (cost associated with damage or deterioration), respectively, in the year j . The planned service life of 60 years is interpreted in such a manner that all necessary repairs and maintenance are carried out up to and including the sixtieth year of service; for this reason $t_G = 60$ years is adopted in the equation. Any unexpired service life of the structure beyond that period, although probably available, will not be considered.

In Table 6.14, the values for capitalized costs according to equation [6.29] have been calculated on the basis of the cost data given in Table 6.13, the

service life data given in Section 6.5.4.4 and a few additional assumptions concerning the extent of deterioration or damage.

The slab is 1.60m wide and 6.30m long, so that its surface area is 10m². A slab with a 15mm concrete cover, without coating, represents the point of reference for the cost comparison. For this slab no extra cost is incurred for increasing durability. For the slab considered as alternative (2), with a 30mm cover, a sum of $15 \times 0.2 = \text{€}3/\text{m}^2$, i.e., a total of €30 per slab, is spent on improving durability. Applying a coating to new concrete costs €25/m², i.e. €250 for the whole slab. This extra investment occurs also in the alternatives (3) and (4).

Maintenance is carried out only in the case of these two last-mentioned alternatives. First, consider alternative (4) with maintenance at 10 year intervals. Maintenance costs €20/m², i.e., €200 for the slab. The calculation of the capitalized cost of maintenance proceeds as follows:

$$\sum_{j=1}^{t_G} \frac{V_j}{(1+r')^j} = \frac{200}{1.02^{10}} + \frac{200}{1.02^{20}} + \dots + \frac{200}{1.02^{50}} =$$

$$\frac{200}{1.28} + \frac{200}{1.49} + \frac{200}{1.81} + \frac{200}{2.21} + \dots + \frac{200}{3.69} = \text{€}5.65$$

For alternative (3), it is assumed that maintenance at 20 year intervals will, in 50% of cases carried out, involve repair work costing €400 instead of €200, i.e., €300 on average. Hence the capitalized cost of maintenance is:

$$\sum_{j=1}^{t_G} \frac{V_j}{(1+r')^j} = \frac{300}{1.02^{20}} + \frac{300}{1.02^{40}} = \text{€}337.5$$

The final item to be determined is the loss, i.e. the cost associated with remedying damage or deterioration. In a more or less arbitrary manner, the following possible cost amounts can be distinguished (see also Section 6.5.4):

- Complete replacement: $10 \times 137.5 = \text{€}1375$
- Major repairs with sprayed concrete over an area of 4m²: $4 \times 112.5 = \text{€}450$
- Minor repairs with polymer mortar over an area of 2m²: $2 \times 125 = \text{€}250$

As a rule, the loss arising from refurbishment to remedy the effects of damage or deterioration will be between €250 and €450. Major repairs or complete replacement occur less frequently. In further calculations, an average figure of €500 will be adopted for this loss. An exception is the occurrence of premature damage in the case of alternative (1). If a slab with a 15mm cover and no coating is found to be giving trouble (due to deterioration) in a number of places within perhaps a 30 year period, it is evident that complete replacement is often likely to offer the most rational remedy.

Otherwise, it is fairly certain that more trouble will occur at other locations in the slab with time. For alternative (1) and $t_L < 30$ years, the loss is accordingly taken as €1000.

To calculate the risk, we must calculate the probability of failure in each year. For the sake of convenience, six 10 year periods will be considered:

$$\sum_{j=1}^{t_G} \frac{P\{F_j\}D_j}{(1+r')^j} = \frac{D_{10}}{(1+r')^{10}} P(0 \leq t_L < 10) + \frac{D_{20}}{(1+r')^{20}} P(10 \leq t_L < 20) + \dots + \frac{D_{60}}{(1+r')^{60}} P(50 \leq t_L < 60)$$

The probability $P(50 < t_L < 60)$ is equal to $P(t_L < 60) - P(t_L < 50)$. All the data needed for assessing the risk are thus available. For alternative (1), i.e., a 15 mm cover and no coating, we obtain:

$$\begin{aligned} \sum_{j=1}^{t_G} \frac{P\{F_j\}D_j}{(1+r')^j} &= \frac{1000 \times 0.09}{1.02^{10}} + \frac{1000 \times 0.25}{1.02^{20}} + \\ &\quad \frac{1000 \times 0.29}{1.02^{30}} + \frac{500 \times 0.15}{1.02^{40}} + \frac{500 \times 0.09}{1.02^{50}} + \frac{500 \times 0.04}{1.02^{60}} \\ &= 75 + 168 + 133 + 34 + 16 + 6 = \text{€}432 \end{aligned}$$

For the period $0 < t_L < 30$, the loss has been taken as €1000 and for $t_L > 30$, it has been taken as €500, as explained above. The factors 0.09, 0.25, etc. are the probabilities of failure in the period 0–10 years, 10–20 years, etc. The calculation for the other design alternatives is entirely similar (except that the loss is then €500 for the entire service life). The results are given in Table 6.14.

Conclusion

By far the most advantageous solution is found to consist in increasing the concrete cover depth. The direct cost that this entails is low (€30), while deterioration or damage occurs only after a long time. To remedy these defects, it is often sufficient to carry out relatively simple repairs, while the interest ensures a substantial decrease in the capitalized cost.

It is notable that applying and maintaining the coating is more expensive than alternative (1), despite the marked reduction in the probability of deterioration or damage. In view of this result, the coating should be applied primarily for aesthetic reasons, its protective effect being an additional bonus. Alternatively, cheaper paints for application to concrete may be used if the object is simply to improve the appearance of the structure; such paints are in general not impermeable to carbon dioxide, however, so that in this respect they offer little or no protection. An advantage associated

with the use of protective coatings is the reduced probability of inconvenience to the residents or users of the building caused by repair work. This applies only to the alternatives requiring frequent maintenance.

6.8.4 Sheet-pile wall alternatives

The Dutch Ministry of Transport, Public Works and Water Management has made life-cycle costing calculations concerning seven alternatives for sheet-pile walls to be applied along the Dutch coast for two design service lives of 20 and 50 years, respectively. Five of the alternatives concern steel sheet piles with various protection systems. The other two are manufactured from high-strength concrete. The walls had to be 13 m high, exceeding the highest tidal level by 2 m and allowing for a tidal movement of 3 m. The sheet piles are driven into the soil to a depth of 6 m.

The following protective measures are considered for the five steel alternatives:

- Organic coating, which should be recoated every 15 years.
- Sacrificial steel layer; a layer of steel is allowed to corrode away.
- Thermally sprayed aluminium.
- Cathodic protection with sacrificial anode (CPSA).
- Cathodic protection with impressed current (CPIC).

The protective measures and their capitalized costs are given in Table 6.15. One high-strength concrete concerns a reinforced concrete of strength class C55/67 according to the European concrete standard EN 206-1 *Concrete, Specification, Performance and Conformity*. The other high-strength concrete can be referred to as a super high-strength concrete of class C200 (beyond the range given in EN 206). The very high strength allows a substantial reduction in thickness of the sheet pile in relation to the C55/67 grade, i.e. 240 kg/m² and 560 kg/m², respectively.

The alternative sheet-pile walls considered are:

- 1 *Steel coated*, type AZ26 of 155 kg steel/m² of which the coating consists of 3 layers of a water-borne epoxy system. It is assumed that the steel must be recoated every 15 years over a length of 7.5 m from the top. Recoating must take place below the tidal zone and in the tidal zone also, which requires the creation of a dry area around the sheet-pile wall during execution.
- 2 *Steel with a sacrificial steel layer*, type AZ36 of 194 kg steel/m². The extra sacrificial thickness of the steel is to suffice for 20 years. A higher thickness was not available. For the design service life of 50 years, a 300 µm aluminium layer is thermally sprayed on the steel after a 20 year period for the top 5 m. (See Section 2.5.4 for protection by metallic coatings.)

Table 6.15 Protective measures for a 20 or 50 year design service life of steel sheet-pile walls and their capitalized cost

Protection type	Thickness layer 20 years μm	Thickness layer 50 years μm	Cost ^a 20 years $\text{€}/\text{m}^2$	Cost ^a 50 years $\text{€}/\text{m}^2$
Organic coating	200	350	$45 + 81 = 126^b$	$45 + 81 + 45 + 25 = 196^b$
Sacrificial steel layer	5600	8600 ^c	16	$28 + 32 + 23 = 83^c$
Thermally sprayed aluminium	260	350	39^d	49^d
Cathodic protection – aluminium layer	260	350		
– sacrificial anode			$28^e + 14^e = 42$	$35^e + 23^e = 58$
CPSA				
Cathodic protection – aluminium layer	260	350		
– impressed current CPIC			$28^e + 28^e = 56$	$35^e + 45^e = 80$

^a Cost per m^2 means average cost per m^2 of a 13m high sheet pile

^b First value is cost per m^2 for application of coating before sheet pile driving; the full sheet pile has been coated. Other values are capitalized cost (see Section 6.8.2.1) for recoating after 15, 30 and 45 years of usage, respectively. These consist of the following investments per m^2 : €91 for the construction of a 'dry dock' in front of the sheet-pile wall, €11 for general costs, €73 for the coating plus application including protection against rain and condensation by canvas, and €20 environmental cost for removing the old coating. The total is €184/ m^2 . Only the upper 7.5 m is recoated

^c The sacrificial layer is not enough to last 50 years, but thicker steel sheet piles are not available in the market. After 20 years, a sprayed aluminium layer of 300 μm at the upper 5m (second value) and cathodic protection with a sacrificial anode on the under 9.5m (third value) is applied. The investments for this additional protection with aluminium after 20 years are assessed to be: cost of the temporary 'dry dock': €91, general costs: €11, aluminium application: €82. The total is €184

^d Only the upper 7m is thermally aluminized by spraying. The cost is averaged per m^2 over 13m

^e The upper 5m is aluminized by spraying (first value), the under 9.5m is cathodically protected (second value). The cost is averaged per m^2 over 13m. The cost per m^2 for CPSA is assessed at €20 and €32 and for CPIC at €38 and €61 for 20 and 50 years, respectively

Below these 5 m, cathodic protection using a sacrificial anode system (CPSA) is applied (see Section 2.5.5.3).

- 3 *Steel with an aluminium layer* on the front surface, with the exception of the section embedded in the soil (6 m). The thickness of the thermally sprayed aluminium is 260 μm for 20 years and 350 μm for the 50 year design service life.
- 4 *Steel with an above-water aluminium layer and underwater cathodic protection with sacrificial (CPSA) anode.* Thermally sprayed aluminium for the upper 5 metres with 260 μm for 20 years and 350 μm thickness of the aluminium for the 50 year design service life. Below the upper 5 metres, CPSA is also applied in the soil.
- 5 *Steel with an above-water aluminium layer and underwater cathodic protection with impressed current (CPIC).* Thermally sprayed aluminium for the upper 5 metres with 260 μm for 20 years and 350 μm thickness of the aluminium for the 50 year design service life. Below the upper 5 metres, CPIC is also applied in the soil.
- 6 *High-strength concrete C55/67 (EN 206), type A60016C* of 560 kg/m².
- 7 *Super high-strength concrete C200* of 240 kg/m².

The capitalized costs for the seven alternative sheet-pile walls (see Section 6.8.2.1) are given in Table 6.16. The real rate of interest applied is 4%. Figure 6.23 shows the total costs.

6.9 Environmental life-cycle assessment

Sustainable development is a key issue in many countries. The European building industry is responsible for about 40% of the total environmental impact. Environmental benefits in this industry will, therefore, make a substantial contribution to sustainability. On the basis of the ISO 14040 series on life-cycle assessment, standards are being developed to assess the environmental impacts from buildings. Life-cycle costing involves an approach that covers the whole lifetime. An ISO standard called *Sustainability in Building Construction – Framework of Assessment of Environmental Impacts from Buildings* will be available in the near future. For the building materials and components an ISO standard is being developed called *Sustainability in Building Construction – Environmental Product Declaration*. At the national level, methods have been developed on how to determine environmental impacts of materials, components and buildings in more detail.

Applying these methods results in environmental impacts and effects. The latter concerns, for instance, the contribution to greenhouse gases, smog, human toxicity and acidification. To simplify the use of those effects they are sometimes aggregated to one or a few eco-indicators. These

Table 6.16 Integral capitalized costs of seven alternatives for sheet-pile walls

Sheet-pile alternative	Cap. cost in €/m ² 20 year service life	Cap. cost in €/m ² 50 year service life
Steel-coated	110 ^a + 15 ^b + 126 ^c = 251	110 ^a + 15 ^b + 196 ^c = 321
Steel with sacrificial thickness	110 ^a + 16 ^b + 16 ^c = 142	110 ^a + 16 ^b + 83 ^c = 209
Steel with aluminium	110 ^a + 15 ^b + 39 ^c = 169	110 ^a + 15 ^b + 49 ^c = 174
Steel with above-water aluminium and underwater CPSA	110 ^a + 15 ^b + 42 ^c = 167	110 ^a + 15 ^b + 58 ^c = 183
Steel with above-water aluminium and underwater CPIC	110 ^a + 15 ^b + 56 ^c = 181	110 ^a + 15 ^b + 80 ^c = 205
Concrete C50/67	89 ^a + 36 ^b = 125	89 ^a + 36 ^b = 125
Super concrete C200 ^d	<278 ^a + 18 ^b = <296^e	<278 ^a + 18 ^b = <296^e

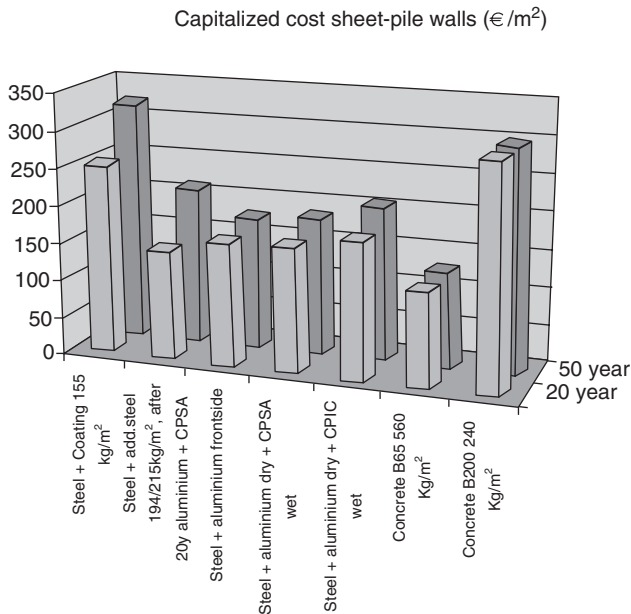
^a Cost of steel or concrete sheet-pile without protection

^b Cost of sheet-pile driving

^c See Table 6.15

^d A durability assessment over a 50 year period is not really possible at this moment

^e Material cost for a C200 concrete grade is likely to decrease in future

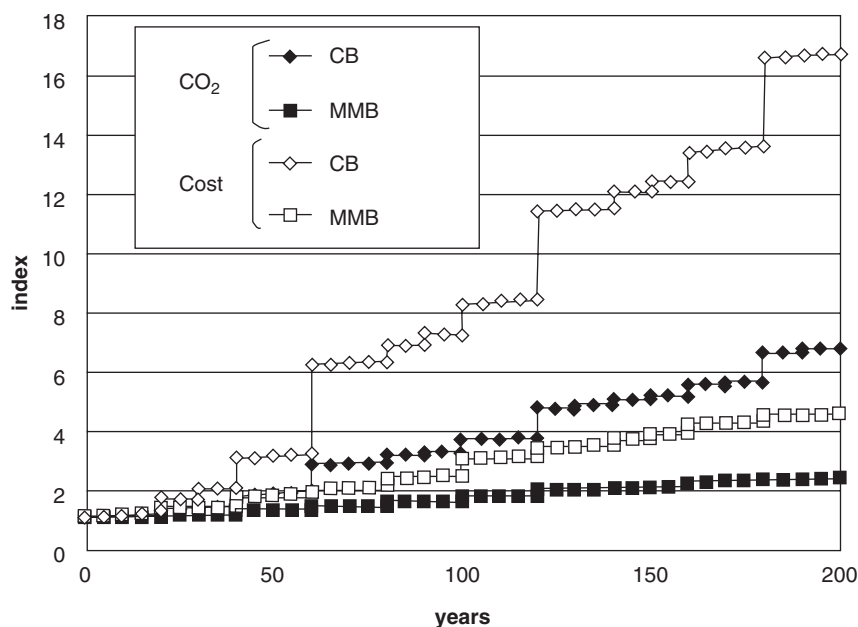


6.23 Capitalized costs of seven sheet-pile wall alternatives.

environmental effects or eco-indicators can be used to optimize the environmental burden of a structure.

The service life of structures and the components of those structures are dominant factors in the environmental impact. In general, replacement or repair of building components of structures during service life greatly enhances that impact.

In future, strategies for design and maintenance are likely to be influenced by optimization of the environmental impact. Figure 6.24 shows the cumulative values of the life-cycle costs and of carbon dioxide (CO_2) emissions for two bridges in a coastal area in Japan.²² One of the bridges is a conventional bridge (CB), the other one a so-called minimum maintenance bridge (MMB). Both are steel bridges with a reinforced concrete deck. For the MMB bridge a number of measures have been taken to decrease the necessity of maintenance, such as the application of a duplex protection system on the steel (galvanizing plus an organic coating), rubber bearings instead of steel, improved asphalt and coated reinforcement for the concrete deck. Obviously, the initial investments for the conventional bridge are substantially lower. However, life-cycle costs and CO_2 emissions are already substantially higher at an age of 25 years.



6.24 Comparison of conventional bridge (CB) with minimum maintenance bridge.²²

6.10 References

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7.1 Introduction

This final chapter discusses three case studies that are typical examples of practical problems. For two of these case studies, specification is requested of the most important requirements that structures have to meet in terms of durability. Durability must then be regarded in relation to reaching the Ultimate Limit State and the Serviceability Limit State over the design service life. This can be extended into other functional requirements, such as fire safety and energy performance requirements.

In the next section the targets that should be met in writing specifications are considered because this is an area in which student engineers are often deficient.

The case studies discussed are:

- Steel and concrete bridges in a warm marine environment.
- Repair of reinforced concrete floors.
- Wooden piles.

Disclaimer: The specifications given in this chapter are unsuitable for use without adjustment to the specific environment and the specific structure.

7.2 Specification

7.2.1 Targets

Targets contained in a specification must:

- Relate to a single performance measure of a particular output.
- Be expressed in a simple manner.
- Specify a time frame or milestone.
- Be specified for each measure selected.

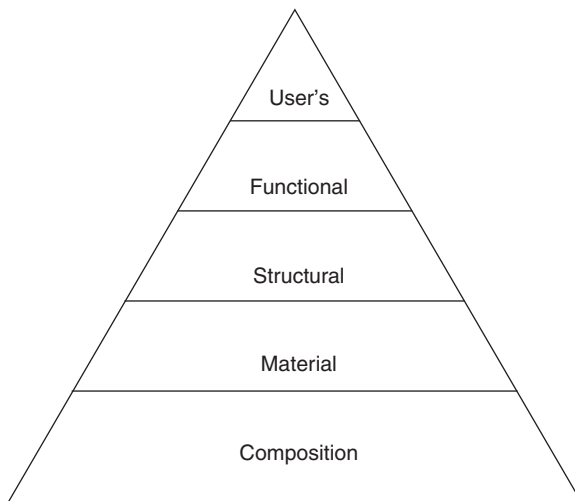
- Be measurable, for example, actual numbers and percentages should be stated. Terminology such as 'optimize', 'increase', 'decrease' or 'not much' should not be used.
- Mention the method of measurement (determination method) unless it is obvious, e.g. time.
- Be realistic and achievable but challenging.
- Be defined in terms that relate to the delivery of outputs.

By meeting these requirements the specifications will be:

- A clear guide to tenderers on minimum performance expected.
- A basis for comparing tenderers' offers.

7.2.2 Prescriptive or performance-based specifications

As discussed in Chapter 6, specifications can be prescriptive or performance-based. Performance-based specifications can be at the user's, functional, structural, material or composition level (see Fig. 7.1). At the composition level, requirements are prescriptive. Within specifications, one requirement can be prescriptive, another one performance-based.



7.1 Levels of specification.

7.3 Case study 1: steel and concrete bridges in a warm marine environment

7.3.1 Description, problem definition and instructions

Two bridges must be designed; they span creeks on the coast of Mumbai (Bombay) in India. One is a cable-suspended bridge and the other a cantilever bridge. In the suspension bridge, the pylons are manufactured from reinforced concrete and the bridge deck is made of steel. The cantilever bridge will be manufactured from prefabricated concrete segments of which the hollow piers and the superstructure are post-tensioned (it is a post-tensioned precast segmental bridge). The bridges should have a service life of 75 years without corrective (essential) maintenance.

Note: The aim of this case study is to draft material specifications concerning durability/service life of these bridges, as well as the maintenance required over a 75-year period.

Specific points are:

- To describe the relevant deterioration mechanisms.
- To write specifications for the following bridge components:
 - Cable-suspended: the concrete pylons, the suspension cables and the steel bridge-deck;
 - Cantilever: the piers and the superstructure.
- To specify required maintenance (frequency, type) for the steel bridge deck.

7.3.2 Relevant deterioration mechanisms

7.3.2.1 *Exposure conditions*

The exposure conditions of the bridges should be known so as to determine the relevant deterioration conditions. The climate of Mumbai can be characterized as a warm marine environment. It is warm and humid all year round; temperatures are relatively stable due to the moderating influence of the sea.

Because of the position of the bridges along the coast, we can assume that chloride loading will be high, drying-out of concrete will be limited and air pollution will be low.

7.3.2.2 *Concrete: pylons, cantilever bridge*

Based on Chapter 3 'Concrete', the following mechanisms are relevant to the concrete parts of the structures:

- Chloride-initiated corrosion of reinforcement in concrete in tidal, splash, spray and atmospheric zones (see Section 3.4.3).
- Salt weathering (see Section 3.2.3.4). It should be noted that due to generally high humidity, deterioration will be less than in warm arid marine climates, such as along the Gulf.

Attention should be paid to:

- Alkali–silica attack; however, this is only relevant if alkali-reactive aggregates are applied.

Not relevant are:

- Carbonation, because of high humidity.
- Sulphate expansion, because the sulphate salts formed in a marine environment, like ettringite, are non-expansive (see Section 3.2.3.1).

7.3.2.3 *Post-tensioning tendons*

For discussion on stress corrosion of post-tensioned tendons in the cantilever bridge see Section 3.4.8. Corrosion of tendons in post-tensioned precast segmental bridges is generally due to:

- Voids present within the grout used to fill the tendon ducts and protect the prestressing steel. Inadequate grouting may allow water to penetrate into the ducts, initiate corrosion of the tendons and potentially lead to failure of the structure.
- Failing corrosion protection of tendons where they pass through the bridge segment joints. For these joints, match-cast epoxy is usually applied.

Although global experience has been highly positive, a number of failures have been known to happen. Thus in the UK for instance, there was the sudden collapse of the Bickton Meadows footbridge in 1967 and of the Ynys-y-Gwas Bridge in 1985 due to corrosion of tendons. This, in addition to other documented problems with post-tensioned bridges led to a ban in 1992 on the construction of new bonded post-tensioned bridges by the UK Ministry of Transport. In 1996, the moratorium on grouted post-tensioned cast-in-place construction was lifted, but remains in force for segmental construction.¹

7.3.2.4 *Steel bridge deck*

The following deterioration mechanisms should be taken into account (they are based on the discussion in Chapter 2):

- Corrosion fatigue (see Section 2.3.2.12). This is by far the most frequently occurring problem in steel bridge decks.
- Uniform corrosion in unprotected steel (see Section 2.3.1).
- Pit corrosion can occur in the case of non-conductive protective layers, such as organic coatings (see Section 2.3.2.4).

7.3.2.5 *Suspension stay cables*

The following deterioration mechanisms are relevant (they are based on the discussion in Chapter 2)

- Stress corrosion (see Section 2.3.2.10).
- Hydrogen embrittlement (see Section 2.3.2.11).

7.3.3 Specifications for durability

7.3.3.1 *Concrete: pylons, cantilever bridge*

Prescriptive

Prescriptive specifications for the concrete in pylon and cantilever bridges is given in Table 7.1. In this specification, the contractor has only to make sure that the requirements are met in practice. If the specifications appear to be insufficient to safeguard durability, the designer is responsible.

NB: Possible legal action to claim indemnity from either designer or contractor greatly differs between countries.

Note: For concrete bridge decks in many countries polymer or bituminous membranes are applied beneath the bituminous asphalt to protect the concrete against ingress of chlorides. In the case of concrete meeting the specifications of Table 7.1 this measure is not required. In the Netherlands the Ministry of Transport, Public Works and Water Management sometimes prescribes treatment with a hydrophobic agent of the concrete surface before applying the asphalt top.³

Performance

A simple requirement at the user's level can be: 'The service life of the concrete structure must be 75 years. In this period, no major maintenance/repair will be required.' In this case, the design for durability is left to the contractor, who will be held responsible if the requirement is not met in practice.

Table 7.1 Prescriptive specifications for the concrete in pylon bridges and cantilever bridges

Subject	Requirement	Product or determination standard
cement	CEM III/B (EN 207) alternative silica fume addition to CEM I	EN 207 <i>Cement</i> and EN 206 <i>Concrete – Specification, Performance and Conformity</i>
cover w/c aggregates	>75 mm, optionally 50 mm under water <0.40 non-alkali reactive	CUR Recommendation 89 ²
chloride content curing	<0.05% of mass of cement keep concrete in the shade for at least 7 days, keep the concrete wet for at least 7 days, apply a curing compound after 7 days	curing compound
cracking	to prevent cracking, the temperature difference between the surface and the core of the concrete must be <10 °C ^a	
design 1	drainage systems on the bridge deck must be made in such a way that the bridge deck ceiling or piers shall not be contaminated when water and gutter discharge fall down	
design 2	auxiliaries, such as metal lamp posts and guiding rails, must not be in direct contact with the reinforcement or the prestressed steel	

^aThis specification is occasionally applied to prevent young concrete cracking but does not guarantee its absence

NB: It can of course be stated that, possibly 25 years after completion, the contractor will no longer exist or will be declared bankrupt, etc., once repair appears to be required. In such cases, insurance can be requested.

On a level that is less demanding for the contractor, it can be stated that so as to obtain approval by the resident engineer, the contractor must show that the service life of the structure will be 75 years according to, for instance, the DuraCrete method, as described in Section 6.5.5. Requirements at the structural level and the material level are given in Table 7.2.

Table 7.2 Performance specifications for the concrete in pylon bridges and cantilever bridges

Subject	Requirement	Product or determination standard
chloride penetration (of the concrete of the structure) cover	charge (coulomb) ≤ 1000	ASTM C1202 'electrical indication of concrete's ability to resist chloride penetration'
aggregates chloride content	>75 mm, optionally 50 mm under water non-alkali reactive $<0.05\%$ of mass of cement	CUR Recommendation 89 ^a
cracking	cracking of young concrete is not allowed	

^aReference 2

Not recommended

The following measures must not be selected:

- Sulphate-resistant Portland cement: this cement shows rapid chloride penetration, whereas there is no sulphate attack (see Section 3.4.3.2).
- Cover ≥ 100 mm: there is no need for it and a thicker concrete cover means more weight and will make it more difficult to prevent cracking.
- Cover ≤ 50 mm: this thickness will be critical, especially when vulnerability for curing is taken into account; 10 mm of the cover can be 'lost' easily due to poor curing.
- Coating on concrete: it will deteriorate rapidly and is not necessary (see Section 3.8.4.3).
- Coated rebars: not reliable (see Section 3.4.7.5).
- Stainless steel rebars: not necessary and expensive.

7.3.3.2 Cantilever bridge prestressing tendons

Prescriptive

- The prestressing steel should be delivered packaged in a waterproof wrapping. The package should include a siccative or a vapour-based inhibitor.
- Apply high-density polyethylene (HDPE) or polypropylene corrugated ducts according to References 3 and 4.

- Contact between the environment and cables during the construction stage is not allowed. This should be achieved by applying a protective gas, e.g. nitrogen.
- Grouts should fully embed the tendons.
- Composition of the grouts:
 - W/C grout <0.35 ;
 - Portland cement with silica fume (8% of mass of cement);
 - non-bleeding (segregation and separation of water is not allowed).
- Box-outs at tendon anchoring should be especially attended to with a view to chloride penetration, e.g. by coating the steel parts of the anchoring system and by applying epoxy bonding agents between the concrete, steel and the box-fill-in.

Reference can be made to the AASHTO Standard Specifications for Highway Bridges. Precast segmental bridges also fall under the jurisdiction of the AASHTO Guide Specifications for Design and Construction of Segmental Concrete Bridges. For grouting, reference can be made to the *Recommended Practice for Grouting of Post-tensioned Prestressed Concrete* of the Post-Tensioning Institute (PTI). Alternatively, reference can be made to FIP recommendations⁵ and guidelines.⁶

7.3.3.3 Steel bridge deck

Prescriptive

The protection system specified in Table 7.3 can be applied to the steel. A typical protection system applied in the Netherlands applied on steel bridge decks with heavy traffic is shown in Fig. 7.2 and specifications are given in Table 7.4.⁷

Performance

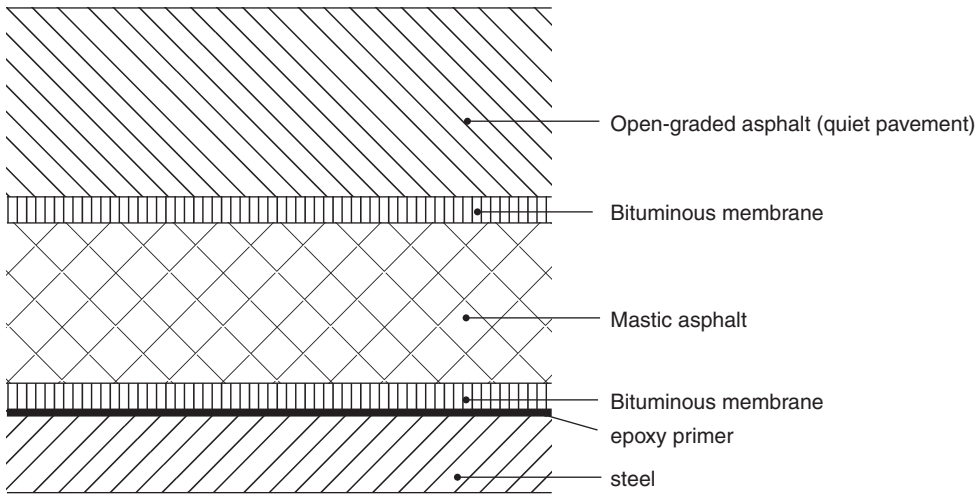
As for the concrete pylons and cantilever bridge, a performance specification can be at a functional level. The following is the author's proposed specification: 'The coating system will show no failures within 15 years, unless caused by accidents. Maintenance after 15 years will be restricted to the application of a new topcoat. Full recoating will not be required before a 30 year period has passed'.

More definitive is the specification method used by the Dutch Ministry of Transport, Public Works and Water Management, as discussed in Section 6.4.2.2, which includes:

- Proven performance in practice.
- Surviving representative accelerated ageing without damage, such as cracks and blisters.

Table 7.3 Prescriptive specification for steel protection of bridge deck in a cable-suspended bridge

Subject	Requirement	Product or determination standard
design considerations		ISO 12944-3
cleaning	Sa 2.5	ISO 8501-1
coating system	appropriate for Corrosivity Category C5-M Class C5-M 'very high (marine)' paint system meeting ISO standard	ISO 12944-2: 1998
on the bridge deck the coating has to be protected against a hot bituminous top	apply a bituminous membrane	ISO 12944-5: 1998(E)



7.2 Steel bridge-deck protection system applied in the Netherlands.⁵

- Meeting the requirement of a no-cracking forecast in a simulation computation using a coating behaviour model.⁸

Naturally the requirements listed above must be adjusted to the Mumbai climate.

Alternatives

- Galvanizing (or aluminizing) by spraying and coating (see Sections 2.5.4 and 6.8.4). Apply a thin high-strength concrete road deck to

Table 7.4 Typical prescriptive specification for steel protection of steel bridge deck with heavy traffic in the Netherlands⁵

Subject	Requirement	Product or determination standard
cleaning	Sa 2.5	ISO 8501-1
primer epoxy		
adhesive bitumen	50–100 µm 125 g/m ²	(Dutch) RAW standards (Standaard RAW bepalingen)
bituminous membrane	4 mm thick, fibre reinforced, SBS modified	
mastic asphalt		

dissipate wheel loads in order to decrease the probability of fatigue corrosion.

Not recommended

The following protective measures are not recommended:

- Cathodic protection, because conduction in the bridge deck is insufficient.
- Hot dip galvanizing, simply because the bridge deck is too large for dipping.

7.3.3.4 Stay cables

Prescriptive

- The strand will be hot dipped galvanized before the last drawing operation (e.g. 180 g/m < zinc coating > 350 g/m).
- It will then be factory sheathed with a tight high-density polyethylene sheet (e.g. >1.5 mm).
- A petroleum wax fills all the inter-wire voids and the interface strand-sheath (e.g. minimum quantity of wax protection = 12 g/m). Alternatively, a high-density polyethene pipe is applied, which is filled with a cementitious grout.⁵
- A smooth duct of a high-density polyethylene (HDPE) shall be applied to protect the cable.⁴

Not recommended

- PVC should not be applied because released chlorides where there is fire can promote corrosion.

- Cathodic protection is not an option due to failing conductance, although galvanizing provides a kind of cathodic protection.

7.3.4 Maintenance

7.3.4.1 *Concrete structures*

Although concrete is designed in such a way that preventive or corrective maintenance should not be necessary, it is, naturally, unwise to refrain from any inspection because errors could have been made in both the design and execution of the works. It is generally recommended to carry out a visual inspection, once a year in the first two years and once every two years after that. If any defect is observed, it should be examined in greater depth. If the frequency of defects increases, the inspection rate should be increased (see discussion on ‘the bath-tub curve’ in Section 6.7.4). Vulnerable parts, such as expansion joints and the applied expansion joint bridging systems, must be inspected each year. If there are defects in the system, e.g. failing moving parts, action must be taken (applying new grease or replacing parts). Bearing plates also need yearly inspections.

7.3.4.2 *Steel bridge deck*

- Stresses and strains in the bridge deck should be monitored regularly. This can be done continuously by applying strain gauges. Stresses and strains should stay within the serviceability limit state.
- Visual inspection of defects in the applied coating system; frequently in the first year and once a year after that.

7.4 Repair of reinforced concrete floors

7.4.1 Description, problem definition and questions

Part 1

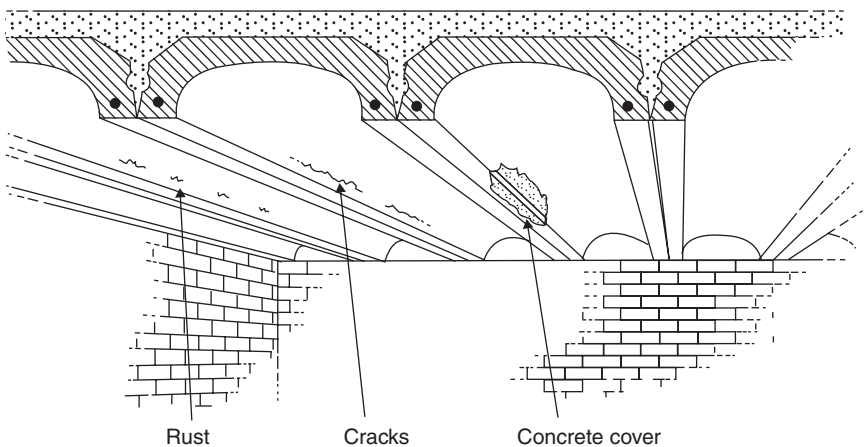
A structural engineer and concrete maintenance expert are requested to investigate the ground floor of a school. The floor is a Kwaaitaal floor. A preliminary inspection by the maintenance department of the school has disclosed that there is corrosion of reinforcement. A procedure is to be drafted to investigate the damage and make a diagnosis based on the investigation.

Part 2

Following the investigation, it is concluded that neither structural safety nor serviceability is endangered within a 10 year period, but beyond that period this state is no longer warranted because of the ongoing corrosion process. However, the school has to be in service for a minimum period of 25 years. The possibilities to safeguard the structural safety and serviceability over a period of at least 25 years will be discussed and a motivated choice made between two methods.

7.4.2 Background

Kwaaitaal precast reinforced concrete floors were manufactured in the Netherlands between 1965 and 1982. A typical cross-section of the floor with the damage observed is given in Fig. 7.3. A typical precast floor segment consists of two ribs. Calcium chloride was mixed into the concrete to accelerate cement hardening. The main problem was that the calcium chloride was applied in the form of flakes, which did not mix in well, leading to a very inhomogeneous chloride distribution throughout the concrete. This differential concentration has caused a highly corrosive situation. Typical corrosion phenomena are shown in Fig. 7.4. Because the Kwaaitaal precast floors and a competitive system called Manta, with the same problem, had been used on a massive scale in dwellings, schools and supermarkets, the size of damage was substantial. This prompted the Dutch government to investigate the problem in depth. The results are given in References 9 and 10.



7.3 Cross-section of Kwaaitaal floor with type of damage observed.



7.4 Photographs of damage of Kwaaitaal floors in crawl space.

7.4.3 Procedure for investigation and diagnosis

The following procedure, based on References 9 and 10, can be applied:

Inspection

It is assumed that the crawl space below the floor is accessible and visual inspection is possible.

The corrosion is classified according to Table 7.5.¹⁰ Each rib is divided into three zones in this classification: two outer parts of one metre each and the middle section. Each rib zone is classified according to Table 7.5.

Technical investigation

If the inspection indicates that the structural safety or serviceability is possibly endangered, a further technical investigation should be performed. This comprises:

- Design drawings and calculations of the floor, if available.
- Specific size of the floor, including span length.
- Position of pipes, installations etc.
- The support of the floor: free, clamped?
- Loads deviating from the design, e.g. an additional wall built on the floor.

Structural investigation

On the basis of the technical investigation and the results of the inspection, the remaining structural safety must be calculated. On the basis of

Table 7.5 Classification of damage per rib zone according to Reference 10 for maximal 4m span length

Classification corrosion	Damage class per rib zone		
	Local (<20%)	Substantial (20–50%)	General (>50%)
initial: rust spots	1	2	3
advanced: light cracking, local loss of concrete ^a	4	5	6
final phase: severe cracking, substantial loss of concrete	7	8	9

^aconcrete that fell down due to corrosion of the rebars

Table 7.6 Classification of damage and time span of action

Classification of damage	Time span of action required
• in fields of six adjacent ribs: – An average decrease of the rebar cross-section of more than 15% – Two or more adjacent ribs: damage class 7–9 and load-bearing capacity <30%: • three adjacent ribs with class 7–9 and remaining load capacity 30–60% • one of the outer floor fields (two ribs) shows damage class 7–9 • floor with ribs of a damage class not exceeding 4–6 • floor with ribs of a damage class not exceeding 1–3	immediate immediate immediate immediate within 5 to 10 years no time limit, regular inspections

structural calculations on Kwaaitaal floors with a span not exceeding 4m, as given in References 9 and 10, the structural damage can be classified as a rule of thumb assessment using the classification given in Table 7.6.

7.4.4 Possibilities to extend the service life by 25 years

From the investigation described in the previous section, it is concluded that expectations are that the floor is still structurally safe and that the serviceability during a ten year period will not be endangered. However, it is expected that beyond the ten year period, this will no longer be the case.

Note: In making this conclusion it is assumed that the environmental conditions, notably humidity, in the crawl space will not change.

However, the school has to be in service for another 25 years. Within ten years, therefore, remedial action (preventive, reactive maintenance) will be required. There are several methods to extend the service life. They are detailed below.

Replacement

Replacing the ground floor by a new floor (Method 3.4, Table 3.6). Obviously, the ground floor will be out of operation during this operation.

Structural strengthening methods

- Placing prestressing tendons underneath the floor elements and post-tensioning them (Method 4.7, Table 3.6), see Fig. 7.5. The tendons are prestressed at the school walls above the foundations and sometimes these foundations have to be reinforced. The walls act as a rim beam for the tensioning.
- Placing support beams (steel, aluminium, wood) underneath the floor. The beams are supported by making holes in the walls under the floor.
- Placing support beams in the crawl space on specially manufactured foundations.



7.5 Prestressing tendons underneath Kwaaitaal floor.

- Filling up the crawl space with a lightweight material, such as foamed concrete. However, this results in the disadvantage that the crawl space will not longer be accessible, which is not always acceptable.

Structural repair

- The reinforcement in the rib is removed and replaced. Temporary or lost shuttering is replaced and a replacing grout/concrete pumped into the shutter (Method 3.2, Table 3.6 and Method 7.2, Table 3.7).
- The same reinstatement of the reinforcement as previously discussed, but now the concrete is replaced by spraying (Method 3.3, Table 3.6. and Method 7.2, Table 3.7). Because of the narrow crawl space, this is not a realistic option.

Stopping the corrosion

- Cathodic protection with impressed current (Method 10.1, Table 3.7). Spalled and cracked concrete must be removed and replaced by a conducting mortar and the anode system installed (see Section 3.9).
- Cathodic protection with sacrificial anode (Method 10.1 Table 3.7). A zinc layer is glued or sprayed onto the concrete surface (e.g. 3M™ Zinc Hydrogel Anode 4727). Spalled and cracked concrete must be removed and replaced by a conducting mortar (see Section 3.9).
- Lowering the relative humidity of the crawl space by covering the soil with a vapour barrier (shells, granular materials, foils, foamed concrete), by installing a ventilation system and/or lowering the ground water level (Principle 8, Table 3.7). The relative humidity must be decreased to a level below 65%, which is difficult to manage. Moreover, decreasing the humidity will increase the carbonation rate and because the concrete quality is low this can proceed quickly. A combination of carbonation and chloride contamination can be deleterious because of the release of bound chloride, as discussed in Section 3.4.3.1.
- Applying a coating to prevent condensation and wetting of the ceiling of the concrete floor. The concrete needs to be dry before applying the coating (Method 2.2, Table 3.6). This method is also difficult to control.
- Applying an insulation layer, e.g. sprayed PUR. The same problem then occurs as in the previous method of coating. In addition, it appears this type of insulation is not vapour tight.

7.4.5 Choice of two methods of restoration

The floor is still structurally safe according to valid in-place design methods and there is, therefore, no need to replace it nor to reinstate the initial level of structural safety. It is sufficient simply to stop corrosion. Within the

methods to stop corrosion, as discussed in the previous section, cathodic protection is the most reliable. Removing spalled and cracked concrete and replacing it with a well-conducting cementitious mortar and subsequently gluing a sacrificial zinc foil anode over the ribs is the simplest method to apply in the narrow crawl space (see Fig. 7.6.). However, there are some doubts with respect to the application and the durability of cathodic protection for this case. Removing and replacing of concrete in the narrow crawl space is difficult. There is no long term experience with the durability of the adhesive of the zinc foil and the effect of possible changes in humidity are not known. Therefore, in the Netherlands, structural strengthening is usually the preferred option. This does not stop corrosion, but provides another load-bearing capacity that can replace the function of the rebars in the Kwaaitaal floors.

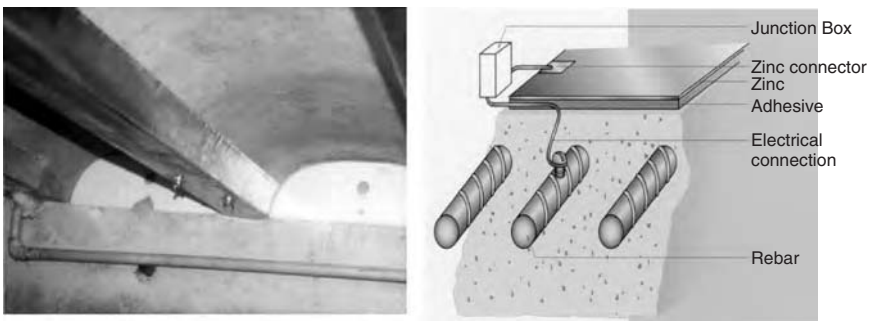
The method of prestressing tendons is frequently applied in the Netherlands. The tendons are applied parabolically; they give an upward force on the floor that compensates the load. Points to which attention should be paid are:

- The loads in combination with the tension on the tendons should not cause too high tension stress in the walls (rim beams).
- Upward stresses due to the tensioning system should be counterbalanced by the floor load.
- The ends of the floor elements should be well-clamped structurally.
- Cold transportation can occur at the joint of the floor and the outer wall of the school, where the tendons are anchored.

7.5 Wooden piles

7.5.1 Description, problem definition and questions

A housing corporation requests wooden foundation piles to be used within the framework of sustainable building. However, during the last few years

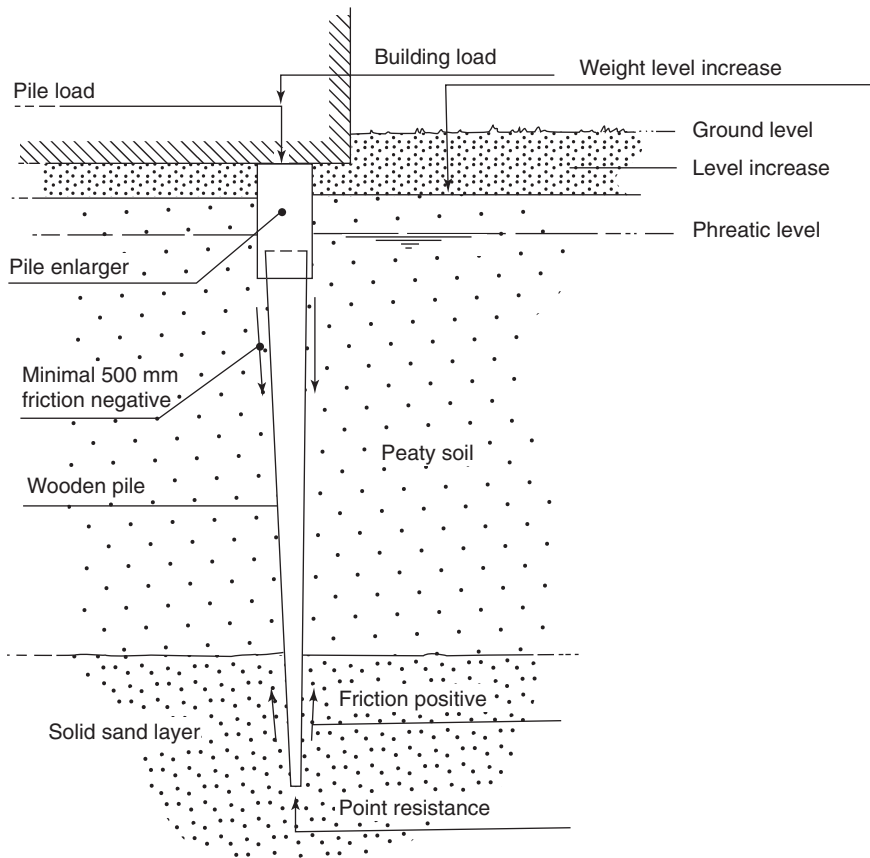


7.6 Zinc foil with glue and conducting layer (courtesy 3M™).

there have been several reports on wooden piles that unexpectedly showed decay, causing severe damage to houses. The houses requested must be built on a peaty soil and the foundation piles must be driven through the peat into a 10m deep sand layer. The ground water table varies yearly by about half a metre (see Fig. 7.7 as illustration). The aim of this case study is to draft specifications for the piles based on the requirement that the piles must not show significant deterioration within a 75 year period.

Specific aspects to be considered are:

- To describe the relevant deterioration mechanisms of the wooden piles.
- To describe the measures to be taken to avoid deterioration.
- To write specifications for the wooden piles with respect to the required service life.



7.7 Diagram of pile with enlarger in peaty spoil driven into a sand layer.

7.5.2 Relevant deterioration mechanisms

Relevant deterioration mechanisms are:

- Decay by fungi (see Section 4.2.1). However, beneath the lowest water table, the piles will not be affected.
- Attack by bacteria (see Section 4.2.4). In general, this is a slow process, but this deterioration process can occur even below the water table and, depending on the wood species, can cause serious problems. These problems are observed in the sapwood of Scotch fir (*Pinus sylvestris*) (French: sapin ecossaise, German: Scottische Tanne, Dutch: grenen) in particular.

7.5.3 Prevention measures

- Attack by fungi can be avoided by applying the wood below the water table only. In the Dutch standard NEN 6711, Article 6.2, it is prescribed that the top of the wooden pile should be at least 400mm below the lowest ground water table. In practice, designers usually situate it at 500mm below.

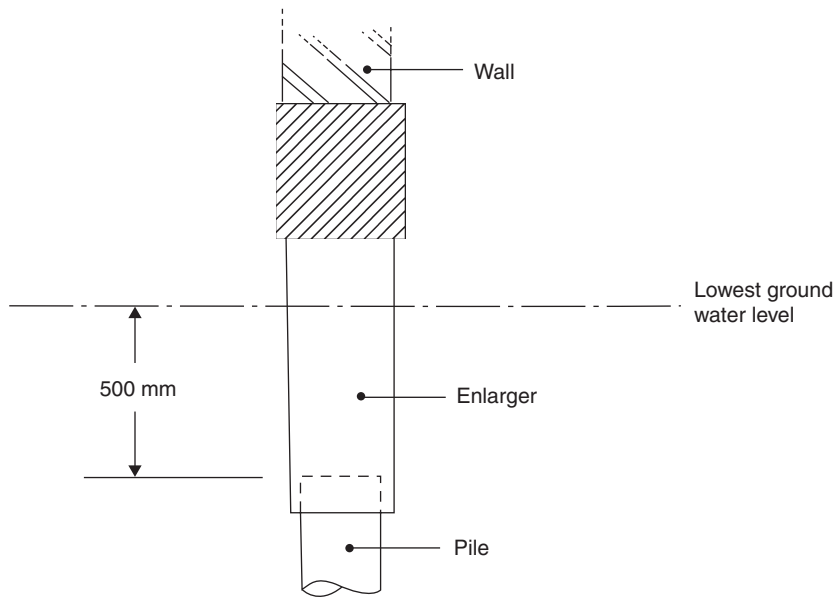
In general, a concrete pile-enlarger (Dutch: oplanger) is applied on top of the wooden pile, which bridges the zone with the fluctuating water table (see Fig. 7.8). Alternatively, wooden enlargers of Durability Class 1 or 2 according to EN 350 (Table 4.2.) can also be applied

- To avoid deterioration by bacteria, the following wood species appeared to be resistant, provided the sapwood content is minimized:
 - Norway spruce (*Picea abies*) (French: épicea, German: Fichte, Dutch: vuren);
 - Japanese larch (*Larix leptolepis*), (French: mélèze, German: Lärche, Dutch: lariks);
 - Douglas fir (*Pseudotsuga menziesii*);
- Alternatively, modified wood can also be applied, if available (currently not available at pile length) (see Section 4.3.5).

Not recommended

Not to be applied are:

- Painted wood or wood treated with water-repellents, because of damage during pile driving.
- Wood protected with oily preservatives, such as creosote or water-borne toxic salts, because the request specified sustainability and, moreover, it is not necessary.



7.8 Pile with enlarger.

Table 7.7 Specifications for durability of wooden piles

Characteristic	Specification
wood species	Norway spruce, fir; Japanese larch or Douglas fir; minimize sapwood content!
wood ground water	meet requirements of BRL 2302 (wooden piles) ^a wooden piles should be at least 500 mm below lowest ground water level
concrete enlargers	meet requirements for concrete given in Dutch standard NEN 7053 <i>Concrete Piles</i>
wooden enlargers	according to EN 350

^aBRL is an abbreviation for guide for technical approval in the Netherlands

- Tropical hardwood: not necessary, expensive and not sustainable, unless harvested sustainably.

7.5.4 Specifications

Prescriptive

Prescriptive specifications are given in Table 7.7.

7.6 References

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