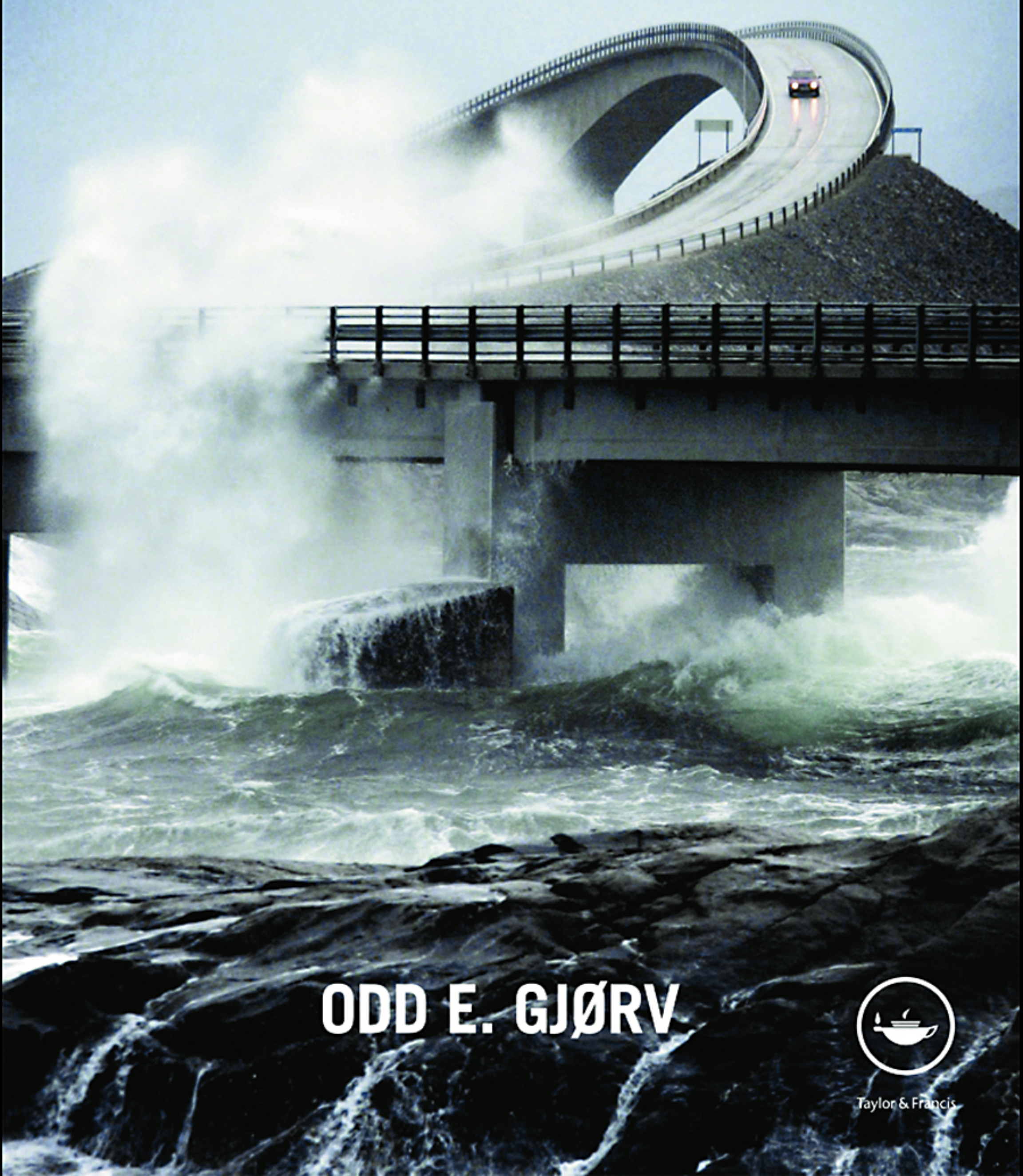
DURABILITY DESIGN OF CONCRETE STRUCTURES IN SEVERE ENVIRONMENTS



ODD E. GJØRV

Taylor & Francis

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Preface

Concrete structures in severe environments include a variety of different types of structure in various types of environment. In addition to all the important concrete infrastructures which are severely exposed to aggressive environments from de-icing salts, most of the extensive experience on concrete structures in severe environments is related to the marine environment. Of the total surface area of the globe, about 70 per cent is covered by ocean water, which means that the area of the oceans makes up about two and a half times the area of the land. When considering the even smaller part of the land area which is inhabitable and increasingly more populated, this indicates an increasing need in the years to come of moving even more activities into the ocean waters and marine environments. This increasing need may be briefly characterized by a few keywords such as raw materials, energy, transportation and space.

Already in the early 1970s, the American Concrete Institute (ACI) came up with a technological forecasting on the future use of concrete, where the rapid development on the continental shelves was pointed out. In this report, not only activities and structures related to oil and gas explorations but also activities and structures that would relieve land congestion were discussed. At an international symposium on concrete sea structures organized by Fédération Internationale de la Précontrainte (FIP) in Tibilisi in 1972, a great variety of new technological solutions to meet this development were discussed. Without going into details, it should be noted that a variety of concrete structures would play an increasing role as a basis for the further activities in ocean and marine environments. Such structures would be of widely differing types and categories, such as:

- non-anchored free-floating structures, e.g. ships, barges and containers;
- anchored structures floating at water surface level, e.g. bridges, operation platforms, moorings, energy plants, airports and cities;
- anchored structures (positive buoyancy) resting above seabed level, e.g. tunnels and storage units;

- bottom-supported structures (negative buoyancy) resting at or below seabed level, e.g. bridges, harbour structures, tunnels, buildings, storage units, caissons, operation platforms and energy plants.

To a great extent the above rapid development has taken place and still will for many years to come; concrete will be the cheap and easily available construction material which can be provided in large quantities. It is well known that the properties of this material can be varied within wide limits. Thus, the density can be varied from 500 up to 4500 kg/m^3 if necessary, either from a buoyancy or structural point of view, while a compressive strength of up to more than 100 MPa can be produced. Experience has also shown that concrete structures in severe environments can remain serviceable for a very long time provided that current knowledge and experience are properly utilized.

Upon completion of new concrete structures, however, experience has shown that the achieved construction quality always shows a high scatter and variability, and in severe environments any weakness in the concrete structure will soon be revealed whatever its constituent materials may be. Therefore, a probability-based approach to the durability design is very important. Since much of the current durability problems may also be related to an absence of proper quality control as well as special problems during concrete construction, the issue of construction quality and variability must also be firmly grasped before any rational approach to a more controlled durability can be achieved. Hence a performance-based concrete quality control during concrete construction with proper documentation of achieved construction quality is also very important.

For a very long time, huge amounts of money and natural resources have been spent on repairs and rehabilitation of concrete structures in severe environments, and this is primarily due to premature corrosion of embedded steel. In recent years, therefore, a rapid development of more advanced procedures for durability design and concrete quality control during concrete construction has taken place, some current experience with which is outlined and discussed in this book.

In recent years, an increasing number of owners have realized that even small additional expenses in order to obtain an increased and more controlled durability are a very good investment compared to that of the rapidly increasing costs for the maintenance and restoration of structural capacity at a later stage.

As a rapidly increasing number of new important concrete infrastructures are being produced, an increased and more controlled durability is not only a technical and economical issue, but also a very important and increasingly more important environmental and sustainability issue, as is also outlined and discussed in this book.

Through my work in obtaining an improved and more controlled durability of new concrete structures in severe environments, I would like to acknowledge a number of my doctoral students from recent years who have been working with various aspects of the durability of concrete structures and contributed to various parts of the procedures both for durability design and concrete quality control as outlined and discussed in this book. These include Tiewei Zhang, Olaf Lahus, Arne Gussiås, Franz Pruckner, Surafel Ketema Desta, Miguel Ferreira, Öskan Sengul, Guofei Liu and Vemund Årskog.

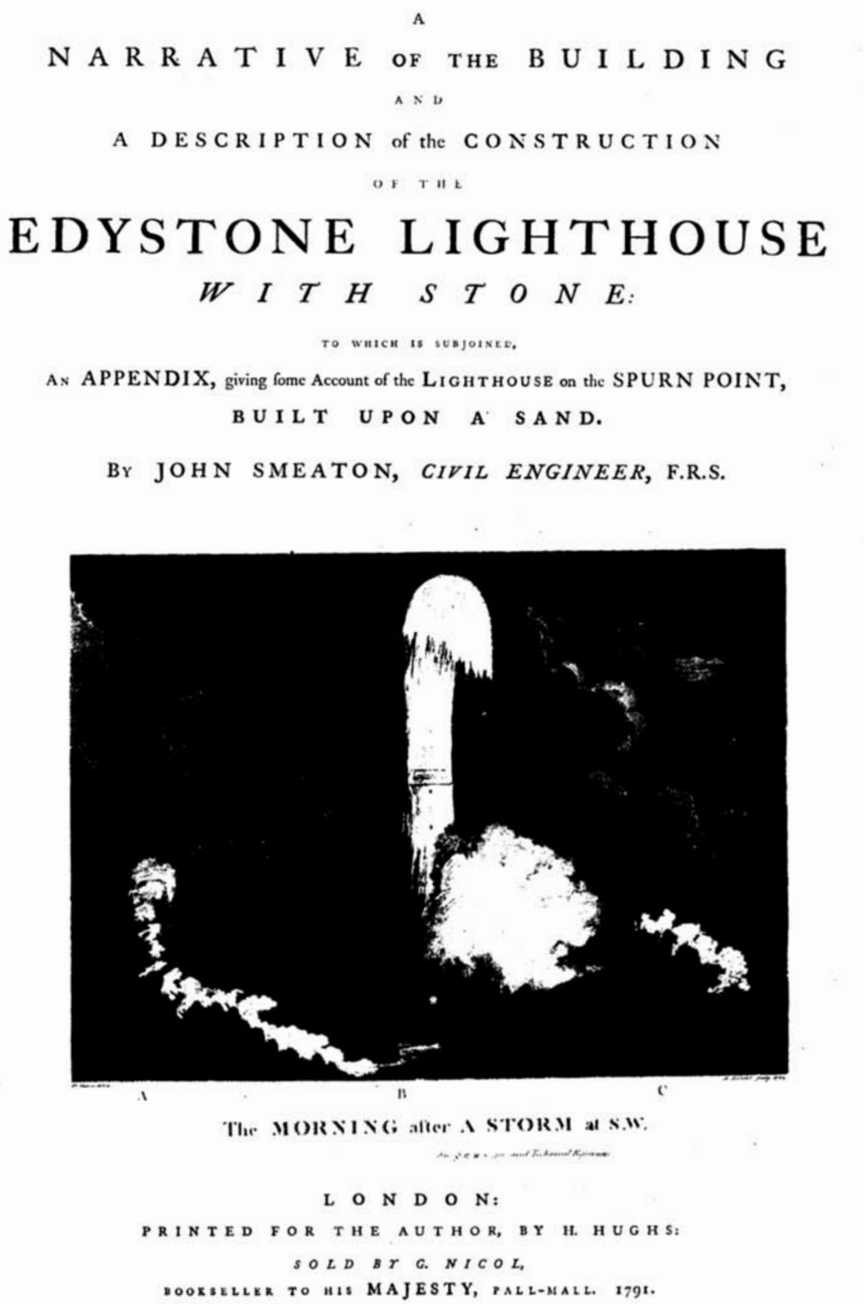
I would also like to thank Roar Johansen of the Norwegian Coast Directorate and Tore Lundestad of the Norwegian Association for Harbour Engineers for their excellent research cooperation and their great interest and encouragement to practically trying out and applying the new knowledge to concrete structures produced in Norwegian harbours over recent years. The opportunity to publish current results and experience gained from the durability design and concrete quality control carried out on recent projects for both Oslo Harbour KF and Nye Tjuvholmen KS in Oslo is also greatly appreciated.

Odd E. Gjørv
Trondheim, June 2008

1 Historical review

When Smeaton constructed the famous lighthouse on the Eddystone Rock at the outlet of the English Channel during the period 1756–1759 (Smeaton, 1791), this was the first time a specially developed type of cement for a severe marine environment was applied (Lea, 1970). When the structure was demolished due to severe erosion of the underlying rock in 1877, this structure had remained in very good condition for more than 100 years. Since Smeaton reported his experience on the construction of this lighthouse (Figure 1.1), all the published literature on concrete in marine environments makes up a comprehensive and fascinating chapter in the long history of concrete technology. During the past 150 years, a number of professionals, committees and national authorities have been engaged in this issue. Numerous papers have been presented to international conferences such as the International Association for Testing Materials in Copenhagen 1909, New York 1912 and Amsterdam 1927; the Permanent International Association of Navigation Congresses (PIANC) in London 1923, Cairo 1926, Venice 1931 and Lisbon 1949; the International Union of Testing and Research Laboratories for Materials and Structures (RILEM) in Prague in 1961 and 1969; the RILEM-PIANC in Palermo in 1965; and the Fédération Internationale de la Précontrainte (FIP) in Tibilisi in 1972. Already in 1923, Atwood and Johnson (1924) had assembled a list of approximately 3000 references, and durability of concrete structures in marine environments continues to be the subject for research and international conferences (Malhotra, 1980, 1988, 1996; Mehta, 1989, 1996; Sakai *et al.*, 1995; Gjørv *et al.*, 1998; Banthia *et al.*, 2001; Oh *et al.*, 2004; Toutlemonde *et al.*, 2007).

Although deteriorating processes such as freezing and thawing and expansive alkali reactions as well as chemical attack also represent some severe challenges, extensive experience has demonstrated that it is not the disintegration of the concrete itself but rather the electrochemical corrosion of embedded steel which poses the most critical and greatest threat to the durability and long-term performance of concrete structures in severe environments (Gjørv, 1968, 1975, 1989, 2002). This is confirmed by



A

NARRATIVE OF THE BUILDING

AND

A DESCRIPTION of the CONSTRUCTION

OF THE

EDYSTONE LIGHTHOUSE

WITH STONE:

TO WHICH IS SUBJOINED,

AN APPENDIX, giving ſome Account of the LIGHTHOUSE on the SPURN POINT,

BUILT UPON A SAND.

BY JOHN SMEATON, *CIVIL ENGINEER*, F.R.S.

LONDON:

PRINTED FOR THE AUTHOR, BY H. HUGHS:

SOLD BY G. NICOL,

BOOKSELLER TO HIS MAJESTY, PALL-MALL. 1791.

Figure 1.1 Front page of the report on the construction of the Eddystone Lighthouse written by John Smeaton in 1791 (source: Courtesy of British Museum).

extensive literature, and already in 1917 this was pointed out by Wig and Ferguson (1917) after a comprehensive survey of concrete structures in US waters.

In addition to conventional structures such as bridges and harbour structures, reinforced and prestressed concrete has increasingly been applied to a large number of very important ocean structures and vessels. This development was foreseen in the technological forecasting published by ACI already in the early 1970s (ACI, 1972). This forecasting pointed out the great potential for utilization of concrete for marine and ocean applications in general and for offshore oil and gas exploration in particular.

In Norway where most of the offshore concrete construction has taken place so far, long traditions have existed on the utilization of concrete in the marine environment. Already in the early 1900s, the two Norwegian engineers Gundersen and Hoff developed and obtained their patent on the 'tremie method' for underwater placing of concrete during the building of the Detroit River Tunnel between the USA and Canada (Gjørv, 1968). From 1910, when Gundersen came back to Norway and became the director of the new contracting company AS Høyer-Ellefsen, his new method for underwater placing of concrete became the basis for the construction of a new generation of piers and harbour structures all along the rocky shore of the Norwegian coastline (Gjørv, 1968, 1970). These structures typically consist of an open reinforced concrete deck on top of slender, reinforced concrete pillars cast under water. Although the underwater cast concrete pillars have been successively replaced by driven steel tubes filled with concrete, this open type of concrete structure is still the most common harbour structure being constructed along the Norwegian coastline (Figure 1.2).

Due to its very long and broken coastline with many fjords and numerous inhabited islands, Norway has a long tradition for use of concrete as a construction material in coastal environments (Figure 1.3). For many years, this included primarily concrete harbour structures. Successively, however, concrete also played an increasing role as a construction material for other applications such as strait crossings (Klinge, 1986; Krokeborg,

Figure 1.2 Open concrete structures are still the most common type of harbour structure built along the Norwegian coastline.

Figure 1.3 Along the Norwegian coastline with its many deep fjords and numerous small and large islands, there are a number of both on-shore and offshore concrete structures (source: Courtesy of NOTEBY AS).

Figure 1.4 The Tromsø Bridge (1960) is a cantilever bridge with a total length of 1016 m (source: Courtesy of Johan Brun).

1990, 1994, 2001). In addition to conventional bridges (Figure 1.4), new concepts for strait crossings such as floating bridges (Figures 1.5 and 1.6) also emerged (Meaas *et al.*, 1994; Hasselø, 2001). Even submerged concrete tunnels have been the subject of detailed studies and planning, and one of several types of design is shown in Figure 1.7 (Remseth, 1997; Remseth *et al.*, 1999).

The rapid development which took place later on the utilization of concrete for offshore installations in the North Sea is well known (Figures 1.8 and 1.9). Thus, since 1973, altogether 34 major concrete structures containing more than 2.6 million m^3 of high performance concrete have been installed (Figure 1.10), most of which have been produced in Norway. In addition, in other parts of the world a number of offshore concrete structures have been produced, and so far a total of 50 various types of offshore concrete structures have been installed (Moksnes, 2007).

For the first offshore concrete platforms in the early 1970s, it was not so easy to produce high strength concrete which was also required to contain a large amount of entrained air for ensuring proper frost resistance. Extensive research programmes were carried out, however, and the quality of concrete and the specified design strength successively increased from project to project (Gjørv, 2008). Thus, from the Ekofisk Tank which was installed in 1973, to the Troll A Platform installed in 1995, the design strength successively increased from 45 to 80 MPa. In addition, the water

Figure 1.5 The Bergsøysund Bridge (1992) is a floating bridge with a total length of 931 m (source: Courtesy of Johan Brun).

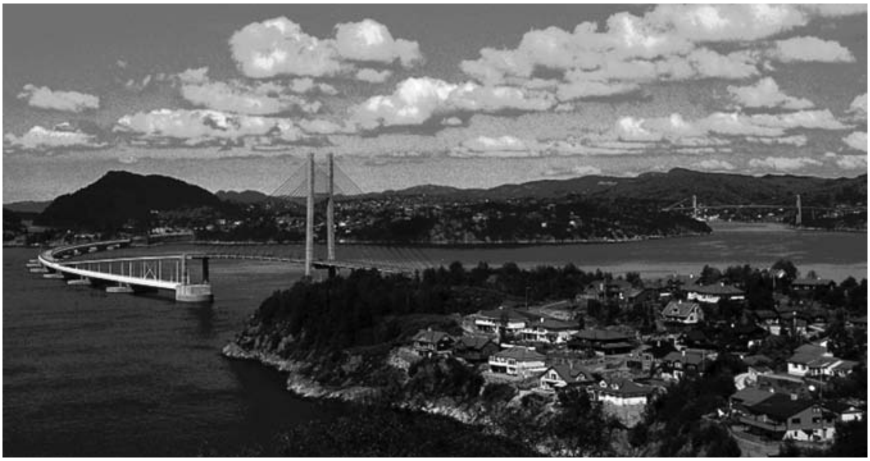


Figure 1.6 The Nordhordlands Bridge (1994) is a combined floating and cable-stayed bridge with a total length of 1610 m (source: Courtesy of Johan Brun).

depths for the various installations successively increased. Thus, in 1995, the Troll A Platform was installed at a water depth of more than 300 m. From the tip of the skirts to the top of the shafts of this gravity-base structure, the total height is 472 m, which is taller than the Empire State Building in New York. The artistic view in Figure 1.11 also demonstrates the

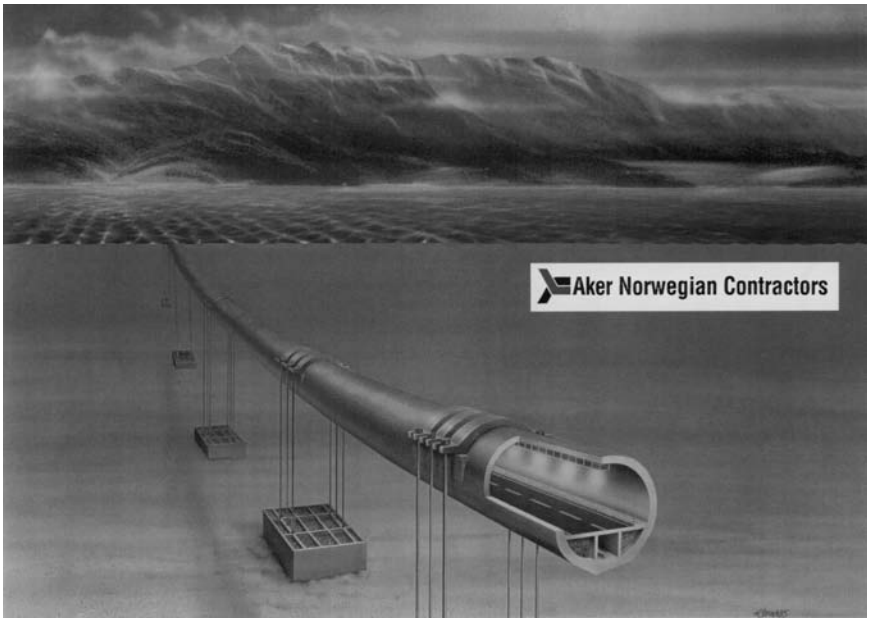


Figure 1.7 One of several designs studied for possible strait crossings by the use of submerged tunnels (source: Courtesy of Norwegian Public Roads Administration).

Figure 1.8 The first offshore concrete platform, the Ekofisk Tank, on its way out of Stavanger in 1973 (source: Courtesy of Norwegian Contractors).

Figure 1.9 The Gullfaks C Platform (1989) during construction in Stavanger (source: Courtesy of Norwegian Contractors).

size of this structure. After production in one of the deep Norwegian fjords (Figure 1.12), the Troll A Platform containing 245,000 m^3 of high strength concrete, 100,000 t of reinforcing steel and 11,000 t of prestressing steel was moved out to its final offshore destination, and this operation was the biggest movement of a man-made structure ever carried out (Figure 1.13). In 1995, the Heidrun platform was also installed in deep water at a depth

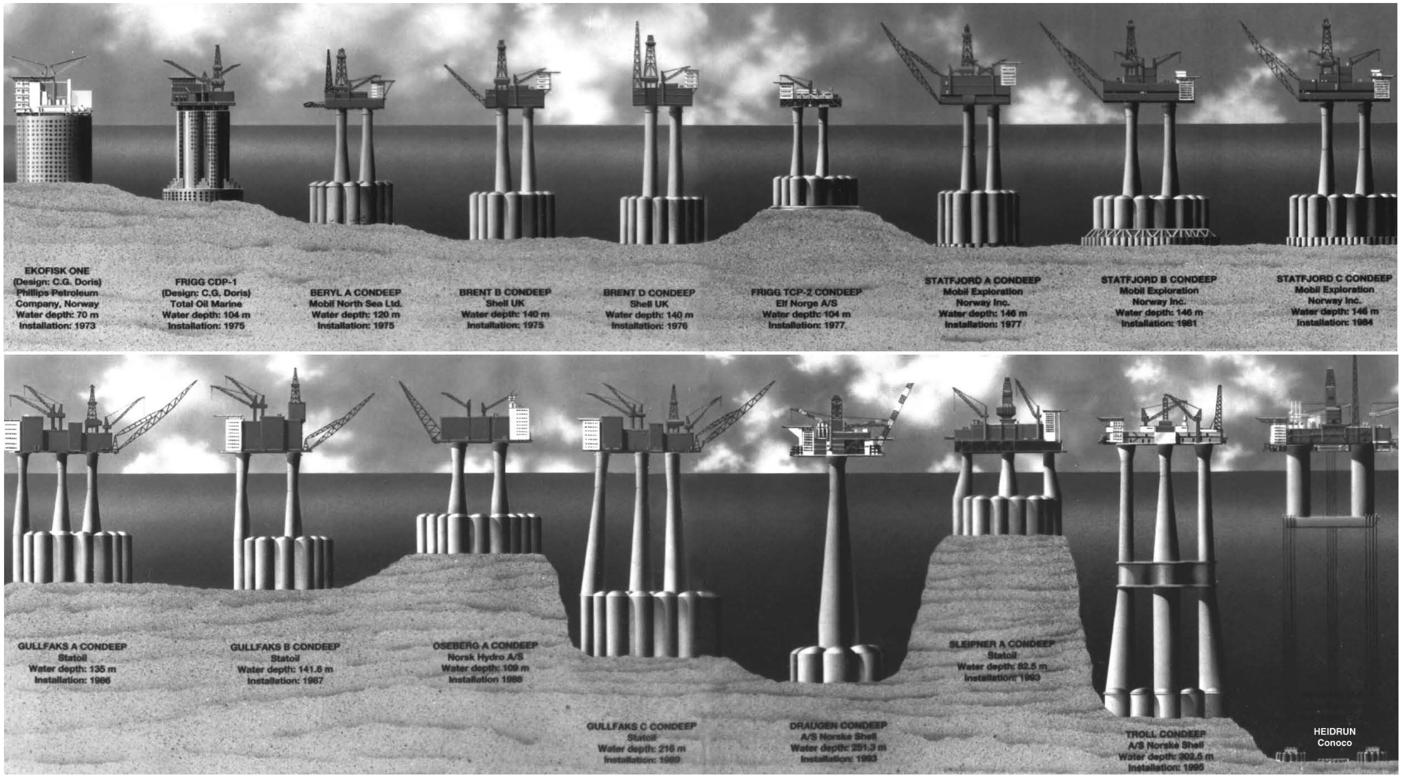


Figure 1.10 Development of offshore concrete structures in the North Sea (source: Courtesy of Aker Solutions).

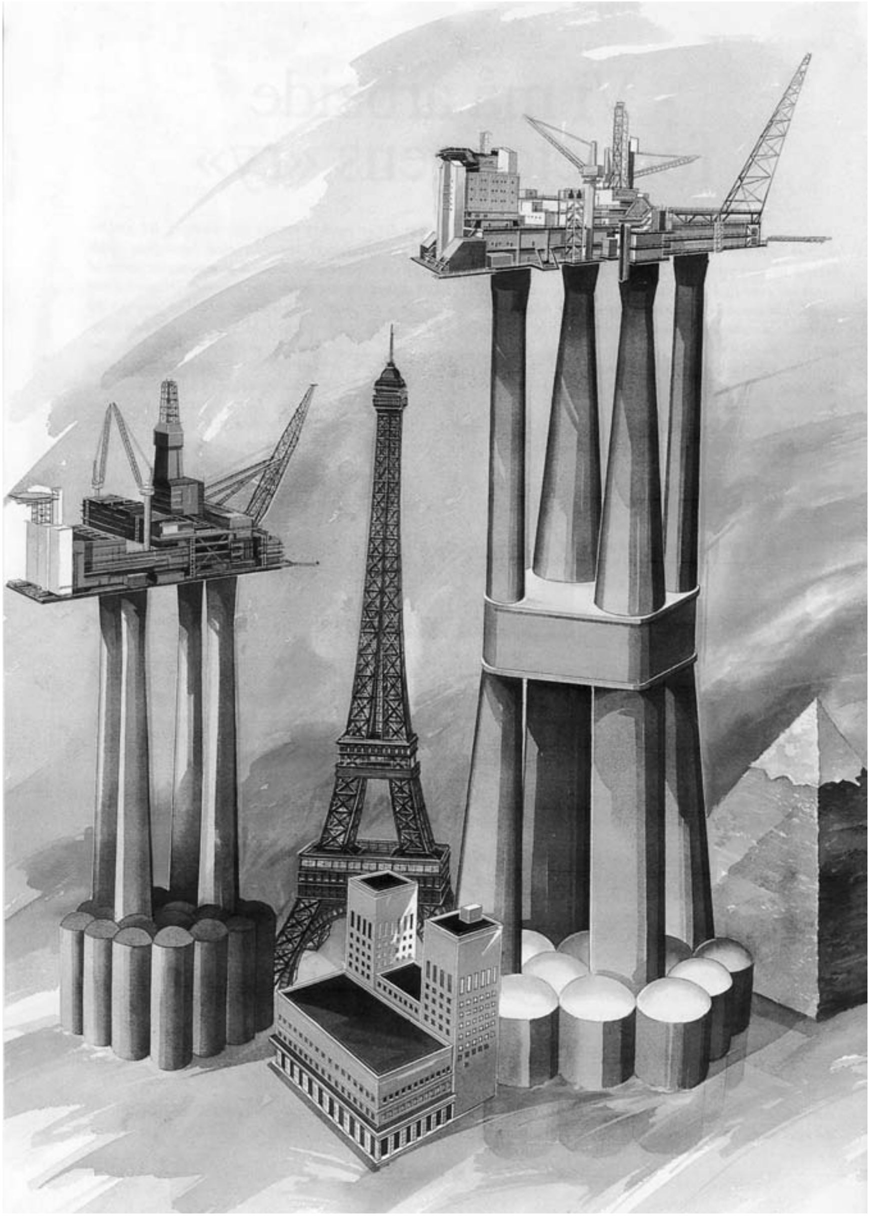


Figure 1.11 Artistic view of the Troll A Platform (1995), demonstrating that the Oslo City Hall is quite small in comparison (source: Courtesy of Per Helge Pedersen).

Figure 1.12 After production in one of the deep Norwegian fjords, the Troll A Platform was ready to be towed out to the North Sea in 1995 (source: Courtesy of Jan Moksnes).

Figure 1.13 The Troll A Platform (1995) on its way out to its final destination in the North Sea (source: Courtesy of Aker Solutions).

of 350 m, but this structure was a tension leg floating platform consisting of lightweight concrete with a design strength of 65 MPa. Both in the design, detailing and construction of all these offshore concrete structures, high safety, durability and serviceability were the subject of the greatest importance.

From the early 1970s, a rapid development of high strength concrete both for new offshore structures and a number of other types of structure took place (Gjørv, 2008). Since high strength and low porosity would also enhance the overall performance of the material, the term 'High Performance Concrete' was successively introduced and specified for concrete durability rather than for concrete strength. As experience with this type of concrete was gained, however, it was discovered that specification of 'High Performance Concrete' was not necessarily good enough for ensuring high durability of concrete structures in severe environments. In recent years, therefore, a new and rapid development of more advanced procedures for durability design has taken place. As a result, new concrete structures with a more controlled and improved durability can now be produced. Thus, for the most exposed parts of the Rion–Antirion Bridge which was constructed in the southern part of Greece in 2001 (Figure 1.14), concrete with an extremely high resistance against chloride penetration was applied, giving a very high safety level against steel corrosion. This type of concrete, which was based on a blast furnace slag cement with a high slag content, showed a 28-day chloride diffusivity of $0.8 - 1.2 \times 10^{-12}\,m^2/s$ according to the RCM method (NORDTEST, 1999), with achieved values

Figure 1.14 The Rion–Antirion Bridge (2001), to which a concrete with an extremely high resistance to chloride penetration was applied (source: Courtesy of Gefyra S.A.).

for the chloride diffusivity on the construction site after one year of $4.0 - 5.5 \times 10^{-13}\,m^2/s$ (Kinopraxia Gefyra, 2001).

Before current experience with durability design of new concrete structures is outlined and discussed, it may be useful to take a brief look at current experience with the field performance of existing concrete structures. A brief outline and review of deteriorating processes as well as codes and practice may also be useful.

2 Field performance

2.1 General

In many countries, extensive field investigations of a large number of important concrete structures in severe environments have been carried out. The results of all these investigations have been reported in comprehensive reports and published in numerous papers in journals and proceedings from various international conferences over a long period of time. For most of the concrete structures, it has primarily been electrochemical corrosion of embedded steel due to use of de-icing salt which has created the biggest and most severe problems (United States Accounting Office, 1979). Already in 1986, it was estimated that the cost of correcting corroding concrete bridges in the USA was US$24 billion with an annual increase of US$500 million (Transportation Research Board, 1986). Later on, annual costs of repair and replacement of US bridges of up to about US$8.3 billion have been estimated by Yunovich *et al.* (2001) and up to US$9.4 billion for the next 20 years by the American Society of Civil Engineers (Darwin, 2007). Already in 1998, annual costs of US$5 billion for concrete structures in Western Europe were estimated (Knudsen *et al.*, 1998), and similar durability problems and extensive expenses from a large number of other countries have also been reported.

For all the concrete structures in marine environments, the environmental conditions may be even more severe. Thus, along the Norwegian coastline, there are more than 300 concrete bridges and 10,000 harbour structures, most of which have been built from concrete (Figure 2.1). For many years, more than half of all these concrete bridges and most of the concrete harbour structures have been severely affected by corrosion of embedded steel (Østmoen *et al.*, 1993; Gjørv, 1968, 1994, 1996, 2002, 2006). In addition, in the North Sea, a number of the offshore concrete structures have shown some extent of steel corrosion.

Internationally, deterioration of concrete infrastructures has emerged as one of the most severe and demanding challenges facing the construction industry (Horrigmoe, 2000). Although corrosion of embedded steel represents the dominating type of deterioration, deterioration due to freezing

Figure 2.1 The Sortland Bridge (1975) is a 948 m-long cantilever bridge in the northern part of Norway (source: Courtesy of Johan Brun).

and thawing and alkali–aggregate reaction also represents a major problem for the durability and long-term performance of concrete structures in many countries. In order to describe the field performance of concrete structures in severe environments in more detail, some current experience based on field investigations of concrete structures, mostly in Norwegian marine environments, is briefly outlined and discussed in the following.

2.2 Harbour structures

Already in the early 1960s, a broad Nordic base of research cooperation was established in order to investigate the current condition and service life of concrete structures in marine environments in the Nordic countries. In Norway, this work was organized by a technical committee established by the Norwegian Concrete Association, and during the period 1962 to 1968 the condition and long-term performance of more than 200 concrete harbour structures along the Norwegian coastline was investigated (Gjørv, 1968, 1970). The majority of all these harbour structures were of the open type with a reinforced concrete deck on top of slender, underwater-cast concrete pillars (Figures 2.2 and 2.3). The structures had varying ages of up to 50 or 60 years and included more than 190,000 m^2 of concrete decks on more than 5000 tremie-cast concrete pillars with a total length of approximately 53,000 m. Of all the concrete structures, more than half were also investigated under water (Figure 2.4).

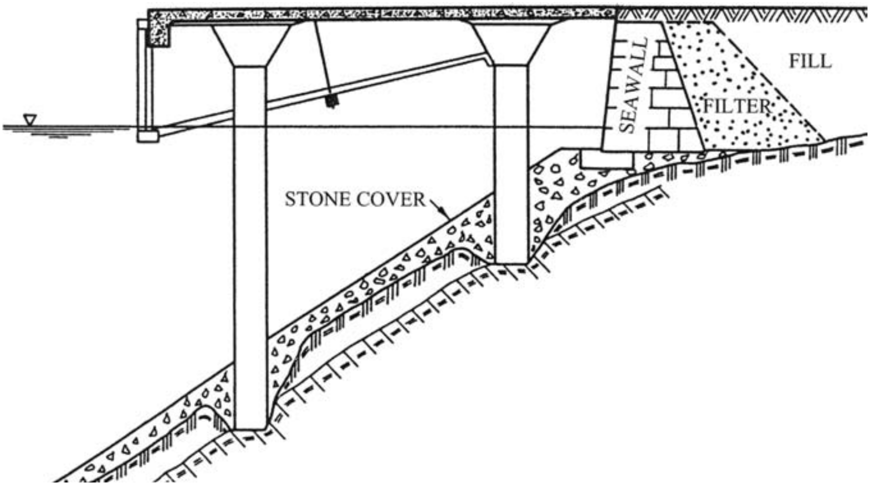


Figure 2.2 A section through an open concrete harbour structure with a reinforced concrete deck on top of slender, reinforced concrete pillars cast under water (source: Gjørv (1968)).

Figure 2.3 An industrial concrete harbour structure (source: Gjørv (1968)).

The overall condition of all the concrete harbour structures investigated was quite good. Even after service periods of up to 50 or 60 years, the structures still showed a high ability to withstand the combined effects of the most severe marine exposure (Figure 2.5) and heavy structural loads. Thus, one of the industrial harbour structures investigated was observed with storage of raw aluminium bars evenly distributed all over the deck as

Figure 2.4 The extensive field investigations of Norwegian concrete harbour structures during the 1960s were carried out by H.P. Sundh and O.E. Gjørv.

Figure 2.5 Concrete harbour structures along the Norwegian coastline are exposed to a most severe marine environment (source: Courtesy of B. Skarbøvik).

shown in Figure 2.6. This deck load was approximately six times that of the original design load. Apart from severe corrosion in all the deck beams, no specific sign of any overloading was observed on this 50-year-old concrete harbour structure.

For the continuously submerged parts of all the concrete structures, no particular trend in the development of damage either due to deterioration of the concrete or due to corrosion of embedded steel was observed. Within the tidal zone, only 35- to 40-year-old structures exhibited some concrete pillars with cross-sectional reductions of more than 20 per cent, mostly due to freezing and thawing (Figure 2.7). Above water, only 35- to 40-year-old structures had deck beams that had been severely weakened due to steel corrosion, while on the whole, both the deck slabs and the rear walls were in much better condition.

For 13 per cent of all the structures investigated under water, only some small and very local areas of deteriorated concrete were observed. In these local areas where problems with the tremie placing of the concrete had given a too porous and permeable concrete, a rapid chemical deterioration of the concrete had taken place. In these local areas, the strength was completely broken down after a relatively short period of time, and the deteriorated concrete was typically characterized by up to 70 per cent loss of lime due to heavy leaching and increased contents of magnesium oxide by up to ten times that of the original content (Gjørv, 1968). It should be

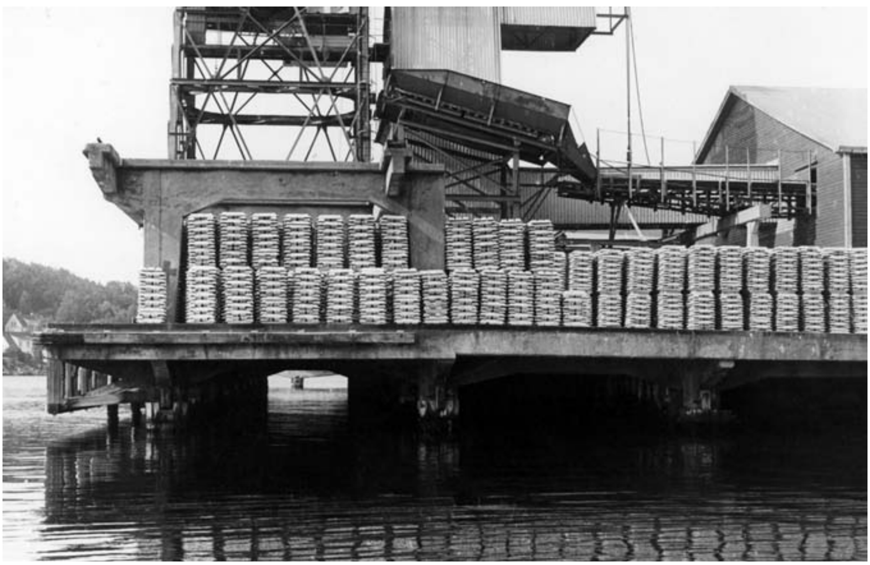


Figure 2.6 A 50-year-old industrial harbour structure (1913) with storage of raw aluminium bars representing an evenly distributed deck load of approximately six times that of the original design load (source: Gjørv (1968)).

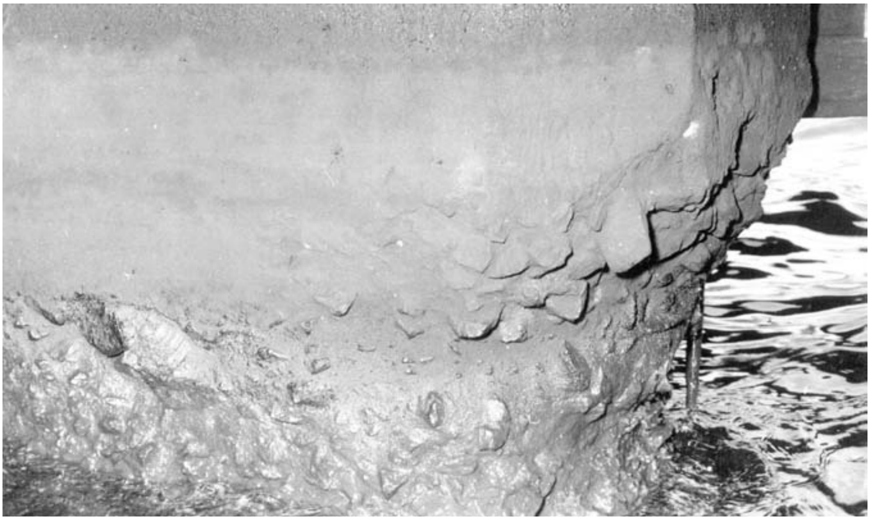


Figure 2.7 Deterioration in the tidal zone mostly due to freezing and thawing (source: Gjørv (1968)).

(a)

(b)

Figures 2.8(a) and (b) Tremie-cast concrete pillar (1929) in excellent condition after 34 years of freezing and thawing in Narvik Harbour (a) and 43 years in Glomfjord Harbour (1920) (b) (source: Gjørv (1968)).

noted that for all of these concrete structures, only pure portland cements had been used, which are known to be quite vulnerable in such aggressive environments.

Already in 1938, extensive long-term tests on concrete in seawater had begun at a field station in Trondheim Harbour. These field tests included more than 2500 concrete specimens based on 18 different types of commercial cement. After a period of 25 to 30 years, the different types of cement showed a wide variation in resistance against the chemical action of seawater, and the less durable the type of cement, the more important was the porosity and permeability of the concrete (Gjørv, 1971).

In order to maintain the cohesiveness of the fresh concrete during tremie placing of the concrete, however, a concrete with a very high cement content of at least 400 kg/m^3 had typically been used. Therefore, where no dilution of the tremie concrete had occurred during placing the concrete, a very good performance was observed even after service periods of up to 50 or 60 years.

Even after many years of freezing and thawing, the overall condition of the concrete pillars was very good. Even in the northern part of Norway with high amounts of frost and tidal waters of up to 2 or 3 m, a very good condition was observed after 30 or 40 years (Figures 2.8(a) and (b)). For those pillars with observed deterioration, the damage was mostly very local, demonstrating a high variability in concrete quality from one pillar to another (Figure 2.9). For most of the concrete pillars, however, the con-

Figure 2.9 Uneven deterioration in the tidal zone due to the high variability of concrete quality from one pillar to another (source: Gjørv (1968)).

crete had been protected by a wooden formwork which had been left in place after concreting, but this formwork had gradually disappeared. Since the tremie concrete had a very high cement content, a high concrete quality in the pillars had largely been achieved. However, most of the concrete structures investigated had been produced in a period before any air entraining admixtures were available.

Above water, more than 80 per cent of all the concrete structures had a varying extent of damage due to steel corrosion, and, in those structures which were observed without any damage, repairs due to steel corrosion had recently been carried out. The first visible sign of steel corrosion in the form of rust staining and cracks typically appeared after a service period of five to ten years, and it was primarily those parts of the structures which had been the most exposed to intermittent wetting and drying which were the most vulnerable to corrosion. Typically, this included the lower parts of the deck beams (Figure 2.10) and the rear parts of the concrete decks adjacent to the seawall (Figure 2.11). When the concrete cover in the lower parts of the deck beams had cracked or spalled off at a very early stage, it was typically observed that the longitudinal bars appeared to be more uniformly corroded, while the protruding beam stirrups showed a more severe pitting-type of corrosion and were mostly rusted through (Figure 2.12).

For the oldest structures, the specified compressive strength in the superstructure had typically varied from 25 to 30 MPa, but successively the specified strength had been increased up to 35 MPa. The specified

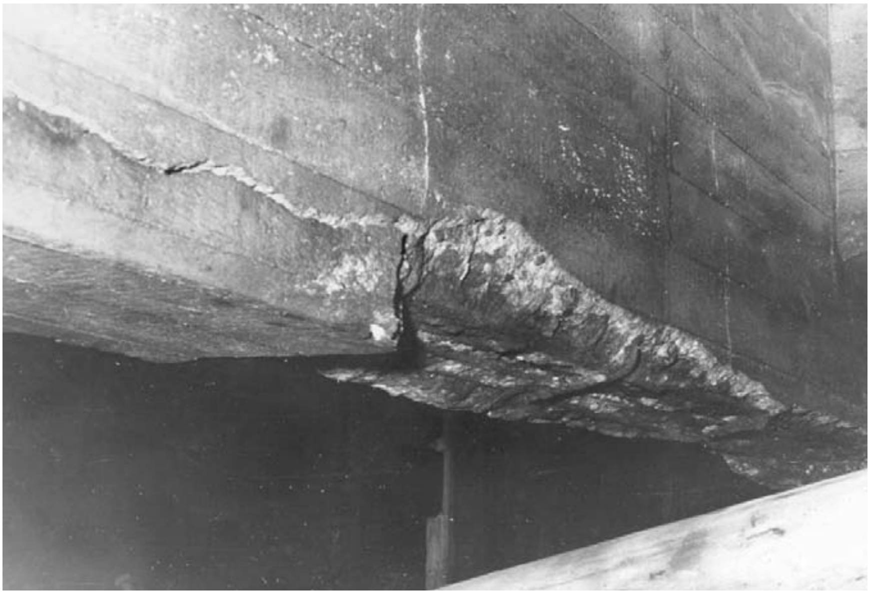


Figure 2.10 The lower parts of the deck beams were more vulnerable to steel corrosion than the deck slabs in between (source: Gjørv (1968)).

Figure 2.11 The rear parts of the concrete deck adjacent to the seawall were more vulnerable to steel corrosion than the rest of the deck (source: Gjørv (1968)).

Figure 2.12 When the concrete cover in the lower parts of the deck beams had cracked or spalled off at a very early stage, the steel bars were more uniformly corroded, while the protruding beam stirrups always showed a more pitting-type of corrosion (source: Gjørv (1968)).

minimum concrete cover in deck slabs, deck beams and the tremie-cast concrete pillars was typically 25, 40 and 70 mm, respectively. For some of the concrete structures, a cover thickness of 100 mm for the concrete pillars had also been specified.

Prior to 1930, experience had shown that the slabs of the open concrete decks had performed much better than the deck beams. This was assumed to be due to an easier and better placing and compaction of the fresh concrete in the deck slabs compared to that of the deep and narrow beams and girders. The practical consequence of this was drawn in 1932, when the first flat type of concrete deck was introduced into Norwegian harbour construction. From then on, a number of structures with a flat type of concrete deck were constructed, and these structures demonstrated a far better long-term performance than those with the beam and slab type of deck (Figure 2.13). Since such a structural design was often more expensive, however, the slab and beam type of deck was gradually reintroduced. It was assumed that if the beams were made shallower and wider, it would be equally easy to place the fresh concrete in such structural elements. After some time, however, even the shallower and wider deck beams showed early corrosion, while the flat type of concrete deck without any beams still performed much better.

What was not known in the earlier days was that after some years a concrete structure exposed to a chloride-containing environment would develop a complex system of galvanic cell activities along the embedded steel. In such a system, the more exposed parts of the concrete structure

Figure 2.13 Harbour structures with a concrete deck of the flat type showed a much better performance than structures with a beam and slab type of deck (source: Gjørv (1968)).

such as the deck beams would always absorb and accumulate more chloride and hence develop anodic areas, while the less exposed parts such as the slab sections in between would act as catchment areas for oxygen and hence form cathodic areas. As a consequence, the more exposed parts of the deck such as beams and girders would always be more vulnerable to steel corrosion compared to the rest of the concrete deck. This was also the reason why the rear parts of the concrete deck close to the seawall would be more vulnerable to steel corrosion than the rest of the concrete deck (Figure 2.11).

The extensive repair work carried out due to steel corrosion in the deck beams also showed a very short service life, mostly under ten years. Typically, the corroded steel in the locally spalled areas had first been cleaned and then locally patched with new concrete as is shown in Figure 2.14. In addition, what was not known in these early days was that such a local patch repair would always cause localized changes in the electrolytic conditions and thus variations in the electrochemical potentials along the steel. As a result, accelerated corrosion adjacent to the local patch repair would develop, the result of which is demonstrated in Figure 2.15. This detrimental effect of patch repairs was first observed and systematically reported by Stratfull in the early 1950s (Gewertz *et al.*, 1958).

In spite of the extensive steel corrosion which had been going on in almost all the concrete harbour structures for a long period of time, the effect of this corrosion on the load bearing and structural capacity of the structures appeared to be rather moderate and slow (Figure 2.16). For each structure investigated, both the effect on structural capacity and extent of damage within the various types of structural elements were rated according to a certain rating system. Thus, the structural condition

Figure 2.14 Typical patch repair of a corroded deck beam (source: Gjørv (1968)).

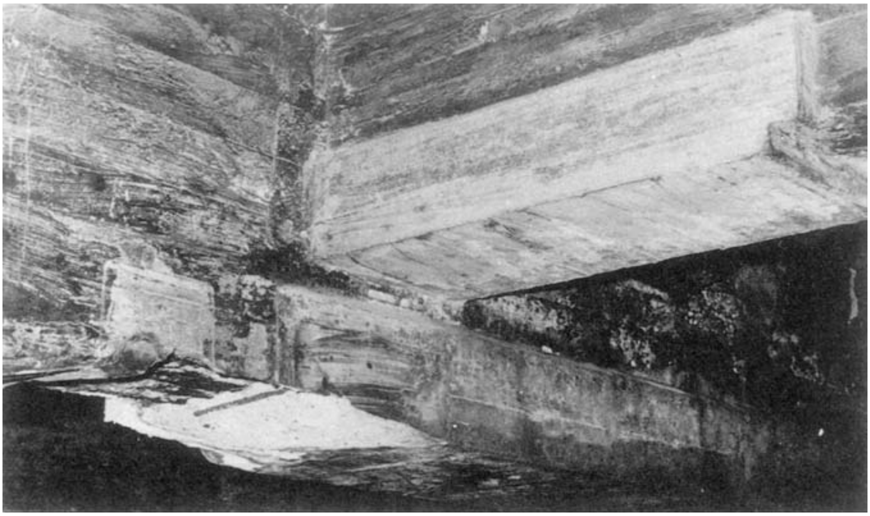


Figure 2.15 Accelerated corrosion adjacent to the local patch repair of a deck beam (source: Gjørv (1968)).

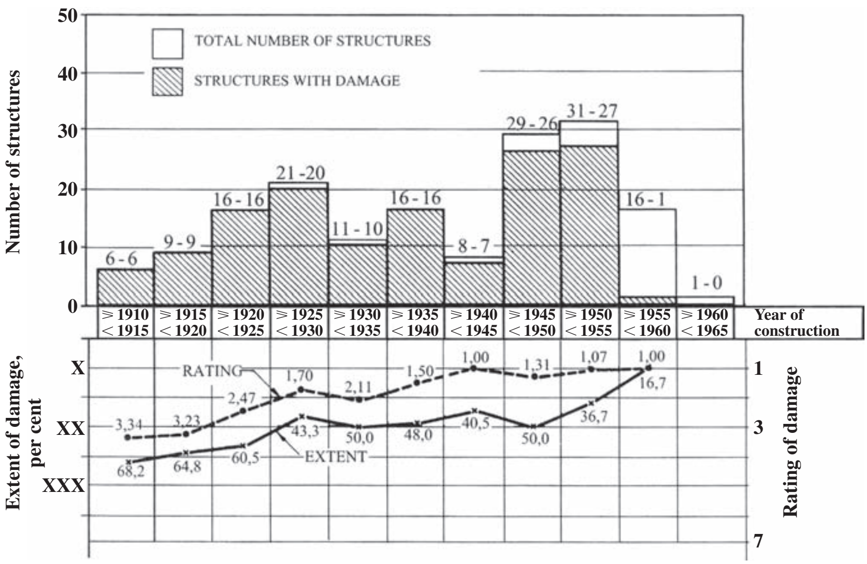


Figure 2.16 Trends in the development of deterioration due to steel corrosion in deck beams (source: Gjørv (1968)).

was rated according to a scale from 1 to 7, where 1 was distinct observed damage and 3 was major damage, but still the elements would fulfil the intended function, while 7 was such a severe impairment that the intended function of the elements would no longer be fulfilled. The extent of damage was rated into three groups, where X was up to one-third and XXX was more than two-thirds of all the same type of structural elements within each structure with observed damage. As may be seen from Figure 2.16, only 35- to 40-year-old structures (1925–1930) had deck beams with structural ratings of more than 2 and extents of damage of more than 50 per cent.

In 1982 to 1983, a more detailed investigation of one of the concrete harbour structures located in Oslo Harbour was carried out (Gjørv and Kashino, 1986). This was a 61-year-old concrete jetty which was going to be demolished in order to create space for new construction work. During demolition, therefore, a unique opportunity occurred for investigating the overall condition of all the embedded steel both in the concrete deck and in the tremie-cast concrete pillars.

An overall plan of the jetty is shown in Figure 2.17, which had an open concrete deck of 12,500 m^2 supported by approximately 300 tremie-cast concrete pillars with cross-sections of 90×90 cm and heights of up to 17 m. Of all these concrete pillars, four were pulled up on shore for a more detailed investigation (Figure 2.18). In 1919 to 1922 when the jetty was constructed, the specified compressive strength for the concrete in the superstructure was 25–30 MPa, but at the time of demolition the in-situ strength typically varied from 40 to 45 MPa. Although the original concrete composition was not known, all concrete structures at that early period of construction had typically been produced with a concrete based on very coarse-grained portland cements giving a very high, long-term strength development. For this particular structure, the specified minimum concrete cover for deck slabs, deck beams and tremie-cast concrete pillars was 30, 50 and 100 mm, respectively.

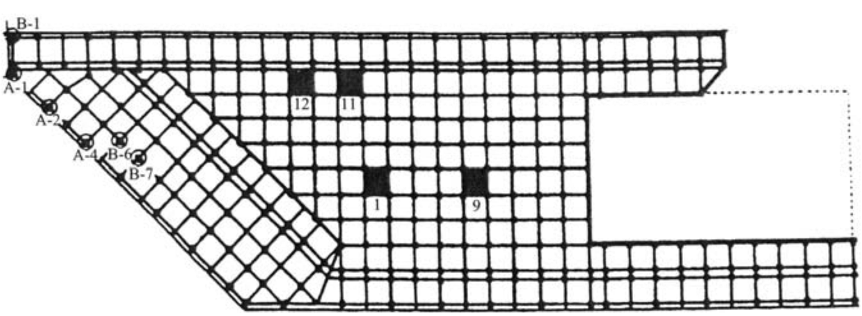


Figure 2.17 An overall plan of the concrete jetty in Oslo Harbour (1922), which was investigated during demolition after 61 years of service (source: Gjørv and Kashino (1986)).

Figure 2.18 Four of the tremie-cast concrete pillars from the concrete jetty in Oslo Harbour (1922) were pulled up on shore for a more detailed investigation.

Although extensive steel corrosion had been going on in all the deck beams throughout most of the service life of this particular jetty, the overall condition of the structure was relatively good even after a service period of more than 60 years. In spite of the extensive visible corrosion damage and deep chloride penetration beyond the embedded steel throughout the whole concrete deck, the demolition revealed that most of the rebars were still in quite good condition, and practically without any visible corrosion. This made up more than about 75 per cent of the total reinforcing steel system. For the rest of the reinforcement, which was mostly located in the lower part of the deck beams, the observed corrosion was very unevenly distributed and quite severe. However, in the lower parts of the deck beams, the cross-section of the bars was seldom reduced by more than 30 per cent, while for the rest of the rebar system, most of the steel bars had reduced cross-sections of less than 10 per cent. The best condition of the rebar system was observed in the deck slabs. These observations demonstrate how efficiently the corroding steel in the lower part of the deck beams had functioned as sacrificial anodes and thus cathodically protected the rest of the rebar system. This protective effect of the most corroding parts of the rebar system may also explain the relatively slow reduction in the structural capacity of all the structures previously investigated (Figure 2.16).

The four tremie-cast concrete pillars which were pulled up on shore showed a very good overall condition. A number of concrete cores showed compressive strengths of 40 to 45 MPa. Upon removal of the concrete cover, the embedded steel in the continuously submerged parts of the pillars showed a very good overall condition, mainly due to lack of oxygen. Above the low-water level, 1 mm-deep pittings on the individual bars were typically observed, while below this level the pittings were mostly under 0.2 mm and only occasionally as much as 0.5 mm.

For the concrete structure as a whole, it was not possible to find any relationship between the half-cell surface potentials observed before demolition and the condition of the embedded steel observed after demolition. The depth of carbonation, which was generally very small, varied according to the prevailing moisture conditions of the concrete. Thus, in the upper parts of the deck with a more dry concrete, a carbonation depth of 2 to 8 mm was typically observed, while for the concrete within the tidal zone and further below, the carbonation depth was generally much smaller, typically varying from 1 to 2 mm and only occasionally as much as 7 mm.

For most of the deck beams the concrete cover was more or less spalled off, so it was not easy to obtain representative observations of the chloride penetration. For the deck slabs, however, the chloride content at the level of the reinforcing steel typically varied from 0.05 to 0.10 per cent by weight of concrete. For the upper part of the pillars above the tidal zone the chloride content varied from 0.15 to 0.25 per cent, while in the tidal zone the chloride content mostly varied from 0.20 to 0.25 per cent (Figure 2.19). For the continuously submerged part of the concrete pillars, an even higher chloride content of 0.30 to 0.35 per cent was typically observed. As clearly demonstrated in Figure 2.19, the chloride content had reached far beyond the specified cover depth of 100 mm.

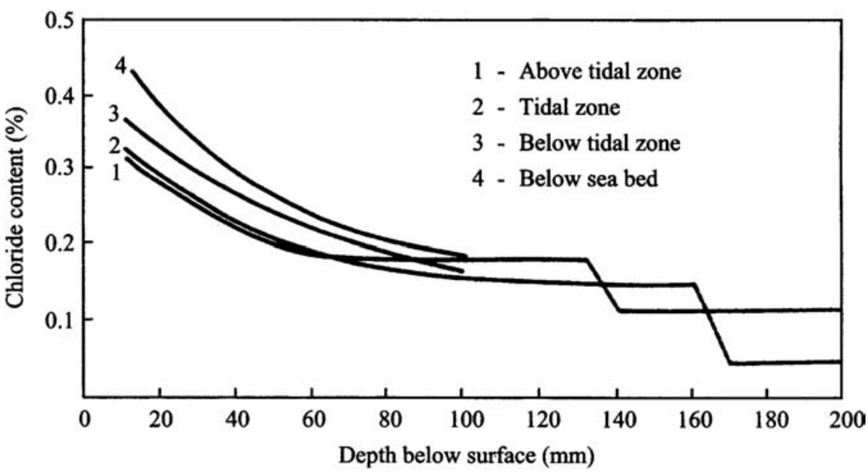


Figure 2.19 Penetration of chloride into the tremie-cast concrete pillars after 61 years of exposure (source: Gjørv and Kashino (1986)).

In more recent years, concrete technology, durability specifications and execution of concrete work for concrete harbour structures have generally improved. However, more recent field investigations of relatively new concrete harbour structures along the Norwegian coastline have revealed that a rapid and uncontrolled chloride penetration still represents a big problem, and steel corrosion may still be observed after service periods of under ten years (Lahus *et al.*, 1998). Thus, detailed field investigations of 20 Norwegian concrete harbour structures constructed during the period from 1964 to 1991 showed that 70 per cent had a varying extent of steel corrosion (Lahus, 1999). After five years of exposure, average chloride content at depths of 25 and 50 mm of 0.8 and 0.3 per cent by weight of cement was observed, while after ten years the corresponding numbers were 1.2 and 0.5 per cent, respectively. After 15 years of exposure, the average chloride content at the depth of 50 mm was 0.9 per cent by weight of cement.

In Trondheim Harbour, steel corrosion in most of the deck beams of a concrete jetty from 1993 after eight years of service was observed (Figure 2.20). According to the Norwegian Concrete Code NS 3420 (Standard Norway, 1986), a concrete quality with a compressive strength of 45 MPa and maximum water/binder ratio of 0.45 had been specified, and this had also been achieved by use of a concrete based on 380 kg/m^3 high performance portland cement and 19 kg/m^3 silica fume (5 per cent). During the field investigations after eight years of service, removed concrete cores showed an average chloride diffusivity according to the RCM method (NORDTEST, 1999) of 10.7×10^{-12} m^2/s, which indicates only a moderate resistance against chloride penetration. Since an average concrete cover in the deck beams of approximately 50 mm was observed, the specified

Figure 2.20 'Turistskipskaia' (1993) in Trondheim Harbour with steel corrosion in most of the deck beams after eight years of exposure (source: Gjørv (2002)).

minimum concrete cover of 40 mm according to Norwegian Concrete Code NS 3473 (Standard Norway, 1989) had also been achieved. Nevertheless, visual observations, detailed mapping of electrochemical surface potentials and extensive chloride penetration measurements revealed a varying extent of steel corrosion in all the deck beams of the structure. After approximately eight years of exposure, the chloride front had mostly reached a depth varying from 40 to 50 mm, as shown in Figure 2.21.

Also at Tjeldbergodden near Trondheim, deep chloride penetration and steel corrosion were observed after eight years of exposure on two industrial harbour structures constructed in 1995 and 1996, respectively (Figure 2.22). In addition, for these concrete structures, the specified durability with respect to both concrete quality and concrete cover according to the current concrete codes had been achieved. During the field investigations eight years on, the testing of chloride diffusivity according to the RCM method (NORDTEST, 1999) only showed an average value of $16.6 \times 10^{-12}\,m^2/s$, which indicates a very low resistance against chloride penetration. After approximately eight years of exposure, the observed chloride front typically varied from 40 to 50 mm (Figure 2.22), and the electrochemical surface potentials revealed extensive corrosion.

Both for the older and newer concrete harbour structures investigated and discussed above, a high scatter and variability of achieved construction quality were typically observed. Thus, for the two concrete harbour structures at Tjeldbergodden, Figure 2.23 demonstrates a high scatter of achieved concrete cover, typically varying from one deck beam to another. In another industrial harbour structure produced in 2001 at Ulsteinvik

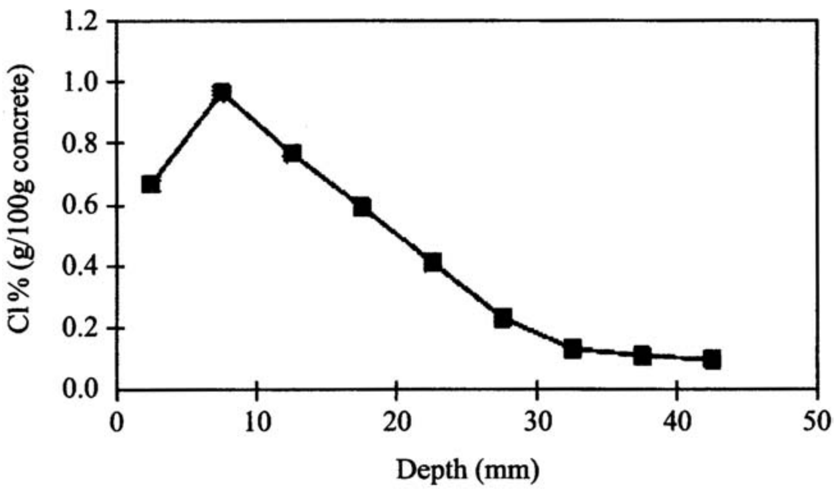


Figure 2.21 Typical chloride penetration in the deck beams of 'Turistskipskaia' (1993) in Trondheim Harbour after eight years of exposure (source: Gjørv (2002)).

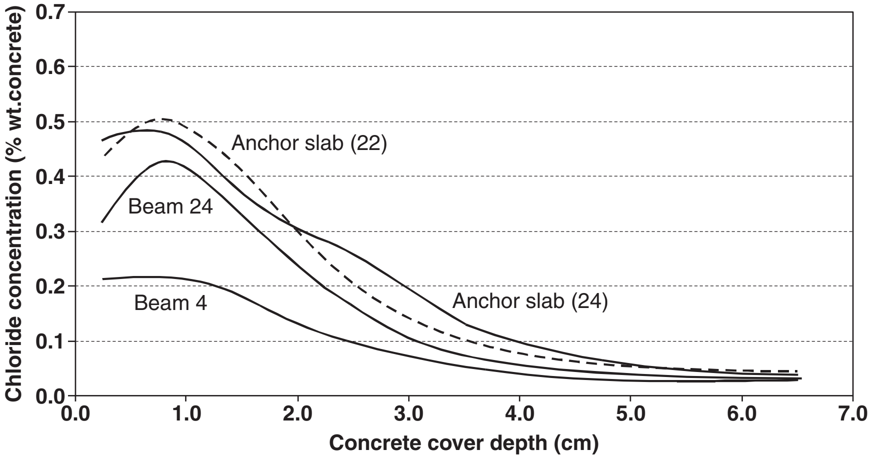


Figure 2.22 Chloride penetration after eight years of exposure in 'Modulkaia' (1995) at Tjeldbergodden (source: Ferreira *et al.* (2003)).

near Ålesund, a detailed investigation of the achieved chloride diffusivity in the concrete deck also showed a high variability (Figure 2.24). These results were based on migration testing of 12 Ø100 mm concrete cores from the concrete deck according to the RCM method (NORDTEST, 1999). Although the achieved concrete quality was in accordance with the

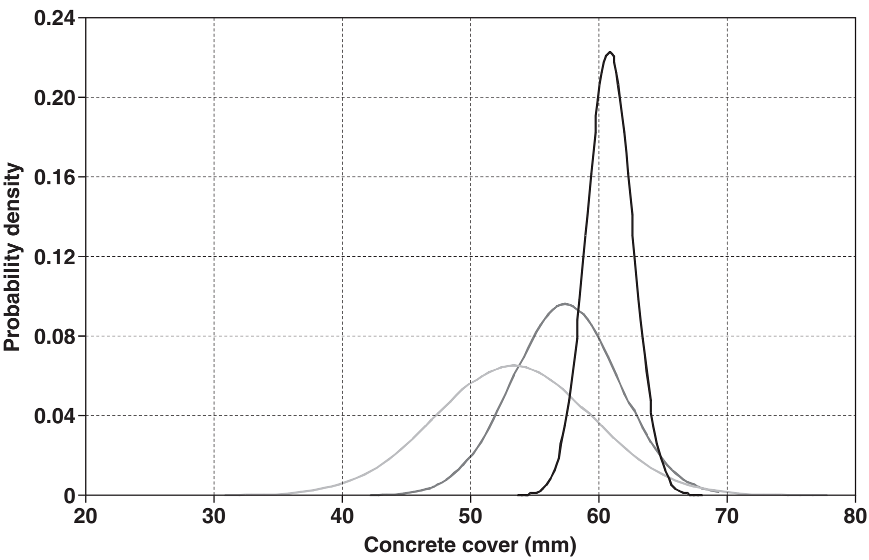


Figure 2.23 Histogram of concrete cover (mm) in three deck beams of the concrete harbour structures at Tjeldbergodden (source: Ferreira (2004)).

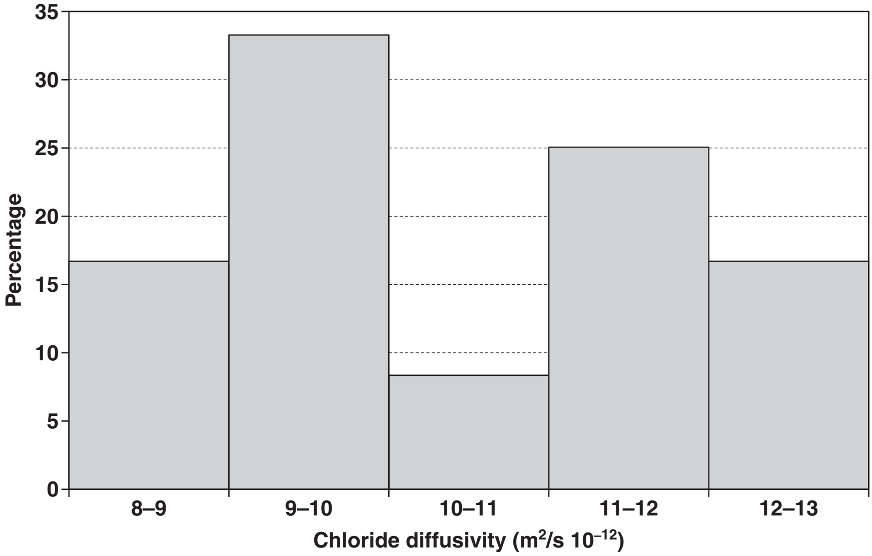


Figure 2.24 Achieved chloride diffusivity in the concrete deck of an industrial concrete harbour structure at Ulsteinvik (2001) (source: Guofei *et al.* (2005)).

current concrete code specification both with respect to water/binder ratio and binder content, the results in Figure 2.24 indicate only a moderate resistance against chloride penetration.

For all the concrete harbour structures described and discussed above, it should be noted that the environmental conditions are quite severe. For certain periods of the year, all the structures are typically exposed to the combination of severe splashing of seawater and high tides. Figure 2.25 shows the 'Turistskipskaia' in Trondheim Harbour on a stormy day. Occasionally, this structure is completely submerged in high tides during heavy storms, as is shown in Figure 2.26.

For much of the concrete construction work in Norwegian marine environments, the construction takes place all year around. Therefore, the risk of early-age exposure to seawater before the concrete has gained sufficient maturity and density is also high. Due to heavy winds and high tides during concrete construction of the 'Nye Filipstadkaia' in Oslo Harbour in 2002 (Figure 2.27), a deep chloride penetration in several of the deck beams occurred (Figure 2.28). During concrete construction, most types of concrete are very sensitive and vulnerable to chloride penetration, and this may represent a special challenge when the concrete construction work is carried out during cold and rough weather conditions, which may often be the case in Norwegian marine environments.

Figure 2.25 'Turistskipskaia' in Trondheim Habour on a stormy day (source: Courtesy of Trondheim Harbour KS).

Figure 2.26 Occasionally, concrete harbour structures are completely submerged in high tides during heavy storms (source: Courtesy of Trondheim Harbour KS).

Figure 2.27 During construction of 'Nye Filipstadkaia' (2002) in Oslo Harbour, deep chloride penetration in several of the deck beams took place due to heavy winds and high tides (source: Courtesy of Oslo Harbour KF).

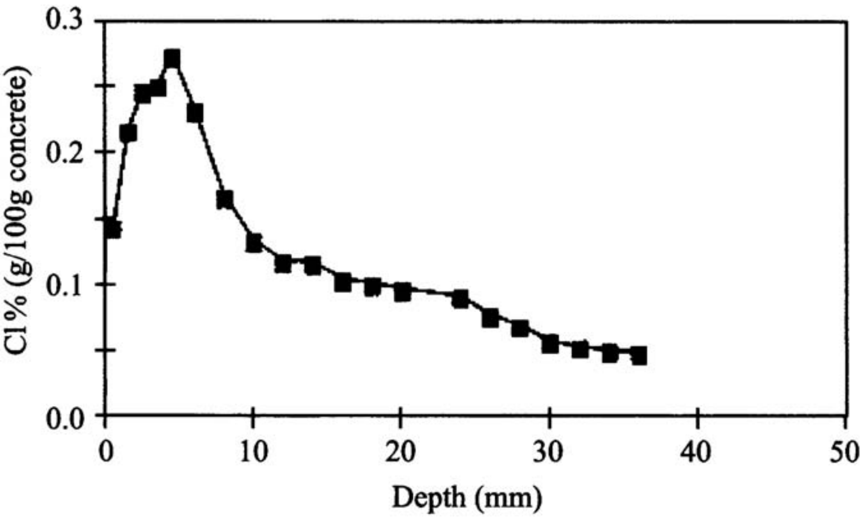


Figure 2.28 Observed chloride penetration in the deck beams during concrete construction of 'Nye Filipstadkaia' (2002) in Oslo Harbour (source: Gjørv (2002)).

In many countries, high temperatures may enhance the durability problems due to increased rates of deterioration. Thus, in the Persian Gulf countries, quite extreme durability problems are being experienced (Matta, 1993; Alaee, 2000). Similar problems have been reported from a number of other countries with hot climates. This is clearly demonstrated in the Progreso Port on the Yocatán Coast in Mexico, as is shown in Figures

2.29–2.32. Due to very shallow waters, two long piers were constructed in order to provide proper harbour facilities. Of the pier constructed in a conventional way in the 1960s, only small parts of the structure still remained, while the neighbouring pier, which was constructed during 1937 to 1941 with stainless steel reinforcements, was still in very good condition when it was investigated in 1998 (Knudsen and Skovsgaard, 1999). None of the concrete qualities in these two piers was very good. After a service period of approximately 60 years, however, detailed field investigations of the old pier showed that the embedded steel (Ø30 mm) was still in very good condition in spite of high chloride content adjacent to the steel, typically varying from 0.6 to 0.7 per cent by weight of concrete at depths of 80 to 100 mm below the concrete surface (Rambøll, 1999).

In more recent years, the old Progreso Pier has been further extended into deep waters (Figure 2.33), so that it now provides port facilities for heavy traffic with a variety of different types of ship (Figure 2.34). Before the old pier was constructed in the late 1930s, it was considered by the owner to be of vital importance to keep this pier in safe operation with as little interruption as possible. Therefore, the owner was willing to pay the additional costs for having a structure with as high durability as possible. Hence, this project clearly demonstrates how the additional costs of stainless steel later on have proved to be an extremely good investment for the owner (Progreso Port Authorities, 2008).

Figure 2.29 Remaining parts of a concrete pier built in a conventional way on the Yocatán Coast in Mexico in the 1960s (source: Courtesy of Rambøll Consulting Engineers).

Figure 2.30 Remaining part of the deck from the concrete pier built on the Yocatán Coast in the 1960s (source: Courtesy of Rambøll Consulting Engineers).

Figure 2.31 Different durability and long-term performance of the two concrete piers on the Yocatán Coast built with black steel in the 1960s (front) and with stainless steel during 1937–1941, respectively (source: Courtesy of Rambøll Consulting Engineers).

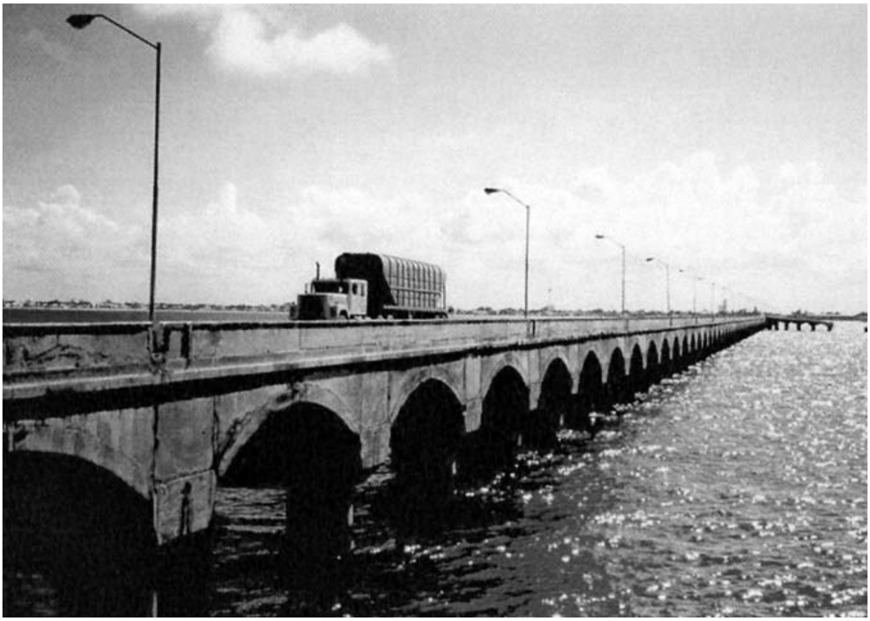


Figure 2.32 The Progreso Pier on the Yocatán Coast built with stainless steel reinforcements in the period 1937 to 1941 (source: Courtesy of Rambøll Consulting Engineers).

Figure 2.33 Overview of the Progreso Pier on the Yocatán Coast in Mexico (source: Courtesy of Progreso Port Authorities).

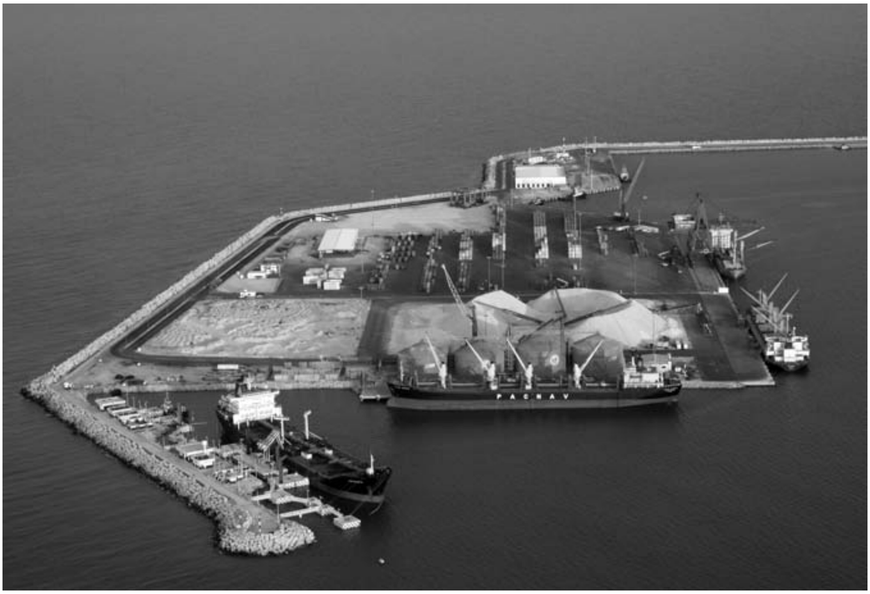


Figure 2.34 Part of the existing port facilities on the Progreso Pier (source: Courtesy of Progreso Port Authorities).

2.3 Bridges

In many countries, extensive field investigations of concrete bridges exposed to both de-icing salts and marine environments have been carried out. From recent field investigations of concrete bridges along the Norwegian coastline, it was revealed that more than 50 per cent of all the 300 existing concrete bridges either had a varying extent of steel corrosion or had recently been repaired due to steel corrosion (Østmoen *et al.,* 1993). Most of these concrete bridges were less than 25 years old, and one of them was so heavily corroded that it was demolished after a service period of about 25 years (Figure 2.35). During the service period of this particular bridge, total repair costs comparable to that of the original cost of the bridge had already been spent (Hasselø, 1997).

For most of the corroding bridges along the Norwegian coastline, large amounts of chloride had penetrated the concrete far beyond the level of the reinforcing steel after a relatively short period of service. Thus, for the Gimsøystraumen Bridge (Figure 2.36), which is the most common type of Norwegian coastal bridge from the 1970s and 1980s, the heavily corroding bridge was the subject of an extensive research programme carried out by the Norwegian Public Roads Administration during the period 1993 to 1997. The objective of this research programme was primarily to investi-

Figure 2.35 Ullasundet Bridge (1970) was demolished after 25 years of service due to heavy corrosion of embedded steel (source: Courtesy of Jørn A. Hasselø).

gate the effect of various types of patching repairs and patching materials in order to bring the steel corrosion under control (Blankvoll, 1997). After 11 years of exposure, extensive chloride penetration had taken place as shown in Figure 2.37, and the deepest chloride penetration was typically observed in those parts of the bridge which were the least exposed to the prevailing winds and salt spray (Figure 2.38). Thus, for the most exposed parts of the bridge, rain had intermittently been washing off the salt from the concrete surface, while for the more protected parts and surfaces the salt had accumulated. The observed chloride penetration typically varied with height above sea level as shown in Figure 2.39.

For the superstructure of Gimsøystraumen Bridge, concrete with a compressive strength of 40 MPa and a minimum concrete cover of 30 mm had been specified. Although such a specified concrete cover was very small for a bridge in such an environment, the achieved concrete cover observed later on was even smaller due to poor workmanship and lack of proper quality control during concrete construction. This is clearly demonstrated in Figure 2.40, where the results of more than 2028 single measurements of concrete cover during the extensive field investigations are shown.

For the Gimsøystraumen Bridge, the observed moisture content in the outer 40 to 50 mm of the concrete was typically very high, with relative humidities in the range of 70 to 80 per cent corresponding to a degree of

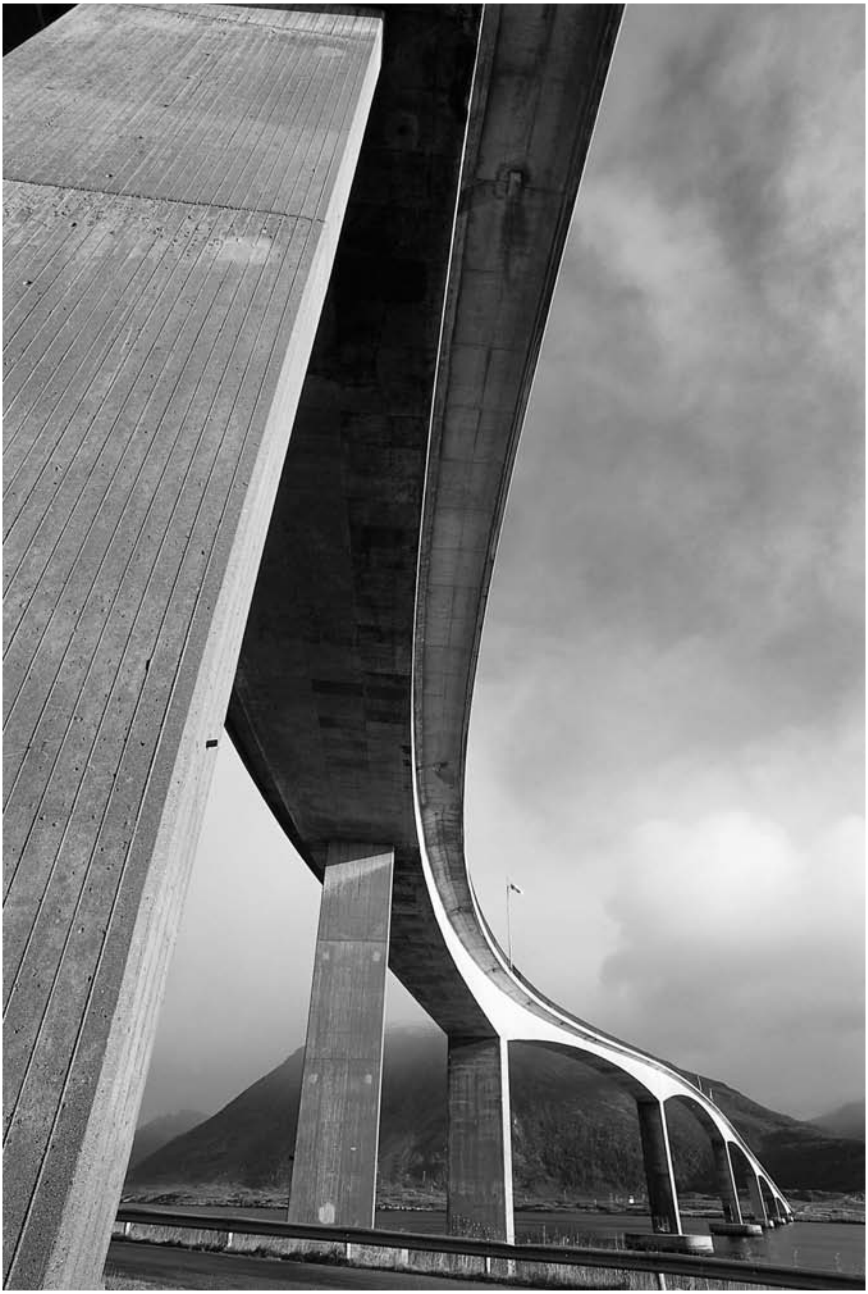


Figure 2.36 Gimsøystraumen Bridge (1981) which was the most common type of Norwegian concrete coastal bridge from the 1970s and 1980s (source: Courtesy of Johan Brun).

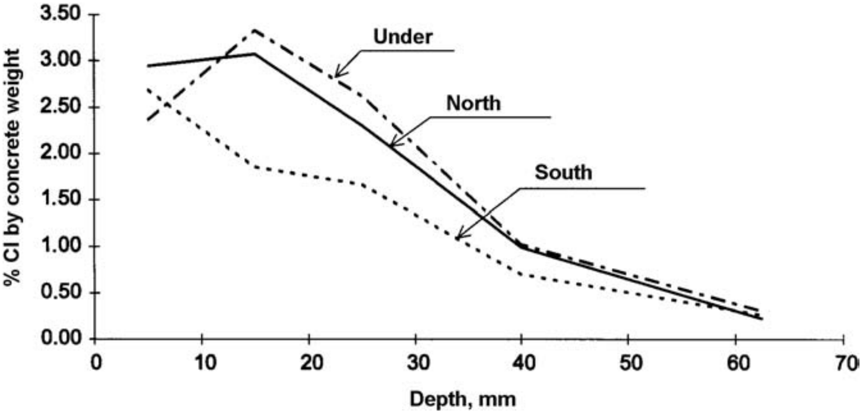


Figure 2.37 Gimsøystraumen Bridge (1981) with deep chloride penetration in the box girder 11.9 m above sea level after 11 years of exposure (source: Fluge (1997)).

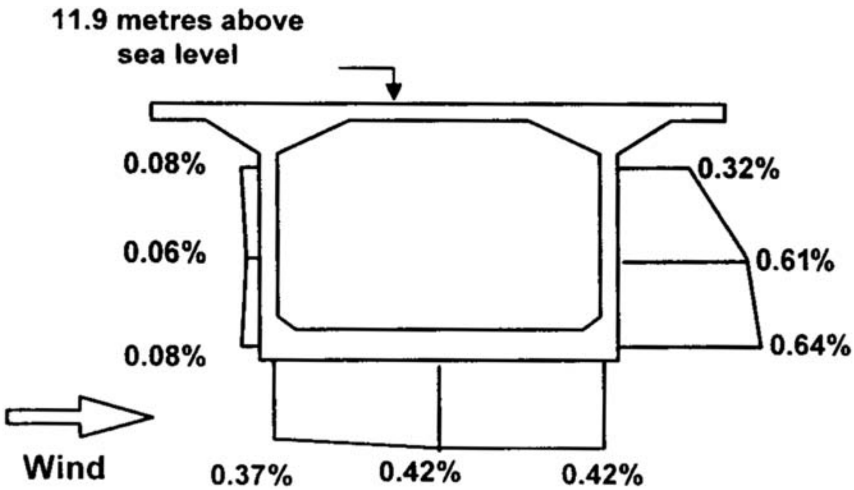


Figure 2.38 Gimsøystraumen Bridge (1981) with the deepest chloride penetration typically observed in those parts of the bridge which were the least exposed to prevailing winds and salt spray (source: Fluge (1997)).

capillary saturation of 80 to 90 per cent (Sellevold, 1997). Also, for other concrete bridges along the Norwegian coastline, very high moisture contents in the concrete have been observed. Although the moisture contents may vary from one structure to another, typical values for the degree of capillary saturation of 80 to 90 per cent have been reported (Holen Relling, 1999). For concrete in the tidal zone, the degree of capillary saturation may be even higher than 90 per cent, while for inland bridges the

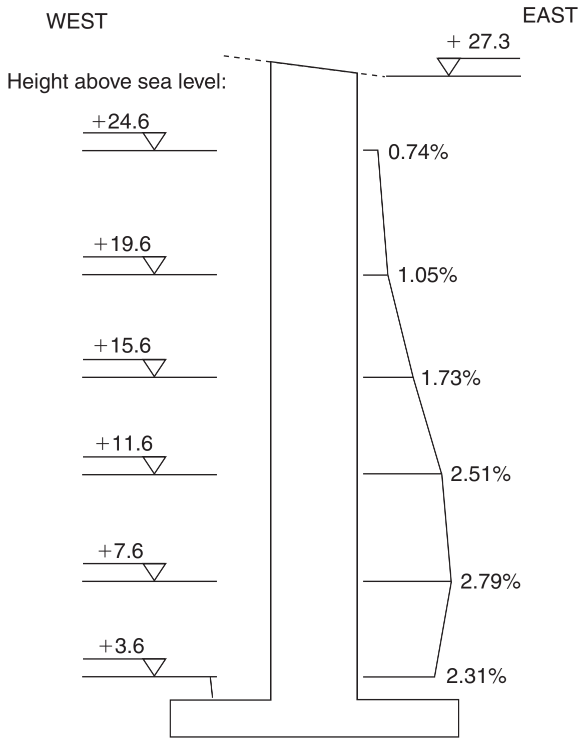


Figure 2.39 Gimsøystraumen Bridge (1981) with typical variation of chloride penetration above sea level (source: Fluge (1997)).

values may be closer to 80 per cent. Thus, for concrete bridges in such environments, the combination of high chloride content and high moisture content provides optimal conditions for high rates of steel corrosion.

In spite of the extensive patch repairs which were very carefully carried out in the research project for Gimsøystraumen Bridge, a continuing heavy steel corrosion was observed already less than ten years after the extensive and costly repairs. This effect of patch repairs is in general accordance with that typically observed and already discussed for all the concrete harbour structures along the Norwegian coastline. It is also in general accordance with extensive international experience, demonstrating that patch repairs of heavily chloride-contaminated concrete structures are not very effective in bringing ongoing steel corrosion under control (Bertolini *et al.*, 2004).

The above effect of patch repairs was first observed and systematically reported by Stratfull already in the early 1950s (Gewertz *et al.*, 1958). The San Mateo–Hayward Bridge (1929) in the Bay Area of San Francisco had been extensively patch repaired, first by local cleaning of the damaged

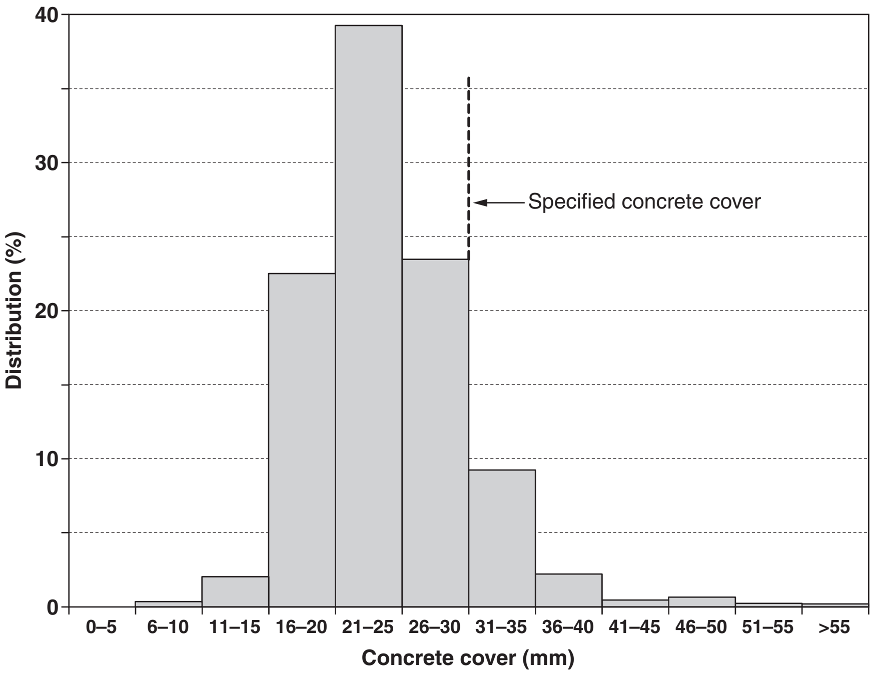


Figure 2.40 Gimsøystraumen Bridge (1981) with observed variation of achieved concrete cover (source: Kompen (1998)).

areas and then by shotcreting. After a short period of time, however, continued steel corrosion was observed. During his extensive field investigation of the bridge, Stratfull for the first time carried out detailed potential mapping of the patch repaired concrete structure with half-cell copper-copper sulphate electrodes. In this way, he demonstrated that the patched areas had typically formed cathodic areas, while anodic areas had formed adjacent to the patched areas as demonstrated in Figure 2.41. During his extensive field investigations, Stratfull also for the first time carried out a number of electrical resistivity measurements along the concrete surface of the bridge. For these measurements, Stratfull had designed his own four-electrode (Wenner) device, as is shown in Figure 2.42. Over a certain period of time, he observed that in those parts of the bridge which showed a high electrical resistivity of typically more than 65,000 ohm cm, a very low and almost negligible corrosion rate took place. Stratfull also observed that the ohmic resistance of the concrete was primarily controlled by the moisture content of the concrete, which typically varied from one part of the concrete deck to another.

Already in the early 1970s, Stratfull also demonstrated that cathodic protection would be the most efficient way to bring the above type of corrosion

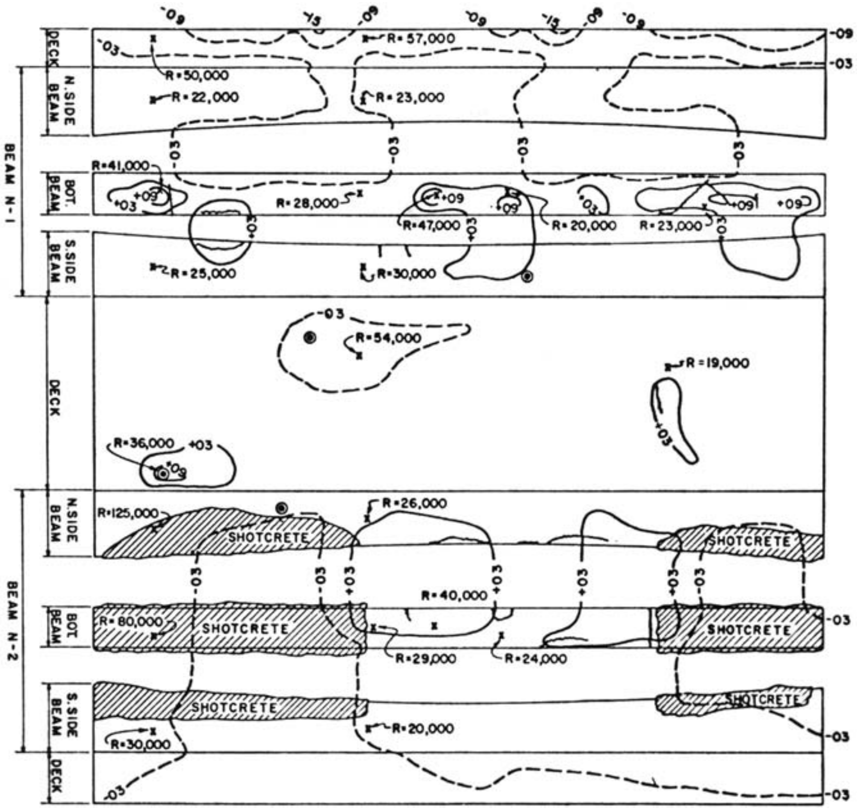


Figure 2.41 Equipotential contours demonstrating how the local patch repairs by shotcreting had typically formed a pattern of anodic areas (solid lines) and cathodic areas (dotted lines) along the concrete surface of the San Mateo–Hayward Bridge (1929) (source: Gewertz *et al.* (1958)).

under control (Stratfull, 1974). Later on, extensive practical experience has confirmed that cathodic protection is the only way of bringing heavy chloride-induced steel corrosion under control (Broomfield, 1997).

Of the younger generation of concrete bridges more recently constructed along the Norwegian coastline, both improved concrete quality with compressive strengths in the range of 45 to 65 MPa and increased concrete cover in the range of 40 to 55 mm have distinctly improved the durability of the bridges. As already discussed, however, the environmental conditions along the Norwegian coastline are quite severe. Thus, for the Storseisund Bridge which was built in 1988, deep chloride penetration was observed after approximately 15 years of exposure (Figure 2.43). This bridge is one of several concrete bridges built along the Atlantic Ocean Road on the west coast of Norway. During the summer season this

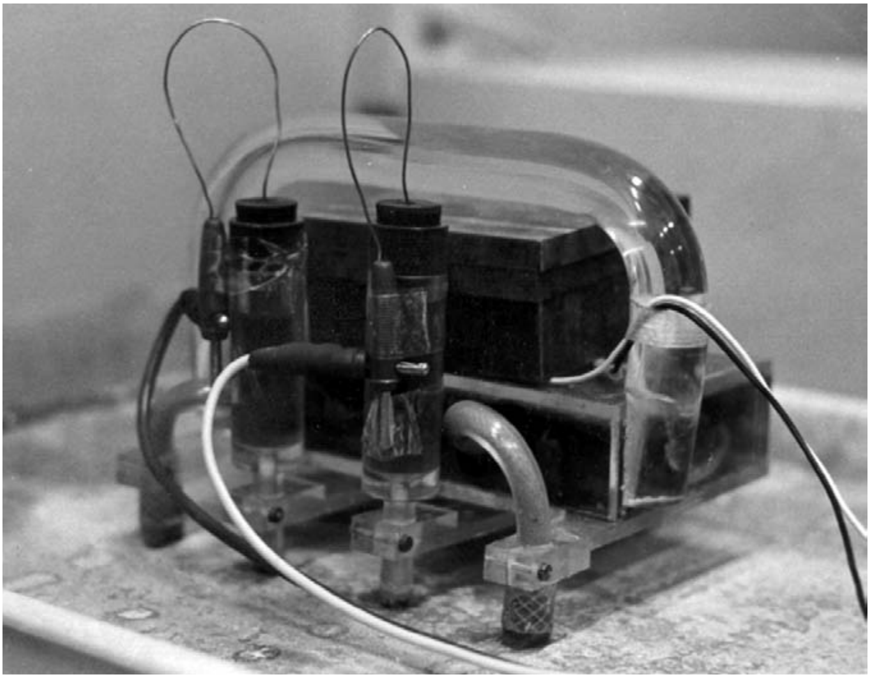


Figure 2.42 The four-electrode (Wenner) device designed by Stratfull in the early 1950s for field measurements of electrical resistivity along the concrete surface of the San Mateo–Hayward Bridge (1929) (source: Stratfull (1970)).

highway is a very pleasant and popular tourist route (Figure 2.44), but during stormy winter days, these bridges are heavily exposed and partly covered by seawater, as is shown in Figure 2.45.

For the 1065 m-long cable-stayed Helgelands Bridge (Figure 2.46) built in 1991 further north along the Norwegian coastline, a deep chloride penetration was observed already shortly after completion of the bridge (Figure 2.47). During the construction, however, parts of the bridge were heavily exposed to severe weather conditions and splashing of seawater before the concrete had gained sufficient maturity and density (Figures 2.48(a) and (b)).

In order to gain experience for increased durability of new concrete bridges along the Norwegian coastline, the Norwegian Public Roads Administration decided subsequently on an experimental basis to further increase both concrete quality and concrete cover in a new bridge to be built at Aursundet on the west coast of Norway. Thus, when the Aursundet Bridge was built between 1993 to 1995 (Figure 2.49), a concrete based

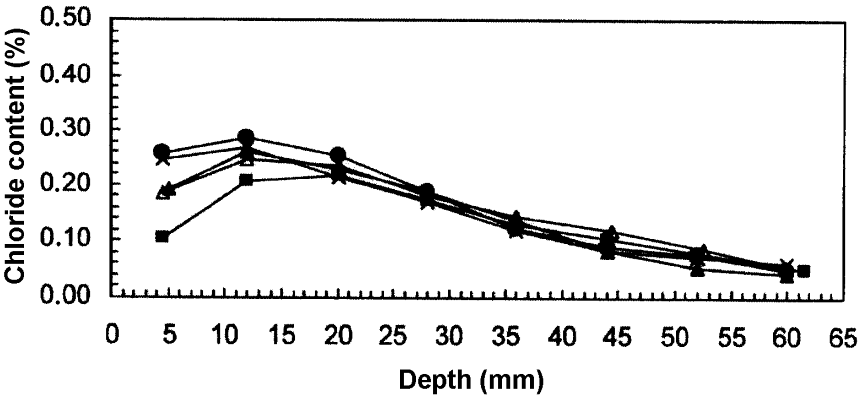


Figure 2.43 Storseisund Bridge (1988) with observed chloride penetration in per cent by weight of concrete after 15 years of exposure (source: Hasselø (2007)).

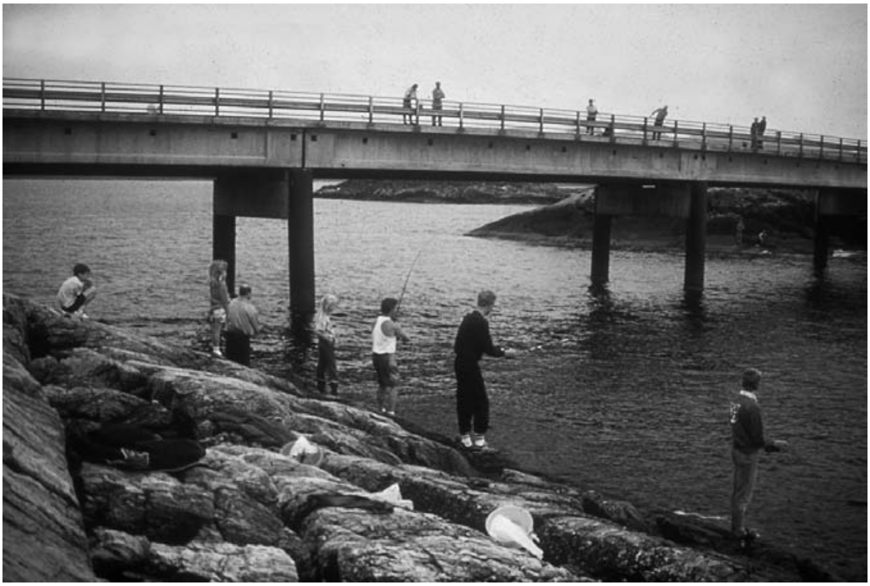


Figure 2.44 During summer, the Atlantic Ocean Road on the west coast of Norway is a very pleasant and popular tourist route.

on 400 kg/m^3 of portland cement with 50 kg/m^3 of silica fume (12.5 per cent) giving a water/binder ratio of 0.40 was applied. This concrete, which showed a 28-day compressive strength of 55 MPa, was combined with a minimum concrete cover of 80 mm in the splash zone. Ten years on, field investigations revealed average chloride penetrations in the eastern and

Figure 2.45 During stormy winter days, the Storseisund Bridge (1988) is heavily exposed and partly covered by seawater (source: Courtesy of Johan Brun).

Figure 2.46 The Helgelands Bridge (1991) is a 1065 m-long cable-stayed bridge with the largest span of 425 m (source: Courtesy of Hallgeir Skog).

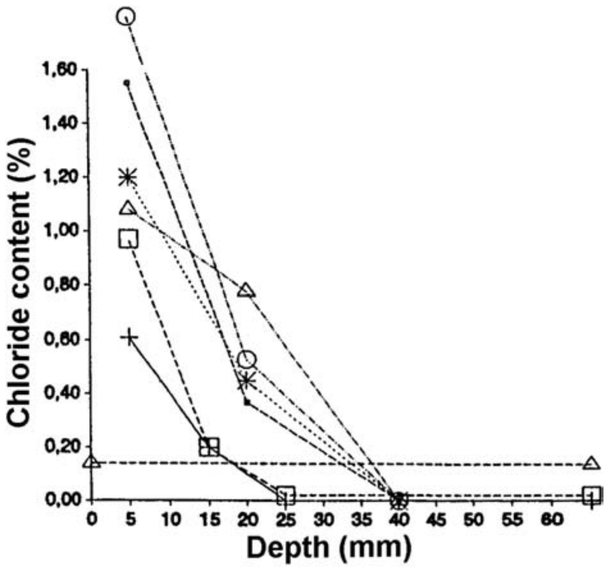


Figure 2.47 The Helgelands Bridge (1991) with observed chloride penetration in per cent by weight of cement shortly after completion of the bridge (source: NPRA (1993)).

western parts of the bridge as shown in Figure 2.50. At the same time, a testing of the chloride diffusivity of the concrete according to the RCM method (NORDTEST, 1999) was carried out. An average observed chloride diffusivity of $6.2 \times 10^{-12}\,m^2/s$ indicated that the applied concrete quality for this bridge had a much higher resistance to chloride penetration compared to that typically applied for earlier bridges.

Also, in a number of other countries, extensive field investigations of concrete bridges both exposed to marine environments and de-icing salts have shown the same type of durability and long-term problems due to steel corrosion as that described and discussed above (Nilsson, 1991; Stoltzner and Sørensen, 1994; Beslac *et al.*, 1997; Wood and Crerar, 1997).

2.4 Offshore structures

For many of the offshore concrete platforms in the North Sea, a service life of only 25 to 30 years was required, but for all of these concrete structures, much stricter durability specifications than that for land-based concrete structures produced in the same period were applied. In spite of the very harsh and hostile marine environment in the North Sea as shown in Figures 2.51 and 2.52, the overall performance of the concrete structures has so far

(a)

(b)

Figures 2.48(a) and (b) During concrete construction, parts of the Helgelands Bridge (1991) were heavily exposed to severe weather conditions and splashing of seawater (source: Courtesy of Hallgeir Skog).

Figure 2.49 The Aursundet Bridge (1995) is a cantilever bridge with a total length of 486 m.

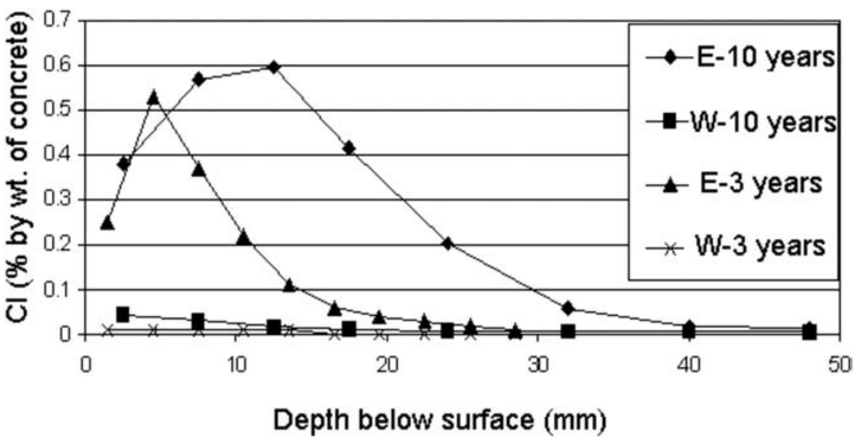


Figure 2.50 The Aursundet Bridge (1995) with observed chloride penetration after three and ten years of splash zone exposure (source: Årskog *et al.* (2005)).

been shown to be quite good (Fjeld and Røland, 1982; Hølaas, 1992; Gjørv, 1994; FIP, 1994; Moksnes and Sandvik, 1996). For most of the structures, no systematic monitoring of the chloride penetration has been carried out, but extensive field investigations have revealed that also for this high performance concrete, a certain rate of chloride penetration has taken place, although at a slower rate compared to that observed for other concrete structures along the Norwegian coastline (Figures 2.53 and 2.54).

For some of the concrete structures in the North Sea, steel corrosion has also created severe durability problems, and costly repairs have been carried out. In particular this is true for the Oseberg A Platform (1988), where the achieved concrete cover was less than that specified in the upper parts of the shafts. For this platform, costly repairs in the form of cathodic protection have been carried out (Østmoen, 1998).

For the first concrete structures in the North Sea produced in the early 1970s, it was not so easy to meet a specified compressive strength of 45 MPa in combination with required air content of at least 5 per cent for ensuring proper frost resistance. The general durability requirements included a water/cement ratio of less than 0.45 or preferably below 0.40, in combination with a minimum cement content of 400 kg/m^3. Nominal concrete covers to the ordinary and prestressing steel of 75 and 100 mm, respectively, were also specified. Already from the mid-1970s, however, it became easier to produce high strength concrete with a water/cement ratio

Figure 2.51 All the concrete platforms in the North Sea are exposed to a very harsh and hostile marine environment.

Figure 2.52 All the concrete platforms in the North Sea are exposed to heavy wave loading.

below 0.40, and, later, the concrete quality gradually increased from project to project reaching a compressive strength of up to 80 MPa for the Troll A Platform which was installed in 1995.

Although very strict durability requirements were specified, most of the concrete structures produced prior to 1980 also had an additional protective surface coating on the concrete shafts in the splash zone. For these concrete structures, a solid epoxy coating of 2 to 3 mm was continuously applied to the concrete surface during slip forming of the structures. When the surface coating was applied at such an early stage when the young concrete still had an under-pressure and suction ability, a very good bonding between the concrete and the epoxy layer was achieved. Thus, for the Statfjord A Platform, Figure 2.53 demonstrates that the epoxy coating had very effectively prevented any chloride from penetrating the concrete during a service period of eight years. Even after 15 years of severe exposure, later investigations have revealed that this protection was still intact and had very effectively prevented any chloride from penetrating the concrete (Aarstein *et al.*, 1998). However, for the Heidrun Platform, which was a floating concrete platform produced in 1995, Figure 2.55 demon-

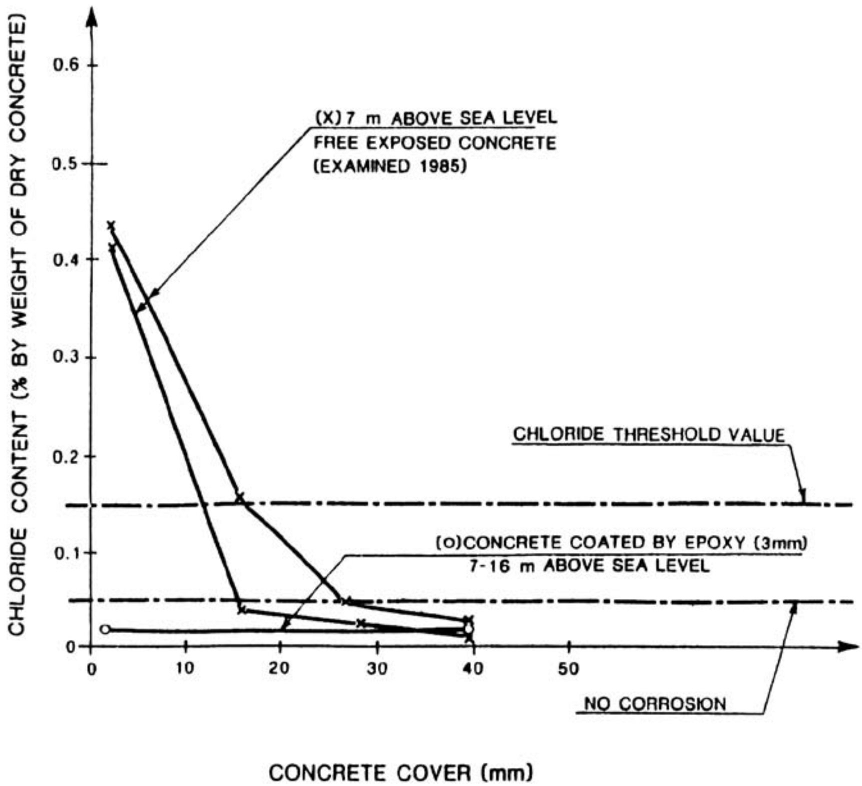


Figure 2.53 Chloride penetration in the Statfjord A Platform (1977) after eight years of exposure (source: Sandvik and Wick (1993)).

strates that the much thinner and poorer surface coating applied to this platform had not been so effective in keeping the chlorides out, even after an exposure period of only two years.

Although the Brent B Platform (1975) was one of the concrete platforms to be built before 1980, this structure was not protected by any surface coating in the splash zone. In conjunction with some installation work carried out on this platform in 1994, a large number of Ø 100 mm concrete cores had to be removed from the utility shaft at two different elevations above water and one elevation below water. After approximately 20 years of exposure, extensive investigations of these removed concrete cores revealed that a deep chloride penetration had taken place, as is shown in Figures 2.56 to 2.59. The chloride penetration was deepest in the upper part of the splash zone and shallowest in the constantly submerged part of the shaft at an elevation of –11.5 m. In the upper part of the shaft at approximately 14 m above water level, a chloride front of

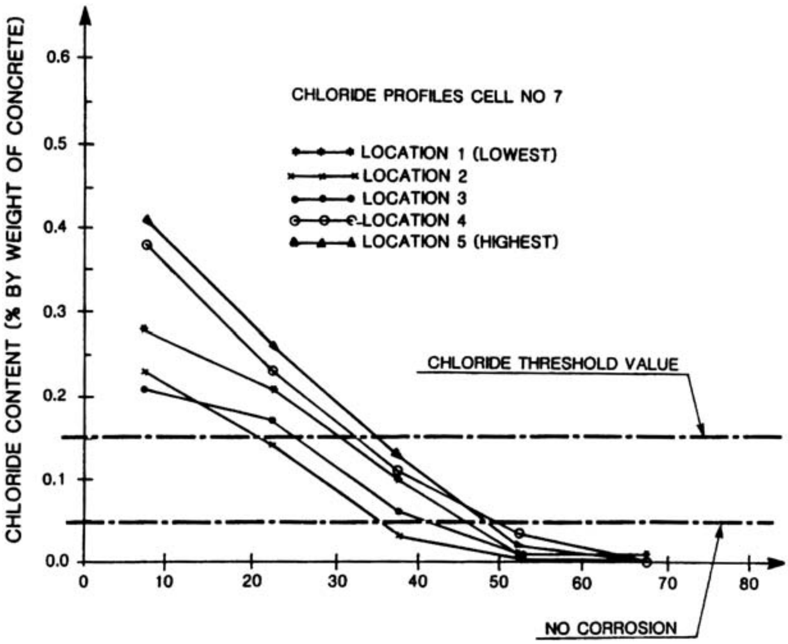


Figure 2.54 Chloride penetration in the Ekofisk Tank (1973) after 17 years of exposure (source: Sandvik *et al.* (1994)).

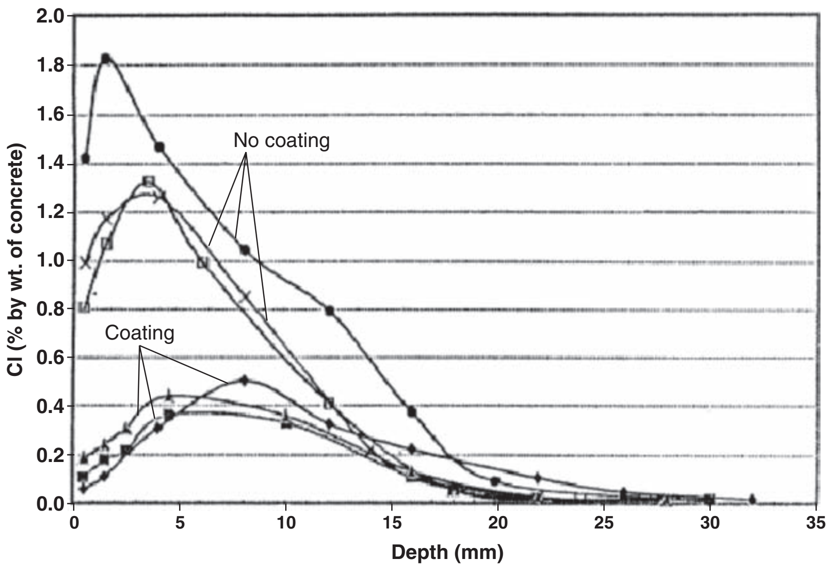


Figure 2.55 Effect of surface coating on chloride penetration in the Heidrun Platform (1995) after two years of exposure (source: Gjørv (2002)).

approximately 0.07 per cent by weight of concrete at a depth of approximately 60 mm was observed. For a nominal concrete cover of 75 mm, this indicates that an early stage of steel corrosion had probably been reached.

In 1994, a number of concrete cores were also removed from below the water level of the Brent C Platform (1978). After approximately 17 years of exposure, investigations of these concrete cores which had been removed from elevations of –9 to –18.5 m also revealed a deep and varying chloride penetration, as is shown in Figure 2.60.

Since both the Brent B and Brent C Platforms were constructed at a very early stage of all the platform construction for the North Sea, it should be noted that the concrete qualities in these early platforms were not as high as those applied for the platforms constructed later on. For The Brent B Platform, however, a concrete with a water/cement ratio of less than 0.40 and a cement content of more than 400 kg/m^3 were applied. During concrete construction, the extensive quality control of the air-entrained concrete above water and the non-air-entrained concrete below water showed average 28-day compressive strengths of 48.4 and 56.9 MPa, respectively. Above water, the specified concrete cover of 75 mm had also been achieved by use of mortar blocks of comparable strength and durability to that of the structural concrete.

During the extensive testing of all the removed concrete cores from the Brent B Platform, the homogeneity of achieved concrete quality was also investigated by use of the RCM method (NORDTEST, 1999). These tests included altogether 14 Ø100 mm concrete cores from all three levels of the concrete shaft, and as may be seen from Figure 2.61, this type of concrete

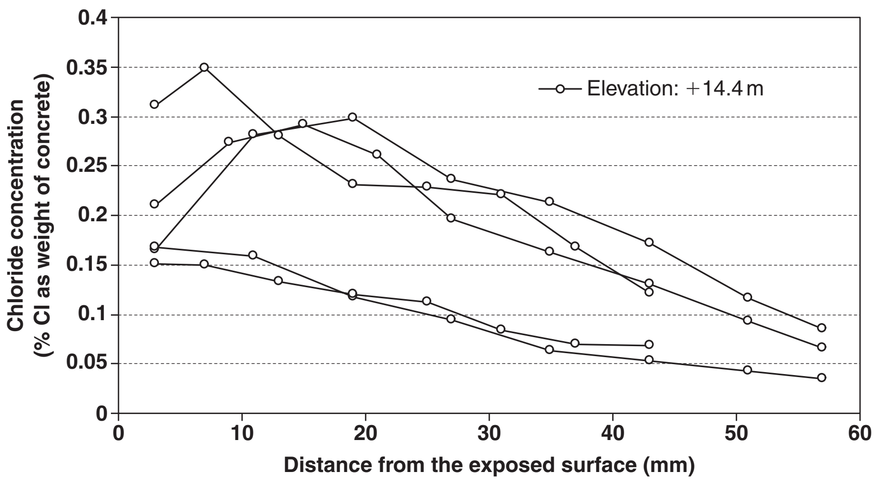


Figure 2.56 The Brent B Platform (1975) with observed chloride penetration at elevation +14.4 m above water after 20 years of exposure (source: Sengul and Gjørv (2007)).

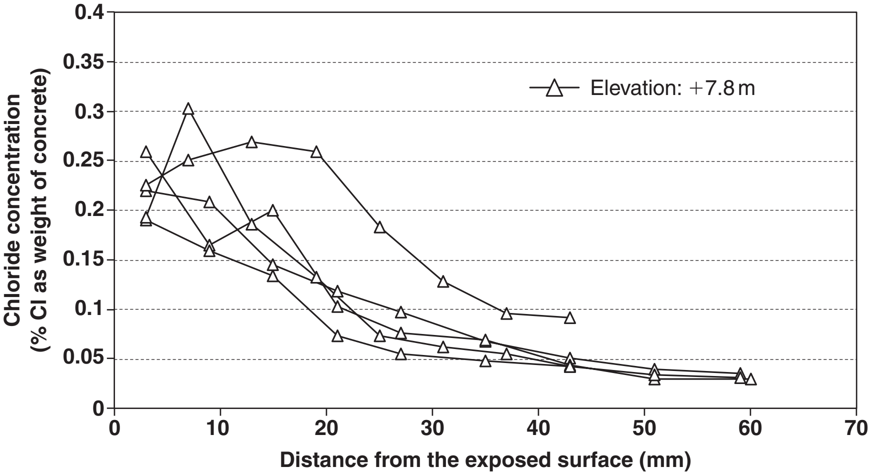


Figure 2.57 The Brent B Platform (1975) with observed chloride penetration at elevation +7.8 m above water after 20 years of exposure (source: Sengul and Gjørv (2007)).

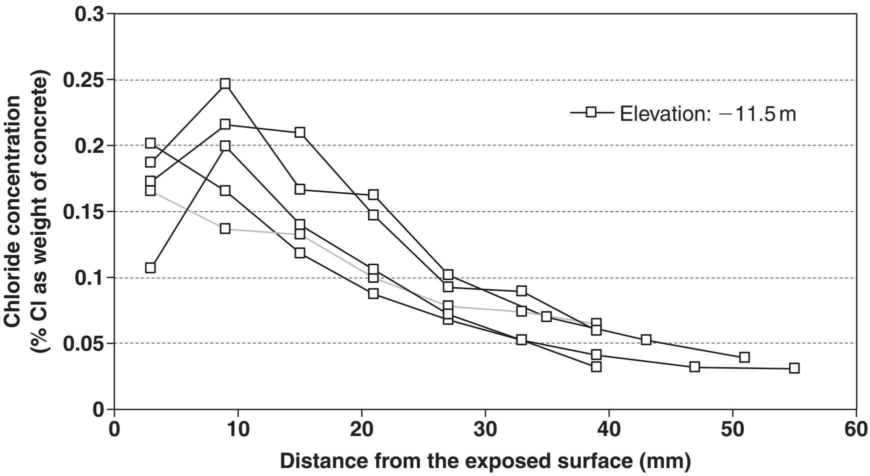


Figure 2.58 The Brent B Platform (1975) with observed chloride penetration at elevation –11.5 m below water after 20 years of exposure (source: Sengul and Gjørv (2007)).

also showed a high scatter and variability with observed chloride diffusivities varying from approximately 20 to 30 × 10^{-12} m^2/s. These results primarily reflect a high inhomogeneity of achieved concrete quality. The level of observed chloride diffusivity also indicates that the concrete had a relatively low resistance to chloride penetration.

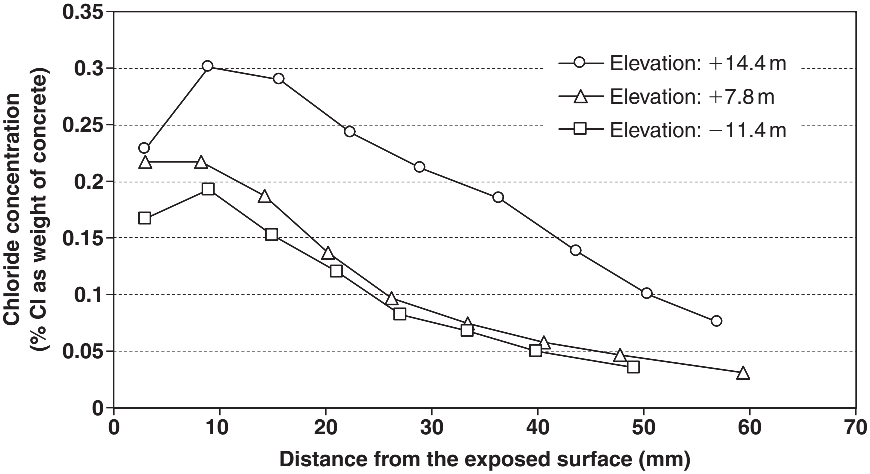


Figure 2.59 The Brent B Platform (1975) with observed chloride penetration after 20 years of exposure (source: Sengul and Gjørv (2007)).

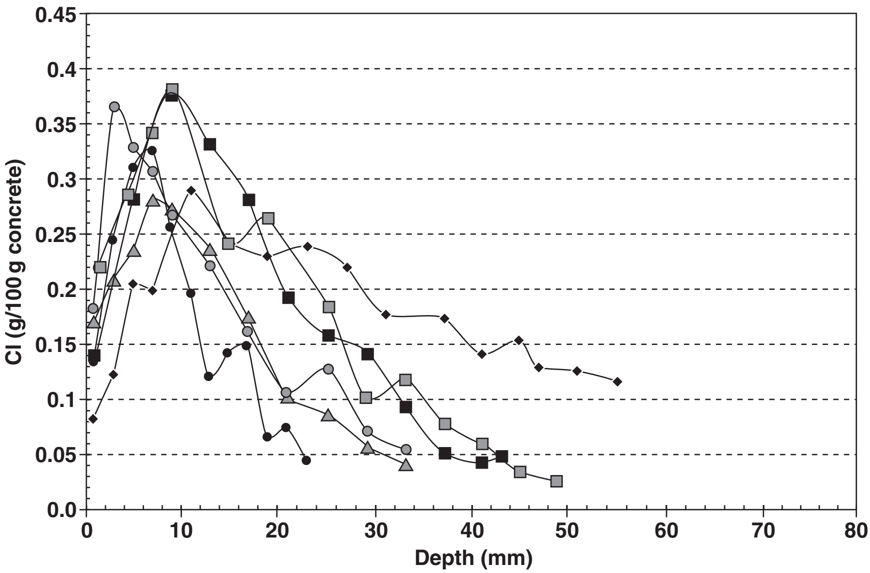


Figure 2.60 The Brent C Platform (1978) with observed chloride penetration below water after 17 years of exposure (source: Gjørv (2002)).

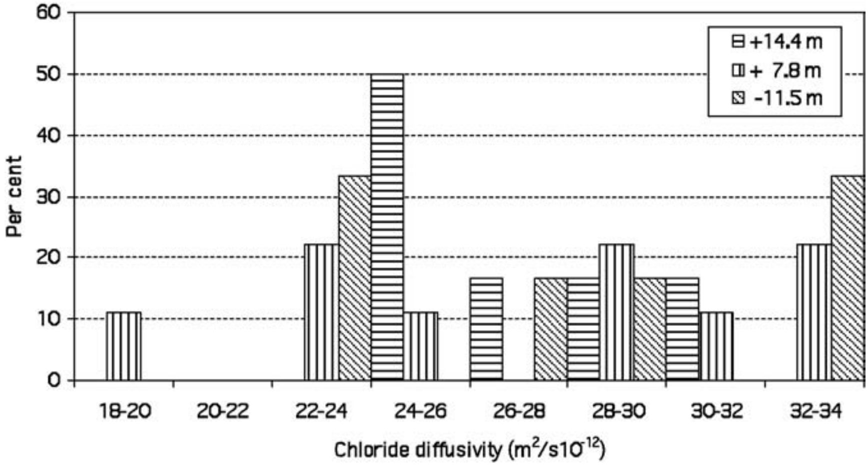


Figure 2.61 Observed chloride diffusivity in the utility shaft of the Brent B Platform (1975) (source: Årskog and Gjørv (2008)).

Figure 2.62 The consequences of corrosion in garages exposed to de-icing salt contamination have occasionally been quite severe (source: Courtesy of Sten H. Vælitalo).

As may be seen from the observed chloride penetration in both the Brent B and the Brent C Platforms, the chloride penetration showed a very high scatter, even in the continuously submerged parts of the structures. Also a number of other concrete platforms in the North Sea, however, have shown a high scatter of observed chloride penetration, indicating a high inhomogeneity of achieved concrete quality.

2.5 Other structures

In addition to the field performance of the various types of concrete structures outlined above, there are also a variety of other types of concrete structures which are showing problems due to an uncontrolled durability. In the marine environments, there are a number of buildings and other facilities which are also suffering from chloride-induced corrosion. In addition to all the highway bridges, there are a large number of garages which are suffering from severe corrosion problems due to de-icing salt, the consequences of which have occasionally been quite severe (Figure 2.62).

When the cars let in the de-icing salt solution, thus contaminating the concrete decks in the garages, a pattern of corrosion activities in the parking bays is often observed, as is shown in Figure 2.63. Such corrosion activity maps, which are the combined result of potential and resistivity measurements along the concrete deck, have proved to be a very efficient tool for condition assessment of existing concrete structures in severe environments (Pruckner, 2002; Pruckner and Gjørv, 2002).

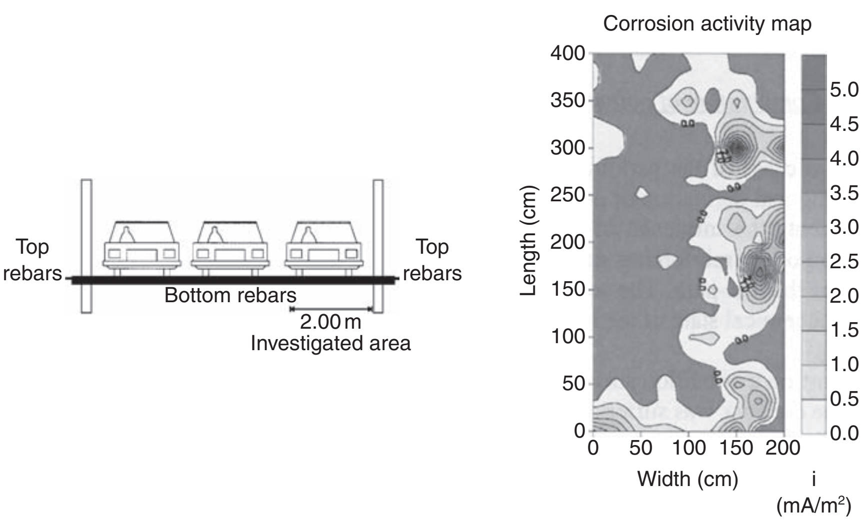


Figure 2.63 Typical pattern of corrosion activities in the parking bays of a garage (source: Pruckner (2002)).

3 Corrosion of embedded steel

3.1 General

Embedded steel in concrete is normally well protected against corrosion, and this is primarily due to the electrochemical passivation of the steel in the highly alkaline pore solution of the concrete. However, when the passivity of the steel partly or completely breaks down either due to concrete carbonation or presence of chlorides and the corrosion starts, this means that the electrochemical potential of the steel locally becomes more negative and forms anodic areas, while other portions of the steel which have the passive potential intact will act as catchment areas of oxygen and form cathodic areas. If the electrical resistivity of the concrete is also sufficiently low, a complex system of galvanic cell activities develops along the steel. In all of these galvanic cells, a flow of current takes place, the amount of which determines the rate of corrosion. Although the size and geometry of the anodic and cathodic areas in the galvanic cells are also important factors, the rate of corrosion is primarily controlled by the electrical resistivity of the concrete and the availability of oxygen.

For dense, high-quality concrete of proper thickness, carbonation-induced corrosion of the embedded steel is not considered to represent any practical problem, and for moist environments, carbonation of the concrete is considered even less of a problem. For concrete structures in severe chloride-containing environments, however, it appears from Chapter 2 that it may be just a question of time before detrimental amounts of chloride reach embedded steel even through thick covers of high-quality concrete. In addition, even a high-quality concrete may show a high inhomogeneity after being placed in the structure.

3.2 Chloride penetration

For concrete structures in chloride-containing environments, the penetration of chloride can take place in different ways. Through uncracked concrete, the penetration takes place mainly by capillary absorption and diffusion. When a relatively dry concrete is exposed to salt-water, the concrete will absorb the salt-water relatively fast, and intermittent wetting and drying can successively accumulate high concentrations of salt in the concrete. For concrete structures in moist marine environments, however, it was discussed in Chapter 2 how intermittent exposure to splashing of seawater mainly gives fluctuating moisture content limited to an outer layer of the concrete. For conditions along the North Sea coast, the moisture content shows only small fluctuations in the outer layer of the concrete, while it appears to be more consistently high deeper below the surface (Figure 3.1). For many concrete structures in Norwegian marine environments, however, constantly high moisture contents are also typically observed in the outer layer of the concrete. For Norwegian concrete coastal bridges, a degree of capillary saturation varying from 80 to 90 per cent in the outer 40 to 50 mm of the concrete was reported by Holen Relling (1999). Thus, for the thickness of concrete cover typically specified for concrete structures in severe chloride-containing environments, the moisture content in the concrete may be quite high, and hence diffusion appears to be the most dominating transport mechanism for the penetration of chloride.

Although penetration of chloride into concrete has been the subject of extensive investigations for a long period of time both from a theoretical and applied point of view, it still appears to be a very difficult issue (Poulsen and Mejlbro, 2006). Even a pure diffusion of chloride ions in concrete is a very complex and complicated transport process (Zhang and Gjørv, 1996). Therefore, when Ficks 2. Law of Diffusion is often applied

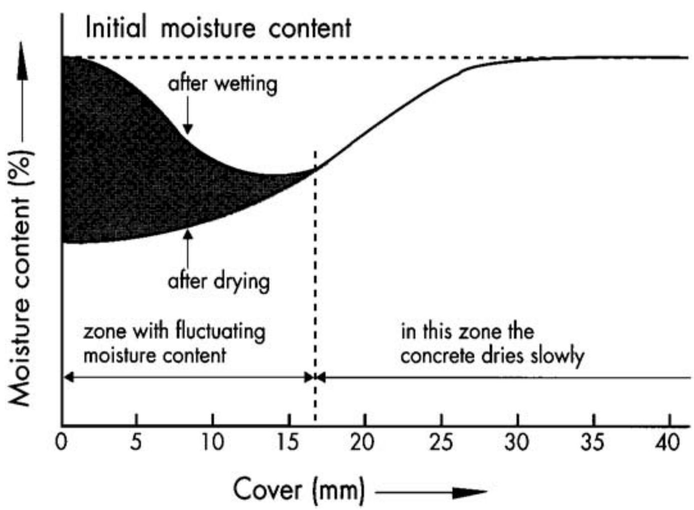


Figure 3.1 Outer layer of concrete with changing moisture content under splash water conditions along the North Sea coast according to Bakker (1992) (source: Bijen (1998)).

for calculation of rates of chloride penetration, it should be noted that such a calculation is based on a number of assumptions and a very rough simplification of the real transport mechanism.

For a general evaluation of the resistance of concrete to chloride penetration, a number of factors have to be considered. Although the water/binder ratio of the concrete is a very important factor for controlling the resistance to chloride penetration, it is well documented in the literature that the type of cement or binder system may be as important as or even more important than the water/binder ratio. Thus the superior effect of granulated blast furnace slag cements on the resistance of concrete against chloride penetration has been well documented from both laboratory experiments and extensive field experience covering a period of more than 100 years (Bijen, 1998). The beneficial effect of both natural and industrial pozzolanic materials such as condensed silica fume, fly ash and rice husk ash is also well documented (Gjørv, 1983; Berry and Malhotra, 1986; Malhotra *et al.*, 1987; FIP, 1998; Malhotra and Ramezanianpour, 1994; Gjørv *et al.*, 1998a).

Figure 3.2 shows the resistance to chloride penetration of four different types of cement for the same concrete composition at a water/binder ratio of 0.45. These cements include one high performance portland cement of type CEM I 52.5 LA (HPC), one fly ash cement of type CEM II/A V 42.5 R with approximately 18 per cent fly ash (PFA) and two blast furnace slag cements of type CEM II/B-S 42.5 R NA with approximately 34 per cent slag (GGBS1) and type CEM III/B 42.5 LH HS (GGBS2) with approximately 70 per cent slag, respectively. The testing was carried out using the

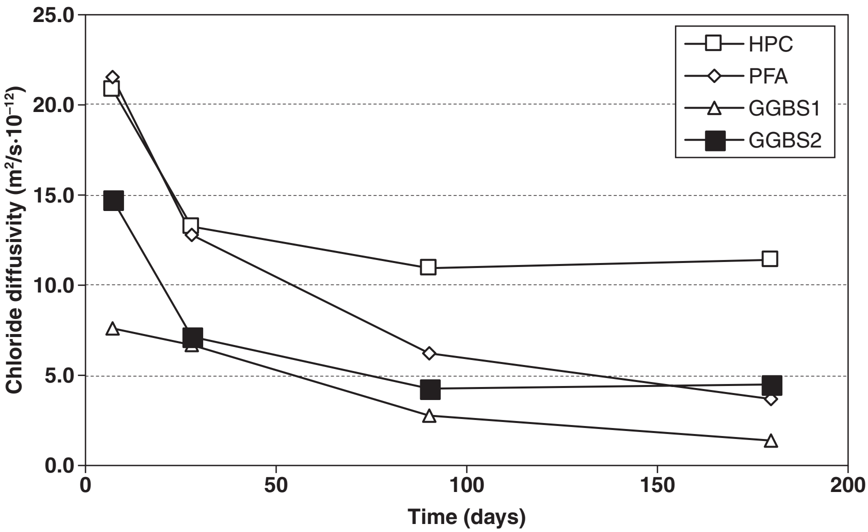


Figure 3.2 Effect of cement type on the resistance of concrete to chloride penetration at a water/binder ratio of 0.45 (source: Årskog *et al.* (2007)).

RCM method (NORDTEST, 1999) on water-cured concrete specimens for periods of up to 180 days. In order to compare the same types of cement in a more dense concrete, the same cements were also tested in combination with 10 per cent silica fume by weight of cement at a water/binder ratio of 0.38 (Figure 3.3).

The results in Figures 3.2 and 3.3 demonstrate that the two slag cements gave a distinctly better resistance against chloride penetration than that of the fly ash cement and a substantially better resistance than that of the portland cement. In the more dense concrete with silica fume, the difference between the various types of cement was smaller than in the more porous type of concrete. However, even in the densest concrete, it was a distinct difference between the slag cements and the two other cements. In addition, both types of slag cement showed a very high early age resistance compared to that of the other types of cement, and this may be important for early age exposure during concrete construction in severe marine environments as discussed in Chapter 2. In order to also test the effect of curing temperature, the same types of cement were tested at curing temperatures of 5 and 12°C. Since ordinary portland cements have been so widely used for marine environments over many years, the 34 per cent slag cement was replaced by an ordinary portland cement of type CEM I 42.5 R (OPC), while all tests were carried out at the water/binder ratio of 0.45. As may be seen from both Figures 3.4 and 3.5, the 70 per cent slag cement (GGBS2) gave a substantially higher early age resistance than both the fly ash cement (PFA) and the two pure portland cements (HPC and OPC). Thus at 5°C, the 28-day chloride diffusivity for the slag

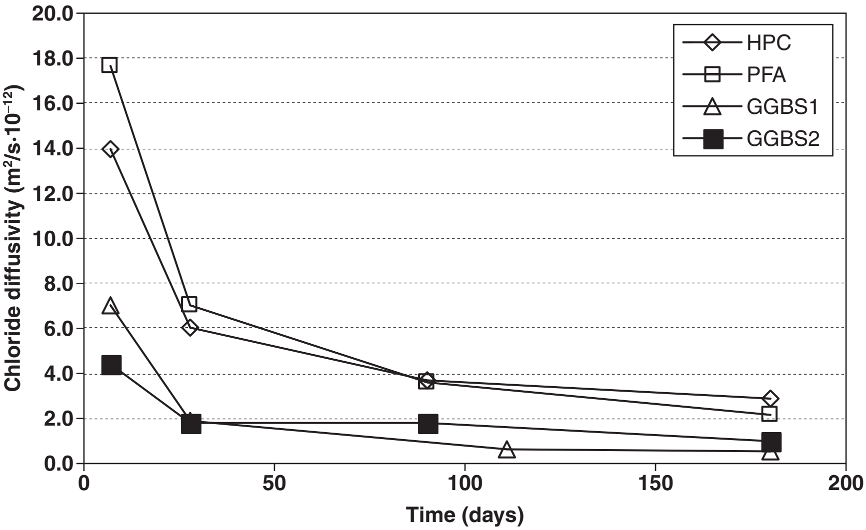


Figure 3.3 Effect of cement type on the resistance of concrete to chloride penetration at a water/binder ratio of 0.38 (source: Årskog *et al.* (2007)).

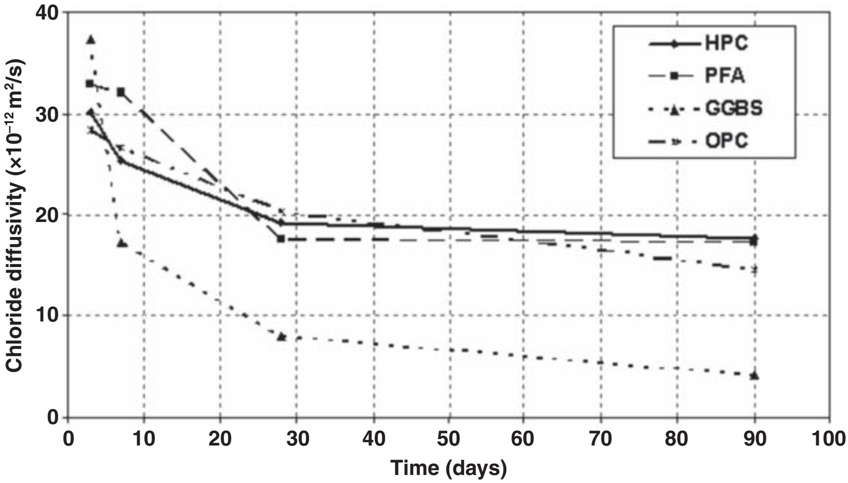


Figure 3.4 Effect of cement type on the resistance of concrete to chloride penetration at a curing temperature of 5°C (source: Liu and Gjørv (2004)).

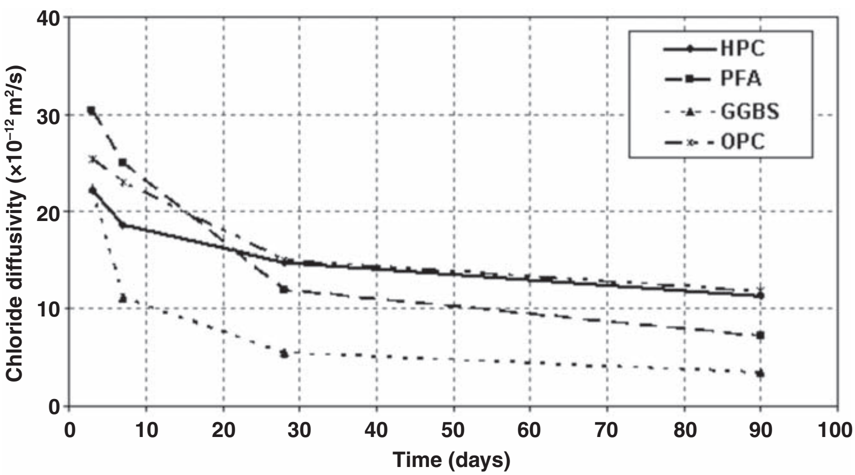


Figure 3.5 Effect of cement type on the resistance of concrete to chloride penetration at a curing temperature of 12°C (source: Liu and Gjørv (2004)).

cement was $7.9 \times 10^{-12}\,m^2/s$ compared to $17.4 \times 10^{-12}\,m^2/s$ for the fly ash cement (PFA) and 19.3 and $20.3 \times 10^{-12}\,m^2/s$ for the two pure portland cements (HPC and OPC), respectively. After 90 days of curing, the corresponding values were 4.1, 17.2, 17.6 and $14.5 \times 10^{-12}\,m^2/s$, respectively. These results clearly demonstrate how concrete structures produced with pure portland cements or fly ash cements in severe marine environments at low temperatures are more vulnerable to early age chloride penetration than those produced with slag cements.

It should be noted that different types of portland cement may also show a different resistance to chloride penetration depending on the C_3A content of the cement. Thus, after 100 years of exposure of large concrete units in a breakwater in Japan, depths of chloride penetration of only 50 to 80 mm were observed (Gjørv *et al.*, 1998b). These concrete units had been produced with a pure portland cement with a very high C_3A content of 15–16 per cent at a water/binder ratio of approximately 0.34. From Figure 3.6, however, it appears that different portland cements with more moderate amounts of C_3A show only small differences in the resistance to chloride penetration compared to that of other types of cement. The results in Figure 3.6 were based on field tests with concrete at a water/binder ratio of 0.40 submerged in seawater for six months.

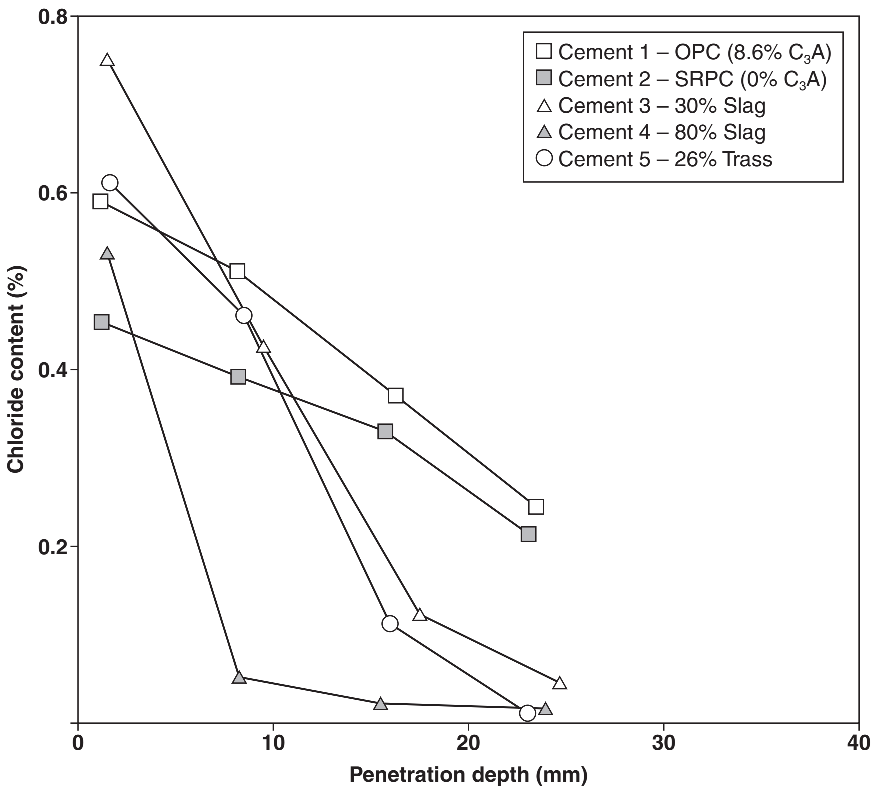


Figure 3.6 Effect of cement type on chloride penetration (by weight of cement) after six months of exposure to seawater (source: Gjørv and Vennesland (1979)).

In recent years, there has been a rapid growing trend in the use of more blended portland cements compared to that of pure portland cements. Replacement materials such as fly ash and blast furnace slag are occasionally also used as separate additions to the concrete mixture, and the question is often raised as to the effect of such replacements on the resistance to chloride penetration. While blast furnace slags are hydraulic binders, most types of fly ash are pozzolanic materials, the main effect of which depends on the amount of $Ca(OH)_2$ available for the pozzolanic reaction. Thus, as the pure portland cement was replaced by more than about 30 per cent low-calcium fly ash, it appears from Figure 3.7 that there was only a very little or no further effect on the resistance to chloride penetration. These results were based on the Rapid Chloride Permeability (RCP) test method ASTM C 1202–05 (ASTM, 2005) on concrete with a water/binder ratio of 0.35 after one year of water curing. Although the portland cement can be replaced by larger amounts of blast furnace slag compared to that of fly ash, also for slag there is an upper limit above which the observed effect is very small. Thus, in Figure 3.8, an ordinary portland cement of type CEM I 42.5 R was replaced by 40, 60 and 80 per cent of blast furnace slag with a Blaine fineness of 5000 cm^2/g. Based on concrete with a water/binder ratio of 0.40 and water curing of up to one year, the resistance of the concrete to chloride penetration was tested by use of the RCM test method (NORDTEST, 1999). After 28 days of water curing, it may be seen from Figure 3.8 that increasing amounts of slag successively reduced the chloride diffusivity from 11.2 to 4.9, 3.6 and $2.3 \times 10^{-12}\,m^2/s$, respectively, while after 365 days, the diffusivity of the slag concretes varied from 3.0 to $1.2 \times 10^{-12}\,m^2/s$ compared to $7.0 \times 10^{-12}\,m^2/s$

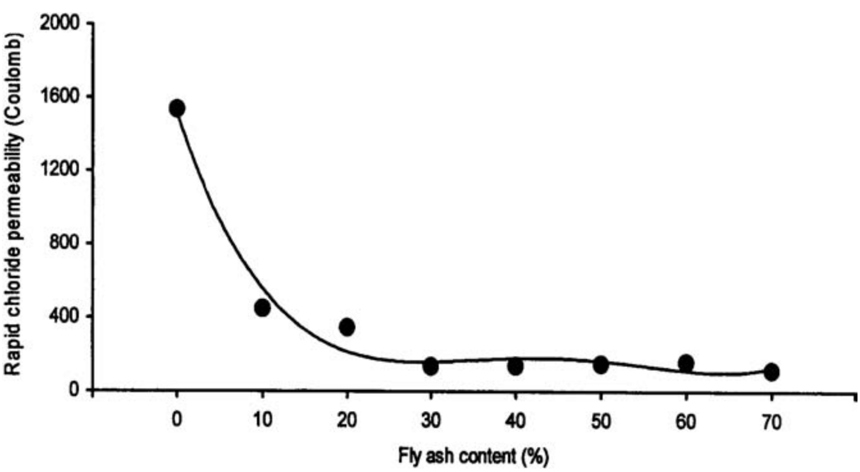


Figure 3.7 Effect of fly ash on the rapid chloride permeability of concrete (source: Sengul (2005)).

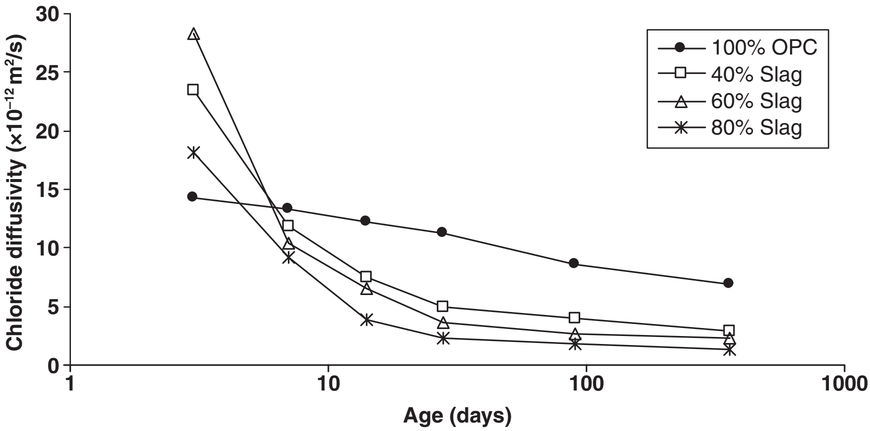


Figure 3.8 Effect of blast furnace slag on the chloride diffusivity of concrete based on RCM testing (source: Sengul and Gjørv (2007)).

for that of the pure portland cement. In parallel, a diffusion testing by use of the immersion test NT Build 443 (NORDTEST, 1995) also showed a similar effect of the increased replacements of the portland cement by slag (Figure 3.9). After 28 days of water curing and a further 35 days of immersion in the salt solution for this particular type of test, the chloride diffusivity was reduced from $12.8 \times 10^{-12}\,m^2/s$ for the pure portland cement to 4.0, 3.1 and $3.2 \times 10^{-12}\,m^2/s$ for the 40, 60 and 80 per cent slag contents, respectively. All these test results are in general agreement with other results reported in the literature (Bijen, 1998). Thus, in Figure 3.10, there is hardly

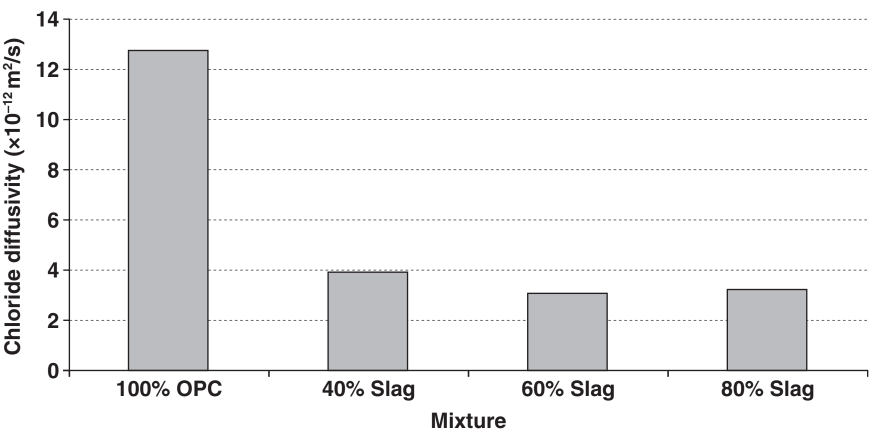


Figure 3.9 Effect of blast furnace slag on the chloride diffusivity of concrete based on the immersion type of diffusion testing (source: Sengul (2005)).

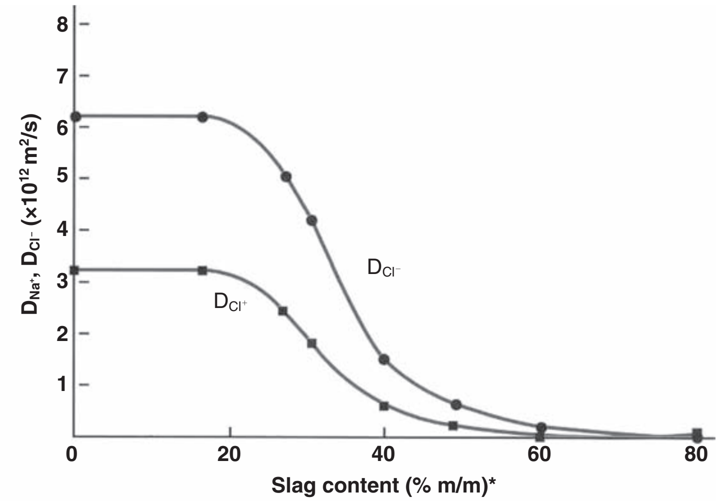


Figure 3.10 Effect of blast furnace slag on the diffusivity of pure cement paste according to Brodersen (1982) (source: Bijen (1998)).

any effect on the chloride diffusivity for slag contents of less than 25 per cent, while for slag contents of 25 to 50 per cent there is a large drop in diffusivity, beyond which there is still a decrease but only to a smaller extent.

For many of the replacement materials for portland cement, further beneficial effects on the resistance to chloride penetration by increased refinement of the replacement materials can be reached. Thus, by using blast furnace slag with a Blaine fineness of up to approximately 16,000 cm^2/g, an extremely high resistance to chloride penetration was observed (Gjørv *et al.*, 1998c).

Although the chemical reactions of pozzolanic materials and blast furnace slags are quite different, the resulting effect on both microstructure and chemical composition of the pore solution in the concrete appears to be quite similar. In both cases, a substantially higher formation of CSH gel with a higher amount of small gel pores (<30 nm) and a smaller amount of large capillary pores compared to that of pure portland cements are formed. The substantially smaller amount of free lime in the hardened cement paste also changes the chemical composition of the pore solution in a beneficial way for the chloride diffusivity. However, a reduced amount of free lime in the hardened cement paste will also reduce the alkalinity of the pore solution, which reduces the critical level of chloride concentration for breaking the passivity of embedded steel.

As already pointed out above, even a pure diffusion of chloride ions into concrete from an external salt solution is a very complex transport process. As part of this complex transport process, the chemical composition of the external salt solution is also very important for the resulting chloride penetration into concrete (Theissing *et al*., 1975). This is clearly demonstrated in Figure 3.11, where a deeper chloride penetration takes place from a salt solution based on $CaCl_2$ compared to that of NaCl, the external chloride concentration being the same. Therefore, the use of de-icing salts based on $CaCl_2$ may give a more severe chloride penetration into concrete compared to that from the NaCl in seawater.

3.3 Passivity of embedded steel

The pore solution of concrete normally attains an alkalinity level in excess of pH 13. In the presence of oxygen, this alkaline solution forms a thin oxide film on the steel surface which very efficiently protects all embedded steel from corrosion. However, the integrity and protective quality of this

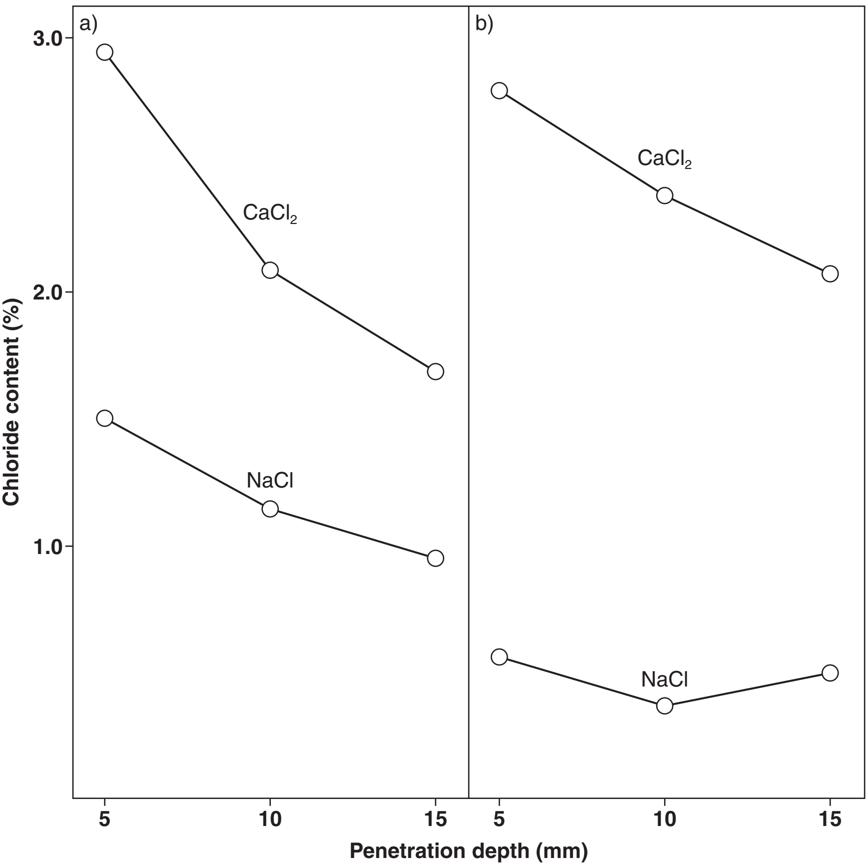


Figure 3.11 Chloride penetration (by weight of cement) into cement paste from two different types of salt solution of the same chloride concentration: (a) after two days of hydration, and (b) after 40 days of hydration before exposure (source: Trætteberg (1977) in Gjørv and Vennesland (1987)).

film depends on a number of factors such as oxygen availability and alkalinity of the pore solution, and the lower the oxygen availability and the lower the alkalinity, the thinner the protective film and the lower the protective quality. Experience indicates that the pH of the pore solution should never drop below a level of approximately 11.5 in order to provide a proper electrochemical protection (Shalon and Raphael, 1959). As soon as the pH drops to approximately 9.0, however, the protective oxide film is completely dissolved and broken down.

For concrete based on pure portland cements, the high alkalinity is due to small quantities of the readily soluble NaOH and KOH. The cement paste also contains a large proportion of the less soluble $Ca(OH)_2$ which buffers the system in such a way that the pH never drops below a level of approximately 12.5. For granulated blast furnace slag cements and portland cements blended with pozzolans such as fly ash or condensed silica fume, however, certain amounts of the $Ca(OH)_2$ are bound up to form new CSH, and hence the reserve basisity is correspondingly reduced. This is demonstrated in Figure 3.12, where increased additions of silica fume successively reduced the basisity of the concrete.

Even for cements with the highest reserve basisity, the alkalinity may still be reduced either by leaching of the alkaline substances with water or by neutralization after carbonation with CO_2. The pore solution in carbonated concrete only has a pH of approximately 8.5.

As previously discussed, carbonation of a dense high-quality concrete is not considered to represent any practical problem. However, the protective

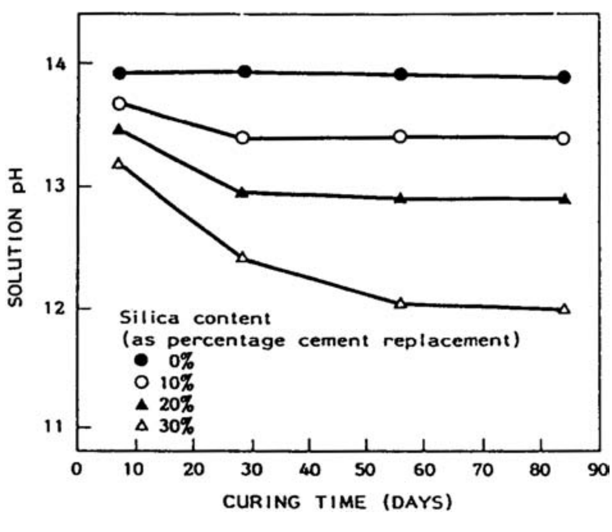


Figure 3.12 Effect of increased additions of silica fume on the basisity of concrete based on portland cement (source: Page and Vennesland (1983)).

oxide film can easily be destroyed by the presence of chloride in the concrete, and the thinner the oxide film, the fewer amounts of chloride are needed in order to destroy the protective film. It is well known that even very small chloride contents in the pore solution may destroy the passivity of the steel. However, only a small part of the total amount of chloride in concrete is dissolved in the pore solution. Some of the chloride is chemically bound and some is physically bound, while the rest exists in the form of free chloride dissolved in the pore solution. It is only these free chlorides in the pore solution which can destroy the protective film and thus start a pitting corrosion. Since a very complex equilibrium between the different forms of chloride in the concrete exists, the amount of free chloride in the pore solution of a given concrete both depends on the degree of water saturation and the temperature of the concrete.

Different types of binder system may also significantly affect both the pore solution alkalinity and the amount of chemically and physically bound chloride in the concrete. Whether the chlorides have been mixed in already during concrete production or have penetrated the concrete later on may also affect the above complex relationship. If the mixed-in chloride content has been too high already from the beginning, the protective film on the embedded steel may never have been formed.

The threshold concentration of chlorides required to destroy the passivity of embedded steel has been the subject of numerous investigations (Bertolini *et al.*, 2004). A number of different measurement techniques have been applied and a number of different threshold values reported, typically varying from 0.02 to 3.04 per cent of total chloride content by weight of cement (Angst, 2008). There are a number of factors affecting the chloride threshold such as pH of the pore solution, potential of the steel and the local conditions along the interface between the concrete and the embedded steel (Glass and Buenfeld, 1997, 2000). As a result, it is not possible to express any unique chloride threshold value for pitting corrosion of embedded steel in concrete. However, only very small amounts of chloride are needed to break the passivity, and as soon as the passivity is broken corrosion starts, and the rate of corrosion is controlled by a number of other factors.

3.4 Corrosion rate

General

For concrete structures exposed to the atmosphere, experience indicates that the corrosion rate may vary from several tens of μm/year to localized values of up to 1 mm/year depending on moisture conditions and chloride content in the concrete (Bertolini *et al.*, 2004). Different temperatures may also affect the corrosion rate very differently (Andrade and Alonso, 1995; Østvik, 2005). In the case of concrete structures permanently submerged in seawater, it was shown in Chapter 2 that the corrosion rate was so low that the corrosive attack was negligible even after a service period of more than 60 years (Gjørv and Kashino, 1986). In Chapter 2, it was also shown how ongoing corrosion in certain parts of the concrete structure had effectively protected other parts of the structure cathodically. Therefore, since both the geometry of the structure and the local environmental conditions are very important factors for the rate of deterioration, it appears very difficult to mathematically predict the long-term effect of an ongoing corrosion on the load-carrying capacity of a concrete structure. For the corrosion to develop into a serious deteriorating process, however, the electrical resistivity of the concrete is a very important factor.

Electrical resistivity

Since the electrical current which is flowing in all the galvanic cells along the embedded steel is transported by charged ions through the concrete, the electrical resistivity of the concrete both depends on concrete permeability, amount of pore solution and the ion concentration of the pore solution. Thus, by decreasing the water/cement ratio from 0.7 to 0.5, it may be seen from Figure 3.13 that the electrical resistivity increased by a factor of more than two for mortar, while for concrete of the same water/cement ratio there was only a small effect. These results clearly demonstrate the effect of permeability on electrical resistivity. When the degree of water saturation of the concrete was successively decreased from 100 per cent to somewhat less than 20 per cent RH, Figure 3.14 shows that the electrical resistivity increased from approximately 7×10^3 to approximately 6×10^6 ohm cm. These results clearly demonstrate the very important effect of the moisture conditions of the concrete and thus the amount of pore solution available for the transport of charged ions. When the portland cement in the concrete was successively replaced by condensed silica fume, both the permeability and the ion concentration of the pore solution were significantly affected, the effect of which is demonstrated in Figure 3.15.

If the electrical resistivity of the concrete becomes sufficiently high, a very small or negligible rate of corrosion may take place. In Chapter 2, a threshold value of $50–70 \times 10^3$ ohm cm for the concrete in the San Mateo–Hayward Bridge was reported, beyond which only a very small corrosion rate was observed (Gewertz *et al.*, 1958). Even for the combination of broken passivity and low electrical resistivity, however, the rate of corrosion may still be very low or negligible depending on the availability of oxygen.

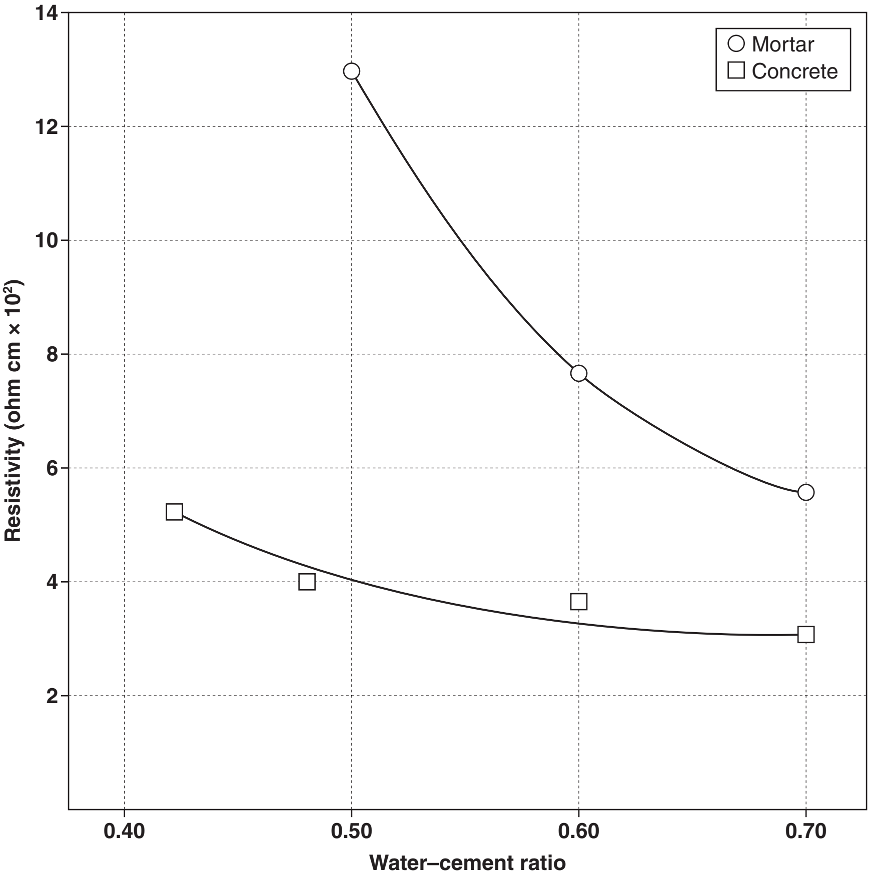


Figure 3.13 Effect of water/cement ratio on the electrical resistivity of concrete (source: Gjørv *et al.* (1977a)).

Oxygen availability

The availability of oxygen depends on several factors. While the concentration of oxygen in the atmosphere is approximately 210 ml/l, the maximum concentration of oxygen in water available for submerged concrete structures is only 5–10 ml/l. The rate of oxygen diffusion through the concrete also depends on whether the oxygen is in a gaseous state or dissolved in water. Although both the permeability and thickness of the concrete cover affect oxygen availability, it may be seen from Figure 3.16 that the degree of water saturation of the concrete is a dominating factor. For oxygen to take part in the electrochemical cathode reaction, it must be in a dissolved state. For concrete submerged in water, Figure 3.17 demonstrates that the thickness of the concrete cover may only

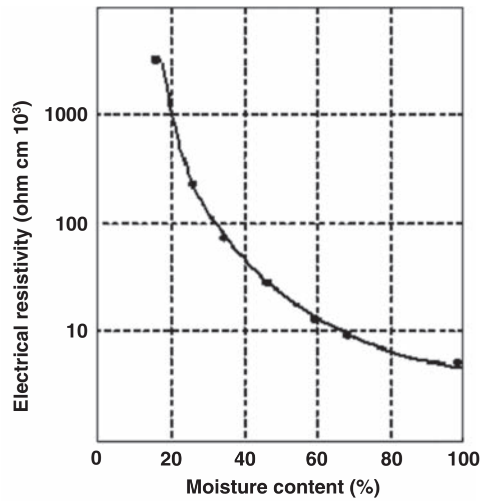


Figure 3.14 Effect of moisture conditions on the electrical resistivity of concrete (source: Gjørv *et al.* (1977a)).

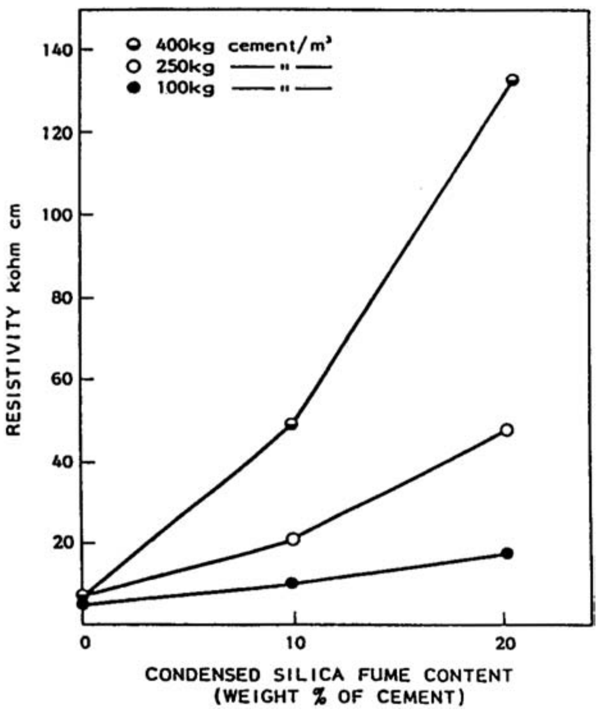


Figure 3.15 Effect of increased additions of silica fume on the electrical resistivity of concrete (source: Vennesland and Gjørv (1983)).

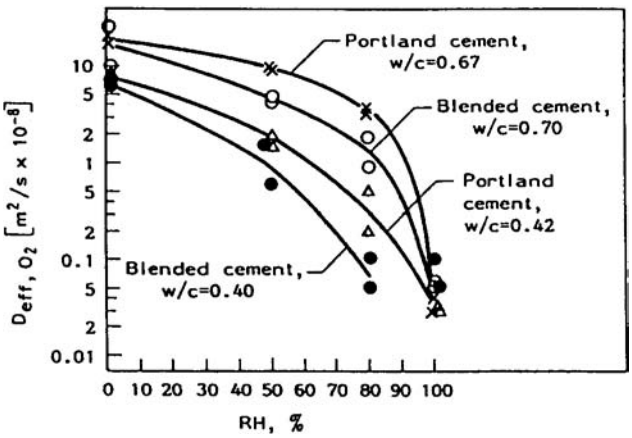


Figure 3.16 Effect of water saturation on the rate of oxygen diffusion (source: Tuutti (1982)).

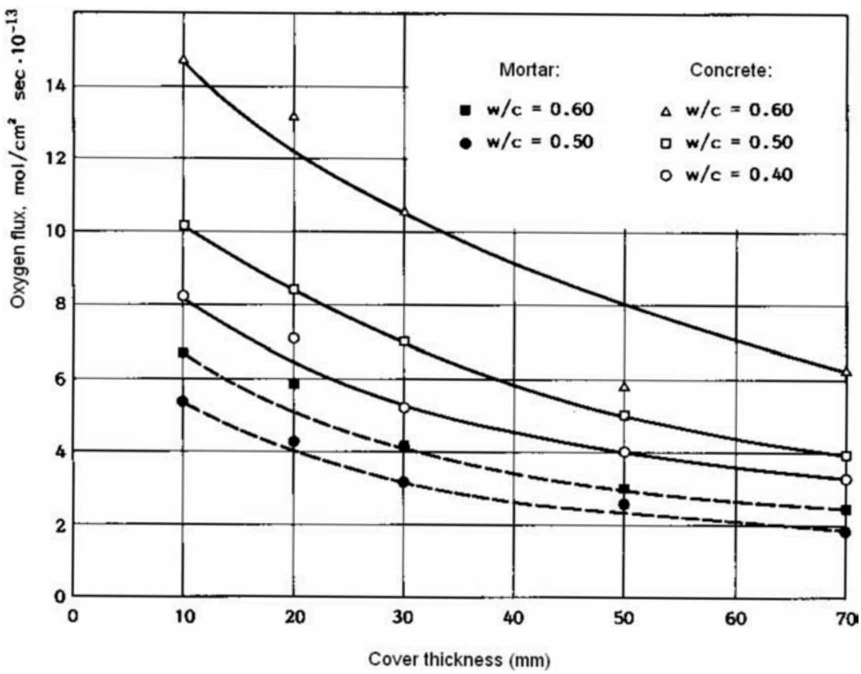


Figure 3.17 Effect of concrete cover on the rate of oxygen diffusion through concrete submerged in water (source: Gjørv *et al.* (1986b)).

have a minor effect on oxygen availability. Thus, for a concrete with a water/cement ratio of 0.40, a reduced concrete cover from 70 to 10 mm only reduced the flux of oxygen by a factor of approximately 2.6. Such results indicate that there is a transition phase between the concrete and the embedded steel which acts as a barrier to the oxygen, and this may explain why the thickness of the concrete cover is not so important for the availability of dissolved oxygen.

While corrosion of embedded steel may not represent any practical problem in submerged concrete structures due to lack of dissolved oxygen, macro-cell corrosion may still develop due to availability of oxygen from the inside of submerged hollow concrete structures (Bertolini *et al.*, 2004). For concrete structures above water, however, the oxygen is so readily available that it does not represent any limiting factor for high corrosion rates to develop.

3.5 Cracks

For concrete structures with cracks in the concrete cover, the electrolytic conditions for the start of corrosion may be significantly affected. For cracked concrete, it is reasonable to assume that increased crack widths give increased penetration of corrosive substances such as air, water and salts, and hence increased probability for corrosion. Based on detailed procedures for the calculation of crack widths, therefore, most codes and recommendations specify upper limitations for characteristic crack widths of typically 0.4 mm for non-aggressive environments and 0.3 mm for more aggressive environments (Standard Norway, 2003).

In spite of the large number of experiments reported in the literature, however, it is not possible to come up with a simple relationship between crack width and probability of corrosion for a given structure in a given environment. Extensive research has revealed that a number of factors affect the possibility of corrosion, and both the possible mechanisms and the practical consequences have been the subject of extensive discussion (Gjørv, 1989). As shown in Figure 3.18, the geometry of the cracks may be very complex, and different effects of the cracks running parallel or perpendicular to the steel bars are observed. In addition, cracks in different types of environment and due to different loading conditions, such as static or dynamic loading, may also affect the observed corrosion differently.

Most of the extensive investigations on the effect of cracks reported in the literature have been carried out on concrete exposed to various types of atmospheric environments. Even based on comprehensive investigations, however, it has not been possible to establish a simple relationship between crack width and development of corrosion. Very often, a certain effect of the crack width could be observed when the cracked concrete was inspected at an early stage of exposure, while later on, the observed effect could be very small or almost negligible.

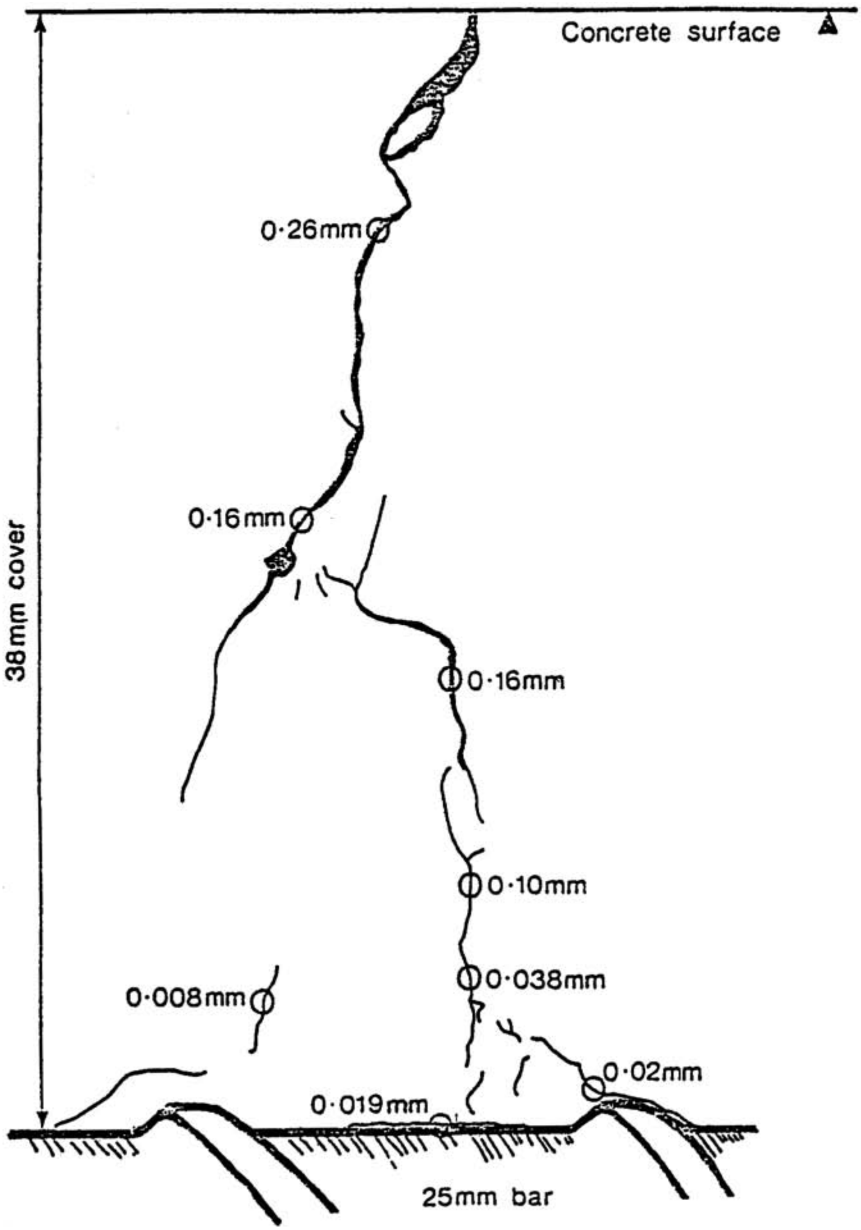


Figure 3.18 The geometry of cracks may be very complex (source: Beeby (1977)).

For concrete structures continuously submerged in seawater, the effect of cracks cannot be evaluated without also taking into account the galvanic coupling between the exposed steel in the crack and the larger portions of the embedded steel system (Gjørv, 1977). Thus, results obtained on the basis of small concrete elements cannot necessarily be extrapolated to large submerged concrete structures.

By simulating the galvanic coupling and the corrosion mechanism which may take place in large submerged concrete structures, laboratory investigations revealed that the observed corrosion in the cracked concrete was significantly less than expected (Vennesland and Gjørv, 1981). This was also true for similar tests carried out under dynamic loading conditions (Espelid and Nilsen, 1988).

Depending on the environmental conditions, it appears that the rate of corrosion is reduced over time by a clogging up of the crack by both corrosion products and other reaction products. In particular, this appears to be effective for cracked concrete under submerged conditions in seawater. Smaller areas of the freely exposed steel in the crack may also be cathodically protected by the adjacent embedded steel. However, if the areas of the freely exposed steel in galvanic coupling with embedded steel in submerged concrete structures becomes too high, the rate of corrosion may be significant due to extremely high cathode-to-anode area ratios (Gjørv, 1977).

3.6 Galvanic coupling between freely exposed and embedded steel

For large submerged concrete structures such as those used for oil and gas exploration offshore, there are a variety of external steel components such as skirts, pipes, supports and fixtures which are in electrical contact with the embedded reinforcing steel system. As the exposed external steel will then be anodic against the embedded steel which will be cathodic, special problems with both the corrosion rate and the corrosion protection of such external steel components may arise (Gjørv, 1977). The rate of corrosion will primarily be controlled by the cathode-to-anode area ratio and by the cathode efficiency of the embedded steel. Hence, for cathodic protection of such freely exposed steel, the current demand will also be controlled by both the area of the cathode and the cathode efficiency of all the embedded steel.

For cathodic protection of freely exposed steel attached to submerged concrete structures, the cathode-to-anode area ratio both depends on the structural design and on the internal electrical continuity within the reinforcing steel system. Although the cathode-to-anode area ratio must be evaluated on an individual basis, measurements on large offshore concrete platforms indicate that such heavily reinforced structures have a very good electrical continuity within more or less the whole embedded rebar system.

Hence, the cathode-to-anode area ratio for a small area of external steel components may be extremely high relative to the area of all the embedded steel. The cathode efficiency which depends on the rate of oxygen diffusion through the concrete cover to the embedded steel is also an important factor. Based on laboratory investigations on submerged concrete of the same quality as that used for concrete platforms in the North Sea, oxygen availability with flux values of up to $0.5 \times 10^{13}\,\mathrm{mol}\,O_2$ per second and cm^2 were observed (Gjørv *et al*., 1986b). In the field, however, experience from existing concrete platforms in the North Sea has shown that both marine growth and biological activities successively reduce the oxygen availability. Only at an early stage of exposure have field investigations revealed a high current drain to the embedded steel and hence a high consumption rate of the sacrificial anodes (Espelid, 1996).

4 Other deteriorating processes

4.1 General

In addition to the electrochemical corrosion of embedded steel, there are also a number of other types of deteriorating processes which may cause problems for the durability of concrete structures in severe environments. For many existing concrete structures, both freezing and thawing and alkali–aggregate reaction represent some severe durability problems. In addition, chemical processes such as sulphate attack, seawater attack and leaching may also represent some problems. Although most of these deteriorating processes may represent potential durability problems for all concrete structures in severe environments, experience has shown that it is much easier to control and avoid such durability problems by taking necessary precautions and following existing guidelines at an early stage of planning. Extensive experience has shown that it is not the disintegration of the concrete itself but rather the corrosion of embedded steel which represents the most critical threat and greatest challenge to the durability and long-term performance of concrete structures in severe environments (Gjørv, 2002). Nevertheless, for all new important concrete structures, it is of the greatest importance to make a proper assessment of all potential durability problems and take the necessary precautions.

For most deteriorating processes, the permeability of the concrete is a key factor governing the rate of deterioration. Even for environments with high sulphate contents, field investigations have shown that ordinary portland cements even with high C_3A content may give the same resistance to sulphate attack as that of low C_3A-containing cements provided that the permeability of the concrete is sufficiently low (Figure 4.1). Since pure portland cements produce large amounts of calcium hydroxide during cement hydration and this constituent is very soluble, pure portland cements generally make the concrete more vulnerable to deterioration than many other types of binder system. Therefore, cements and binder systems based on blast furnace slag or portland cements blended with various pozzolanic materials generally make the concrete more resistant from a durability point of view. In spite of this, both freezing and thawing and

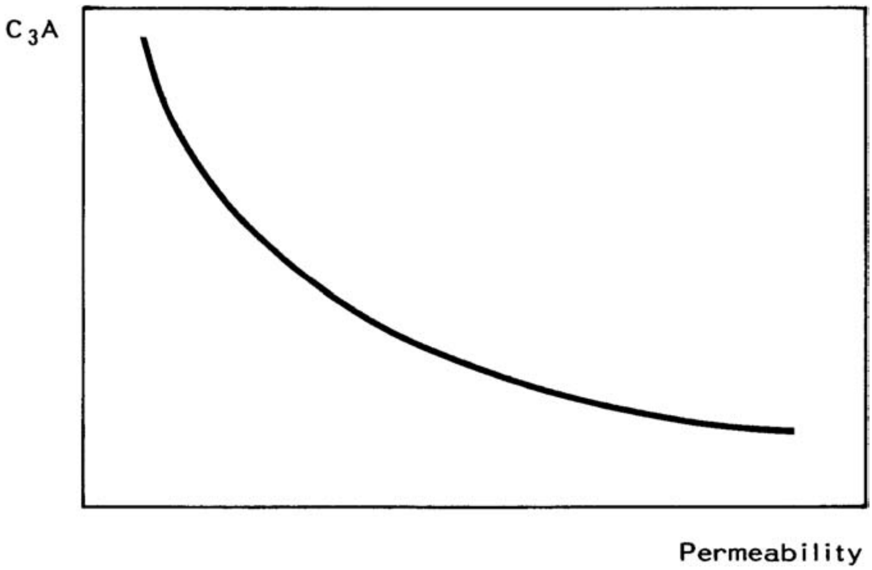


Figure 4.1 Combinations of C_3A content and permeability for obtaining the same sulphate resistance of concrete based on pure portland cements (source: Gjørv (1983)).

alkali–aggregate reaction may cause severe durability problems if no precautions are taken.

4.2 Freezing and thawing

Although the resistance of concrete to freezing and thawing has been the subject of extensive research throughout a major part of the twentieth century, this type of deterioration still causes durability problems in many countries. This is primarily due to the widespread use of de-icing salt on concrete pavements and highway bridges. By comparing the field performance of concrete structures in various environments, it appears that frost action from highway environments is far more aggressive to the concrete compared to that of marine environments (Petersson, 1995).

A general problem with both specification and assessment of the frost resistance of concrete is the lack of correlation between existing test methods and field performance. In addition, different test methods give different and conflicting test results. Another problem is the production of concrete with a good and stable air void system during transportation, handling and placing of the fresh concrete. This has been a problem for a long time, and extensive investigations of existing concrete structures with intentionally entrained air have revealed that no air was present, or what was there was inadequate by current standards (Klieger, 1980; Manning, 1980; Gjørv and Bathen, 1987). Even for simple concrete mixtures without any superplasticizer, the production of a good and stable air void system has been shown to be a problem, while the presence of many superplasticizers may increase the problem with achieving a proper air void system (Okkenhaug and Gjørv, 1992). The increasing use of binder systems based on fly ash represents an even bigger challenge. Most types of fly ash have an uncontrolled and varying content of carbon which tends to absorb the air entraining admixture, and some types of carbon compounds tend to absorb more of the admixture than others (Gebler and Klieger, 1983; Nagi *et al.*, 2007). In order to approach this problem, new types of air entraining admixtures are being introduced (Vanderwerf and Watson, 2007). In addition, much attention has been given in recent years to finding out whether a frost-resistant concrete without any air entrainment can be produced.

Okada *et al.* (1981), Foy *et al.* (1988) and Gagne *et al.* (1990) have all reported a good frost resistance of non-air entrained concrete with water/cement ratios in the range of 0.25 to 0.35. However, Malhotra *et al.* (1987), who also tested a number of concretes with different types of cement and water/cement ratios in the same range, concluded that air entrainment was necessary for these concretes to be frost resistant.

For salt scaling, there are even more conflicting results in the literature. Petersson (1984) reported that the deterioration of high strength concrete was small for the first 50 to 100 freeze–thaw cycles, while during the following 10 to 20 cycles, a rapid deterioration with total destruction took place. However, both Foy *et al.* (1988) and Gagne *et al.* (1990) as well as Hammer and Sellevold (1990) have shown that it is possible to produce non-air entrained concrete with proper resistance to salt scaling. These investigations were based on concretes with water/cement ratios of no more than 0.37 and testing of up to 150 freeze–thaw cycles.

Concrete based on different types of cement and binder system may also show varying resistance to frost action. Thus, granulated blast furnace slag cements have been shown to give a somewhat reduced frost resistance compared to that of other types of cement. Therefore, in some countries with frost action such as in Norway, the current concrete code does not include the use of any blast furnace slag cements (Standard Norway, 2003). However, recent investigations have shown that if the concrete is made dense enough, even slag cements with up to 70 per cent slag will give a very good frost resistance even without any air entrainment (Årskog and Gjørv, 2007). These investigations were based on concrete mixtures with 390 kg/m^3 blast furnace slag cement of type CEM III/B 42.5 LH HS in combination with 39 kg/m^3 silica fume (10 per cent), giving a water/binder ratio of 0.37. With one reference mixture without any air entrainment and two mixtures with increased air contents of up to about 3 per cent and 6 per cent, respectively, excellent frost resistance for all the concrete mixtures regardless of air entrainment was observed. For this investigation, parallel testing based on both the CDF method (Setzer *et al.*, 1996) and the Swedish method SS 137244–3 (Swedish Standard, 1995) with up to 112 freeze–thaw cycles was carried out. As a result, this type of concrete was approved for use in the new big city development project currently under construction in Oslo Harbour (Chapter 11). Only for poorer concrete qualities with water/binder ratios of 0.45 or more, extensive Dutch experience has shown that blast furnace slag cements may give a slightly reduced frost resistance compared to that of pure portland cements (Årskog and Gjørv, 2007). For concrete structures in severe environments, however, only concrete having water/binder ratios of 0.40 or less should be applied.

4.3 Alkali–aggregate reaction

Although the first durability problems caused by alkali–aggregate reaction (AAR) were observed on several Californian concrete structures and reported by Stanton already in 1940, it took a long time to recognize that AAR could also be a durability problem in many other countries. In 1947, Mielenz *et al.* (1947) published a blacklist of potential alkali-reactive aggregates and minerals, and in 1975, Gilliot (1975) suggested that AAR should be subdivided into the following three groups of reaction:

1 Alkali–silica reaction
2 Alkali–carbonate reaction
3 Alkali–silicate reaction.

Later on, several international publications showed that even more stable silicious rocks such as granite, quartzite, shist and sandstone could also be alkali reactive. Lists of reported alkali-reactive aggregates have been published both by Coull (1981) and Dolar-Mantuani (1983). Since 1974, extensive international experience with AAR in concrete has been published in a number of journals and practical recommendations. The proceedings from the large series of international conferences on alkali–aggregate reaction also reflect much of the current knowledge and experience with AAR, the last proceedings of which were published from the twelfth and thirteenth conferences organized in Beijing in 2004 (Tang and Deng, 2004) and in Trondheim in 2008 (Broekmans and Wigum, 2008), respectively.

Already during the 1950s, extensive investigations carried out by Idorn (1967) revealed that AAR was a major problem for the durability and long-term performance of concrete structures in Denmark (Figure 4.2). Later on, a large number of countries have recognized AAR to be a serious problem, and eventually, durability problems and maintenance costs of existing concrete structures due to AAR have already made up a large

Figure 4.2 Typical damage due to alkali–aggregate reaction in a retaining concrete wall of a bridge.

proportion of the total maintenance costs of existing concrete structures. Thus, only for Western Europe it was estimated that the damage caused by AAR was making up as much as 10 per cent of the total maintenance costs of all important concrete infrastructures, with annual costs of about €5 billion (European Commission, 1997). Although it is difficult to estimate the costs of reduced service life, this is probably significantly higher than the maintenance costs.

For concrete structures suffering from AAR, the first sign of cracking and deterioration may vary, but generally it takes a long time. Thus, for the faster reacting types of aggregates such as porous flint, it may take only two to five years, while for the more slow reacting types of aggregate such as sandstone, it may take as much as ten to 20 years. The rate of deterioration is primarily determined by type of aggregate and type of environment.

Inspections of concrete structures suffering from AAR are very expensive, since they require use of both expensive equipment and skilled personnel. Testing of aggregate for AAR may also be quite difficult, because considerable experience is needed both for selection of a proper test method for the given type of aggregate and for interpretation of the test results. Current knowledge has clearly shown that existing test methods for alkali reactivity in many cases are not suitable for certain types of aggregate. Thus, many examples are given in the literature demonstrating that two of the most widely adopted test methods such as the mortar bar test ASTM C 227 (ASTM, 2003a) and the chemical method ASTM C 289

(ASTM, 2003b) are not able to detect the slow/late expansive types of aggregate such as sandstone, greywacke and rhyolite. Both a proper selection of a reliable test method and a proper interpretation of the test results may have large economical consequences.

Successively, however, much experience has been gained in order to select proper qualities of aggregate and proper combinations of aggregate and binder system in order to reduce or avoid any problems due to AAR. In order not only to rely on accelerated test methods, many countries have also established special field exposure sites in order to provide more systematic experience with local types of aggregate in relevant types of environment. Such experience not only provides a basis for correlation with accelerated test methods; it also provides a proper basis for specifications and guidelines for production of concrete being resistant to AAR.

5 Codes and practice

5.1 General

From the field performance of concrete structures outlined and discussed in Chapter 2, the question may be raised why all the offshore concrete structures built for the oil and gas industry in the North Sea during the 1970s and 1980s have performed so much better than all the land-based concrete structures built for the public owners along the Norwegian coastline during the same period. For most of the offshore concrete structures the required service life was only 25 to 30 years, while for all the land-based concrete structures the required service life was more than double.

For many years, steel was the traditional structural material for the offshore oil and gas industry. Therefore, when the first concept for offshore installations in the North Sea based on concrete was introduced in the late 1960s, the offshore technical community was very sceptical about the use of concrete as a structural material for such a harsh and hostile marine environment (Gjørv, 1996). At the same time, however, the results and recommendations from the comprehensive field investigation of the more than 200 concrete structures along the Norwegian coastline had just been published (Gjørv, 1968). As described in Chapter 2, most of these concrete structures showed a high structural capacity even after 50 or 60 years of the combined action of severe marine exposure and heavy structural loads. The overall good condition of all these marine concrete structures contributed, therefore, to convincing the offshore technical community that concrete could also be a feasible and reliable structural material for offshore installations in the North Sea.

For the international operators in the North Sea, however, a structural material that showed corrosion problems already after a service period of five to ten years and at the same time was difficult to repair was not acceptable. Therefore, in order to get acceptance for the first offshore concrete platform for the North Sea, both increased concrete quality and increased concrete cover beyond that specified in current concrete codes were required. Since much of the observed durability problems for the land-based concrete structures could also be related to an absence of proper quality control as well as special problems during concrete construction, very strict programmes for quality assurance and quality control during concrete construction were also required before concrete could be accepted as a structural material for offshore installations in the North Sea.

5.2 Durability requirements for offshore concrete structures

In order to meet the new challenge from the more demanding offshore industry, the international organization for prestressed concrete structures, Fédération Internationale de la Précontrainte (FIP) established a Concrete Sea Structures Commission, from which the first edition of *Recommendations for the Design of Concrete Sea Structures* was published in 1973 (FIP, 1973). The durability requirements in these new recommendations were based on both the extensive field experience from all the concrete structures along the Norwegian coastline as well as other current international experience on concrete structures in marine environments. Shortly after, both the Norwegian Petroleum Directorate in their Regulations (NPD, 1976) and Det Norske Veritas (DNV) in their Rules (DNV, 1976) adopted the new and stricter durability requirements for fixed offshore concrete structures given by FIP.

The new durability requirements in the FIP recommendations were slightly different for the various zones of exposure such as the submerged, splash and atmospheric zones, respectively. For the most severe exposure in the splash zone, however, the water/cement ratio should not exceed 0.45 and preferably be 0.40 or less, subject to the attainment of adequate workability. A minimum cement content of 400 kg/m^3 should also be applied, and ordinary reinforcement and prestressing tendons should be protected by a nominal concrete cover of 75 and 100 mm, respectively.

Following the first breakthrough for use of concrete in the development of the Ekofisk Oil Field, rapid development took place as previously described in Chapters 1 and 2. For the first concrete platform, it was not easy to produce a concrete with a water/cement ratio of 0.40 and high compressive strength in combination with 4 per cent to 6 per cent entrained air for attainment of proper frost resistance. For this platform, therefore, a water/cement ratio of 0.45 was used. After the Ekofisk Tank was installed in 1973, however, the requirements for concrete quality successively increased from project to project. Already for the Brent B Platform which was installed in 1975, concrete with a water/cement ratio of 0.38 and a compressive strength of 48.4 MPa in combination with 4.9 per cent air was produced for the splash zone. In addition, for all the other North Sea platforms produced later on, concrete with a water/cement ratio of 0.40 or less was produced, while the compressive strength successively increased by up to 80 MPa for the Troll A Platform which was installed in 1995 (Gjørv, 2008). The regular quality control of water permeability showed a very dense concrete with water penetration depths of typically less than 2 mm according to ISO/DIS 7031 (Standard Norway, 1989), while water permeability values of typically less than 10^{-13} kg/Pa.m.s were observed (Gjørv and Løland, 1980; Gjørv, 1994).

For all the offshore concrete structures in the North Sea, very strict requirements for upper crack widths were also specified. Although the importance of such crack requirements for durability was not very clear (as previously discussed in Chapter 3), the consequences of these requirements were occasionally so severe that they were more decisive for the structural design and the amount of steel installed than that of the wave loads from the so-called '100-year wave'.

In addition to the strict durability requirements as outlined above, most of the concrete platforms built before 1980 were also protected in the splash zone by an additional solid surface coating. This coating was typically a 2 to 3 mm thick epoxy coating, which was continuously applied during slip forming of the structures. During concrete construction, extensive programmes for quality assurance and quality control were also implemented for ensuring the best possible construction quality.

The good performance of all the offshore concrete platforms in the North Sea demonstrates that already from the early 1970s, it was possible both to design and produce quite durable concrete structures even for the most aggressive and harsh marine environment. However, the very strict requirements both to the durability and achieved construction quality were based on proper utilization of current knowledge and experience. For the professional operators in the North Sea, both a high degree of safety and low operation costs of the installations were of the utmost importance.

Although the oldest concrete structures in the North Sea still appear to be in quite good condition, several of these structures have already suffered some extent of steel corrosion, and very costly repairs have also been carried out for some of them (Chapter 2). In spite of both the very homogeneous concrete production and the very strict programmes for quality control during concrete construction, much of the deep chloride penetration and corrosion problems observed may be ascribed to a high variability of the achieved construction quality. It appears, therefore, that such a high variability of the finely placed concrete is an inherent part of concrete as a structural material. In addition, for all these offshore concrete structures, however, the specification for concrete durability was mainly based on prescriptive requirements for concrete composition and execution of concrete work, the results of which are not easy to verify and control during concrete construction. Apart from one of the platforms which was built in The Netherlands with blast furnace slag cement, all the other offshore concrete platforms were also produced with pure portland cements of type CEM I, which is not the best binder system for giving a high resistance against chloride penetration (Chapter 3). Only for a few of the youngest concrete platforms, 2 per cent to 5 per cent silica fume was applied in order to stabilize the highly flowable fresh concrete from segregation during concrete construction.

For the Brent B Platform (1975) which was not protected by any surface coating in the splash zone, a deep chloride penetration and early stage of corrosion were observed after approximately 20 years of exposure (Chapter 2). The concrete in this platform was produced with a water/cement ratio of less than 0.40 and more than 400 kg/m^3 cement, while the specified nominal concrete cover was 75 mm. Thus, for this particular concrete structure, all the durability requirements were fulfilled both according to the current European and Norwegian concrete codes (CEN, 2000; Standard Norway, 2003a, 2003b, 2003c).

5.3 Durability requirements for land-based concrete structures

For all the land-based concrete structures built along the Norwegian coastline from the early 1970s and up to the late 1980s, the main requirement for concrete quality was mostly based on a given compressive strength. During the booming construction activities which took place both in Norway and many other countries during this period, a large number of both concrete harbour structures and concrete bridges were built along the Norwegian coastline. For all of these concrete structures, the requirements to compressive strength and concrete cover typically varied from 30 to 40 MPa and from 25 to 50 mm, respectively. For all of these structures, only pure portland cements of type CEM I were also applied.

For all the land-based concrete structures built during this period, poor workmanship in combination with an absence of proper quality control during concrete construction also typically resulted in poor construction quality. In Chapter 2, this was clearly demonstrated for the Gimsøystraumen Bridge which was built in the northern part of Norway during 1978 to 1981. Extensive repair of this bridge after 11 years of service revealed a very deficient concrete cover, as previously demonstrated in Figure 2.40.

In many countries, a higher performance was often perceived for older concrete structures built before 1970, and this may be related to the rapid development in concrete technology that took place in the early 1970s, the consequences of which were probably not properly understood by the construction industry. Until the late 1960s in Norway, a 30 MPa type of concrete was typically produced with a cement content of more than 400 kg/m^3 (Rudjord, 1967). Therefore, this type of concrete was much more robust from a durability point of view compared to that produced later on. For the old type of concrete, poorer workability properties could sometimes give a poor compaction and hence a durability problem. During the 1970s and 1980s, however, new and more finely ground portland cements were introduced, and new organic and mineral admixtures became available. At the same time, both aggregate production and concrete mixing were optimized. Therefore, it became increasingly possible to meet the specified requirement for compressive strength with successively less cement. In addition, the increased production track often resulted in poorer workmanship and concrete curing. As a result, the durability properties of the new concrete became successively impaired. Typically, it was not until extensive durability problems were experienced in the field that new durability specifications in several steps of revised concrete codes were introduced. In most countries, however, this upgrading of the code requirements for concrete durability lagged far behind technical development and current knowledge.

Although most codes for concrete durability have been upgraded a number of times during the past 30 years, current code specifications for concrete durability are still based almost exclusively on prescriptive requirements for concrete composition and execution of concrete work, the results of which are neither unique nor easy to verify and control during concrete construction. As a result, this descriptive approach has shown to yield insufficient and unsatisfactory results. In recent years, therefore, much research has been carried out in order to develop new procedures and recommendations for durability design and performance-based specifications for concrete durability. As a result, it is now possible to specify performance-based requirements for concrete durability, which also provide a basis for performance-based concrete quality control and documentation of compliance with the specified durability. Therefore, for concrete structures where durability and service life are of special importance, the durability requirements in current concrete codes should only be considered as minimum requirements for the durability of the structures.

For concrete structures in marine environments, the current European Concrete Code EN-206–1 (CEN, 2000) still allows for concrete qualities based on water/binder ratios of up to 0.45. In the national amendment to this code, the current Norwegian Concrete Code NS-EN 206–1 (Standard Norway, 2003a) has reduced this upper level for the water/binder ratio to 0.40. It was not until 1986, however, that the Norwegian concrete codes had any limitation to or requirement for the water/cement ratio for new concrete structures in marine environments (Standard Norway, 1973, 1986). In 1986, however, an upper level for the water/binder ratio of 0.45 was introduced.

After having experienced extensive durability problems with all their concrete coastal bridges (Chapter 2), the Norwegian Public Roads Administration (NPRA) in 1988 introduced their own and stricter durability specifications for new concrete coastal bridges (NPRA, 1988). Thus, from 1988, an upper water/binder ratio of 0.40 was required, while from 1996, this upper level was reduced to 0.38 for the most exposed parts of the concrete bridges (NPRA, 1996). In 1994, a great step forward was made when the Norwegian Public Roads Administration also introduced additional specifications in order to better ensure the minimum concrete cover specified (NPRA, 1994). By specifying a maximum deviation of ±15 mm for the concrete cover to structural steel, an increased concrete thickness of 15 mm to the minimum concrete cover was required.

However, for all the concrete structures built along the Norwegian coastline from the early 1970s, it may be seen from Table 5.1 that it took a long time before the durability requirements introduced by FIP in 1973 for offshore concrete structures were adopted in the current concrete codes for land-based concrete structures in Norwegian marine environments. This slow upgrading of current concrete codes for concrete durability is also typical for most concrete codes in other countries.

Along with the rapid development of concrete technology for all the offshore applications in the North Sea, a rapid international development on high strength concrete also took place (Gjørv, 2008). Since high strength concrete with its low porosity and high density generally enhances the overall performance of the material, the term 'High Performance Concrete' was soon introduced, which is inclusive in the term 'High Strength Concrete'. Internationally, therefore, 'High Performance Concrete' was successively specified for concrete durability rather than for concrete strength. Although a number of definitions of both 'High Strength Concrete' and 'High Performance Concrete' exist, these terms are also mostly specified on the basis of an upper level for water/cement ratio or water/binder ratio, but neither is the term 'water/cement ratio' nor 'water/binder ratio' unique or easy to define any longer.

For many years, when concrete was mostly based on pure portland cements and simple procedures for concrete production, the concept of water/cement ratio was the fundamental basis both for characterizing and

Table 5.1 Development of durability requirements for concrete structures in Norwegian marine environments (splash zone)

Year	*Code*	*Maximum w/b-ratio*	*Minimum concrete cover (mm)*
1973	Fédération Internationale de la Précontrainte, FIP	0.45 (0.40)	75
1976	Norwegian Petroleum Directorate, NPD	0.45 (0.40)	75
1976	Det Norske Veritas, DNV	0.45 (0.40)	75
1973	Norwegian Concrete Code NS 3473	No requirement	25
1986	Norwegian Concrete Code NS 3420	0.45	–
1988	Norwegian Public Roads Administration, NPRA	0.40	–
1989	Norwegian Concrete Code NS 3473	–	50
1996	Norwegian Public Roads Administration, NPRA	0.38	60
2000	European Concrete Code EN-206–1	0.45	–
2003	Norwegian Concrete Code NS-EN-206–1	0.40	–
2003	Norwegian Concrete Code NS 3473	–	60

specifying concrete quality. Since a number of different cementitious materials and reactive fillers are now being applied for concrete production, the concrete properties are increasingly being controlled by the various combinations of such materials. In addition, the concrete properties are also increasingly being controlled by the use of various types of processed concrete aggregate, new concrete admixtures and sophisticated production equipment. As a result, the old and very simple terms 'water/cement ratio' and 'water/binder ratio' for characterizing and specifying concrete quality have successively lost their meaning. As a consequence, there is a great need for performance-based definition and specification for concrete quality. In particular this is true for characterizing and specifying concrete durability.

In the USA, the 'Rapid Chloride Permeability Test' (ASTM, 2005) had already been introduced in the early 1980s by Whiting (1981), and successively, this test method has also been introduced and widely adopted internationally. However, since this test method is only an empirical test method based on the measurement of the total electrical charge passed through the concrete over a short period of time, it does not give any basic information about the resistance of the concrete to chloride penetration. In spite of this, however, it was a great step forward when this test method was introduced both for the specification and control of concrete durability.

In order to stimulate the use of high performance concrete for highway applications in the USA, the Federal Highway Administration already in the early 1990s defined high performance concrete (HPC) by the following four durability and four strength parameters, which included (Goodspeed *et al.* 1996):

Durability
- freeze/thaw durability
- scaling resistance
- abrasion resistance
- chloride permeability.

Strength
- compressive strength
- elasticity
- shrinkage
- creep.

Based on requirements for each of the above parameters, four different performance grades were defined, and details of test methods for determining the performance grades given. Subsequently, applications of the various HPC grades for various exposure conditions were recommended.

For the traditional production of land-based concrete structures, extensive experience has shown that problems related to durability and execution of concrete work have been underestimated for many years. Main emphases have been given to the design for structural capacity, while the design for durability in combination with improved quality control during concrete construction have been neglected. However, the extensive experience in field performance of concrete structures in severe environments as briefly outlined and discussed in Chapter 2 demonstrates that a new approach both to the durability design and to the quality control during concrete construction is greatly needed.

6 Probability of steel corrosion

6.1 General

Depending on the resistance of the concrete against chloride penetration and the thickness of the concrete cover, it may take many years before the chlorides reach the embedded steel. After the chlorides have reached the steel and the corrosion process starts, however, it may take only a few years before visual damage in the form of cracks and rust staining appears, but it may take a long time before the load-carrying capacity of the structure is severely reduced. Schematically, this deteriorating process takes place as shown in Figure 6.1. As soon as the corrosion process starts, a very complex system of galvanic cell activities develops in the concrete structure, as previously discussed in Chapter 3. In this system of galvanic cell activities, the deterioration appears in the form of concentrated pitting corrosion in the anodic areas of the rebar system, while the adjacent

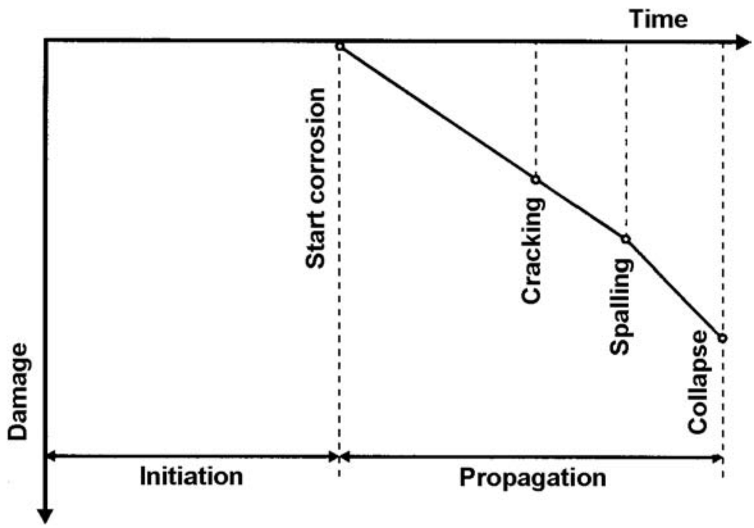


Figure 6.1 Deterioration of a concrete structure due to steel corrosion (source: Tuutti (1982)).

cathodic areas act as catchment areas for the oxygen. Although larger portions of the rebar system eventually become depassivated, all of these areas will not necessarily corrode. As also discussed in Chapter 3, the steel in the first and most active corroding parts of the structure will act as sacrificial anodes and thus cathodically protect other parts of the structure. Since both the structural shape and the local environmental exposure will affect this pattern of deterioration, it appears to be very difficult to develop a general mathematical model for predicting the time necessary before the load-carrying capacity of the structure as a whole becomes severely reduced. Although several attempts at developing such a mathematical model have been made (Lu *et al.*, 2008), it appears that no reliable mathematical model for this very complex deteriorating process currently exists. Already in the early 1970s, however, Collepardi *et al.* (1970, 1972) came up with a relatively simple mathematical model for estimating the time necessary for the chlorides to reach the embedded steel through concrete of a given quality and thickness.

Although it is possible to estimate the time necessary before the corrosion starts, this does not provide any basis for estimating the real service life of the structure. As soon as the corrosion process starts, however, the owner of the structure has got a problem, which at an early stage represents only a maintenance and economic problem, but later on develops into a more difficult controllable safety problem. As a basis for the durability design, therefore, efforts should be made in order to obtain the best possible control of the chloride penetration during the initiation period before any corrosion starts. It is in this early stage of the deteriorating process that it is both technically easier and much cheaper to take necessary precautions and select proper protective measures for control of the further deteriorating process compared to that later on.

Since all the input parameters needed for calculating the rate of chloride penetration through the concrete cover always show a high scatter and variability, it is very appropriate to combine this calculation with a probability analysis which can take some of this scatter and variability into account (DuraCrete, 2000). In this way, it is possible to estimate the probability for the chlorides to reach the embedded steel during a certain service period for the given structure.

For such a probability-based durability design, a serviceability limit state (SLS) must also be defined. Although various stages of the deteriorating process may be chosen as the basis for such a serviceability limit state, the onset of steel corrosion is a very critical stage which has been chosen as the serviceability limit state for the following durability design.

In recent years, a rapid development of models and procedures for probability-based durability design of concrete structures has taken place (Siemes and Rostam, 1996; Engelund and Sørensen, 1998; Gehlen, 2000; DuraCrete, 2000; fib, 2006), and in many countries such durability design has been applied to a number of important concrete structures (Stewart and Rosowsky, 1998; McGee, 1999; Gehlen and Schiessl, 1999; Gehlen, 2007). Also, in Norway such durability design has been applied to a number of concrete structures, where safety, durability and service life have been of special importance (Gjørv, 2002, 2004). In the beginning, this design was primarily based on the results and guidelines from the European research project DuraCrete (2000), but successively, as practical experience with such design was gained, the basis for the design was simplified and further developed for more practical applications. Thus in 2004, this design was adopted by the Norwegian Association for Harbour Engineers (NAHE) as general recommendations and guidelines for durability design of new concrete structures in Norwegian harbours, new and revised editions of which have already been issued (NAHE, 2007a, 2007b).

In the following, a short outline of the above durability design is given and discussed. First, the theoretical basis for calculation of both chloride penetration and probability of corrosion is given. Second, the necessary input parameters for the durability analysis are described and discussed. Finally, some durability analyses are carried out in order to demonstrate how calculations of corrosion probability can be applied as a basis for the durability design. Later on (Chapter 11), current experience from practical applications to some recent commercial projects is also briefly presented.

6.2 Calculation of chloride penetration

As previously discussed in Chapter 3, rather complex transport mechanisms for penetration of chlorides into concrete exist. In a very simplified form, however, it is possible to estimate the rates of chloride penetration by use of Ficks 2. Law of Diffusion according to Collepardi *et al.* (1970, 1972) in combination with a time-dependent chloride diffusion coefficient according to Takewaka and Mastumoto (1988) and Tang and Gulikers (2007) as shown in equations 6.1 and 6.2:

$$C(x,t) = C_S\left[1 - erf\left(\frac{x}{2\sqrt{D(t)\cdot t}}\right)\right] \tag{6.1}$$

In this equation, $C(x,t)$ is the chloride concentration in depth x after time t, C_S is the chloride concentration at the concrete surface, D is the concrete chloride diffusion coefficient and *erf* is a mathematical function.

$$D(t) = \frac{D_0}{1-\alpha}\left[\left(1+\frac{t'}{t}\right)^{1-\alpha} - \left(\frac{t'}{t}\right)^{1-\alpha}\right]\left(\frac{t_0}{t}\right)^{\alpha}\cdot k_e \tag{6.2}$$

In this equation, D_0 is the diffusion coefficient after the reference time t_0, and t' is the age of the concrete at the time of chloride exposure. The parameter α represents the time dependence of the diffusion coefficient, while

k_e is a parameter which takes the effect of temperature into account (Polder and deRooij, 2005):

$$k_e = \exp\left[b_e\left(\frac{1}{293} - \frac{1}{T-273}\right)\right] \tag{6.3}$$

where *exp* is the exponential function, b_e is a regression parameter, and T is the temperature.

The criterion for steel corrosion then becomes:

$$C(x) = C_{CR} \tag{6.4}$$

where $C(x)$ is the chloride concentration at the depth of the embedded steel, and C_{CR} is the critical chloride concentration in the concrete necessary for onset of corrosion.

6.3 Calculation of probability

For the structural design of concrete structures, the main objective is always to establish the combined effects of loads (S) and the resistance to withstand the loads (R) in such a way that the design criterion becomes:

$$R \geq S \quad \text{or} \quad R - S \geq 0 \tag{6.5}$$

When $R < S$, failure will occur, and since all the factors affecting R and S also show a high scatter and variability, all established design procedures have properly taken this into account.

In principle, the durability design takes the same approach as that of the structural design. In this case, the effect of both loads (S), which is the combined effect of both chloride loads and temperature conditions and the resistance to withstand the loads (R), which is the resistance to chloride penetration, must be established. Although neither S nor R is comparable to that of the structural design, the acceptance criterion for having the probability for 'failure' less than a given value is the same.

In Figure 6.2, the scatter and variability of both R and S are demonstrated in the form of two distribution curves along the y-axis. At an early stage, there is no overlapping between these two distribution curves, but over time, a gradual overlapping from time t_1 to t_2 takes place. This increasing overlapping will at any time reflect the probability of 'failure' or the probability for corrosion to occur, and gradually, the upper acceptable level for the probability of 'failure' (t_{SLS}) is reached and exceeded.

In principle, the probability of failure may be written as:

$$P(failure) = P_f = P(R - S < 0) < P_0 \tag{6.6}$$

where P_0 is a measure for failure probability.

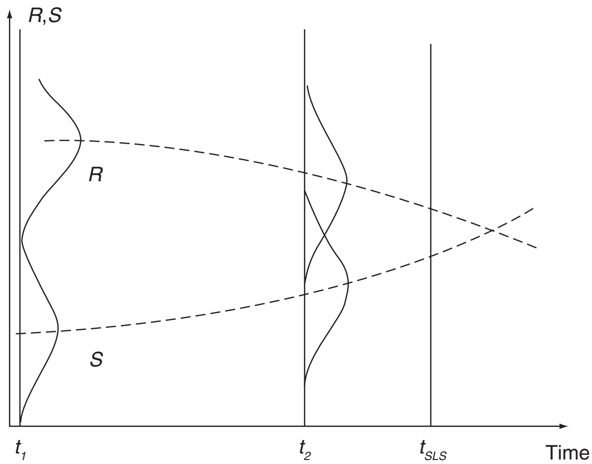


Figure 6.2 The principles of a time-dependent reliability analysis.

In current codes for reliability of structures, an upper level for probability of failure of 10 per cent in the serviceability limit state is often specified (Standard Norway, 2004). Therefore, an upper probability level of 10 per cent for onset of corrosion has also been adopted as a basis for the durability design as described in the following.

Normally, the failure function includes a number of variables, all of which have their own statistical parameters. Therefore, the use of such a failure function requires numerical calculations and the application of special software. Currently, there are several mathematical methods available for evaluation of the failure function, such as:

- FORM (First Order Reliability Method)
- SORM (Second Order Reliability Method)
- MCS (Monte Carlo Simulation).

6.4 Calculation of corrosion probability

In principle, the calculation of corrosion probability may be carried out by the use of any of the above mathematical methods in combination with the appropriate software. Based on current experience with durability design of concrete structures in chloride-containing environments, however, calculation of chloride penetration by use of equation 6.1 in combination with a Monte Carlo Simulation has proved to give a very simple and appropriate basis for calculating the probability of corrosion (Gjørv, 2004). Although such a combined calculation may also be carried out in different ways, a special software DURACON (Figure 6.3) for this calculation has been developed (Ferreira *et al.*, 2004a).

Figure 6.3 Opening window for the software DURACON (2004).

Primarily, the above calculation of corrosion probability provides the basis for the durability design of new concrete structures as outlined and discussed in the following. As a result, it is possible to specify a certain service period before a probability of 10 per cent for corrosion is reached. For the given environmental exposure, requirements for both concrete quality (chloride diffusivity) and concrete cover can then be specified. For the later operation of the concrete structure, however, the above calculation also provides the basis for condition assessment and preventive maintenance as outlined and discussed in Chapter 10. Based on the observed rates of the real chloride penetration taking place during operation of the structure, calculations of the future probability of corrosion then provide a basis for the preventive maintenance of the structure.

For both types of probability calculation, certain input parameters to the analyses are needed. For the durability design of new concrete structures, the necessary input parameters are described and discussed in the following. For the calculation of future corrosion probability during operation of the structure, the necessary input parameters are described and discussed later on (Chapter 10).

6.5 Input parameters

General

In general, the durability design should always be an integral part of the structural design for the given concrete structure. At an early stage of the design, therefore, the overall durability requirement of the structure should be based on the specification of a certain service period before 10 per cent probability of corrosion is reached. For the given environmental exposure, the durability analysis then provides the basis for specification of a proper combination of concrete quality and concrete cover. Before the final requirements for concrete quality and concrete cover are given, however, it may be necessary to carry out several calculations for various combinations of possible concrete qualities and concrete covers. For all of these calculations, proper information about the following input parameters is needed:

- *Environmental loading*
  - chloride load, C_S
  - temperature, T

- *Concrete quality*
  - chloride diffusivity, D_0
  - time-dependence factor, α
  - critical chloride content, C_{CR}

- *Concrete cover*, X

All the above parameters may have different distribution characteristics. If nothing else is known, however, a statistical normal distribution may be assumed. For each parameter, proper information on both average value and standard deviation is then needed.

In the following, some general guidelines for determination and selection of the above input parameters are given.

Environmental loading

Chloride load, C_S

For a concrete structure in a chloride-containing environment, the chloride load on the concrete surface (C_S) is normally defined as shown in Figure 6.4, which is the result of a regression analysis of observed data on chloride penetration and curve fitting to Ficks 2. Law. This surface chloride concentration (C_S) is normally higher than the maximum observed chloride concentration in the surface layer of the concrete structure (C_{max}). The surface chloride concentration is primarily the result of the local environmental exposure, but both concrete quality and geometrical shape of the structure also affect the accumulation of this surface chloride concentration. For all concrete structures, therefore, the accumulated surface chloride concentration shows a very high scatter and variability, as previously demonstrated in Chapter 2. For the durability analysis, however, it is important to estimate and select a proper value of the surface chloride concentration (C_S) which is as representative as possible for the most exposed and critical parts of the structure. In some cases, it may also be appropri-

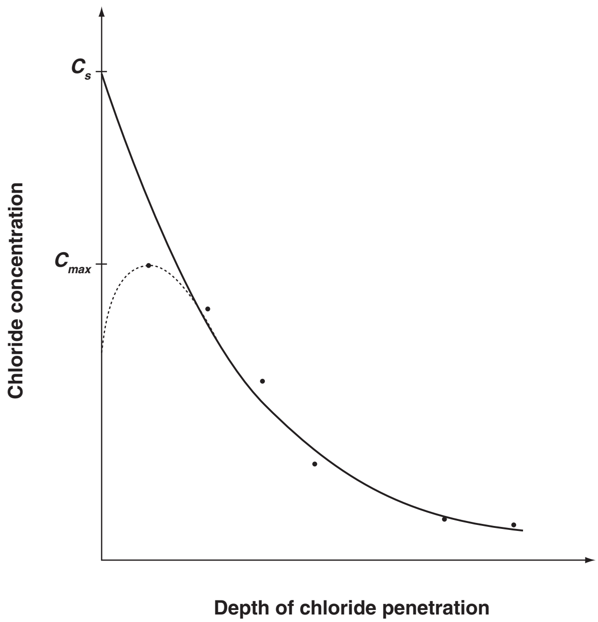


Figure 6.4 Definition of the surface chloride concentration (C_S) based on a regression analysis of observed data on chloride penetration.

ate to select different chloride loads for different parts of the structure and then carry out separate probability calculations for the various parts of the structure.

For a new concrete structure, it may not be easy to estimate and select a proper value for the chloride load as generally described above. If possible, therefore, data from previous field investigations of similar types of concrete structures in similar types of environment should be applied. In many countries, a large number of both concrete bridges and concrete harbour structures in severe chloride exposure environments have been the subject of extensive field investigations. In Norwegian marine environments, extensive measurements of chloride penetration have been carried out on a large number of concrete structures as previously described in Chapter 2, some data from which have been plotted in Figures 6.5 and 6.6. For the individual concrete structures, experience has shown that the surface chloride concentrations (C_S) successively accumulate over a certain number of years before they tend to level out to fairly stable values for the given environmental exposure.

Although the selection of chloride loads for new concrete structures should preferably be based on local experience from similar concrete structures exposed to similar environments, some general experience available from the literature may also provide a basis for the selection of a proper

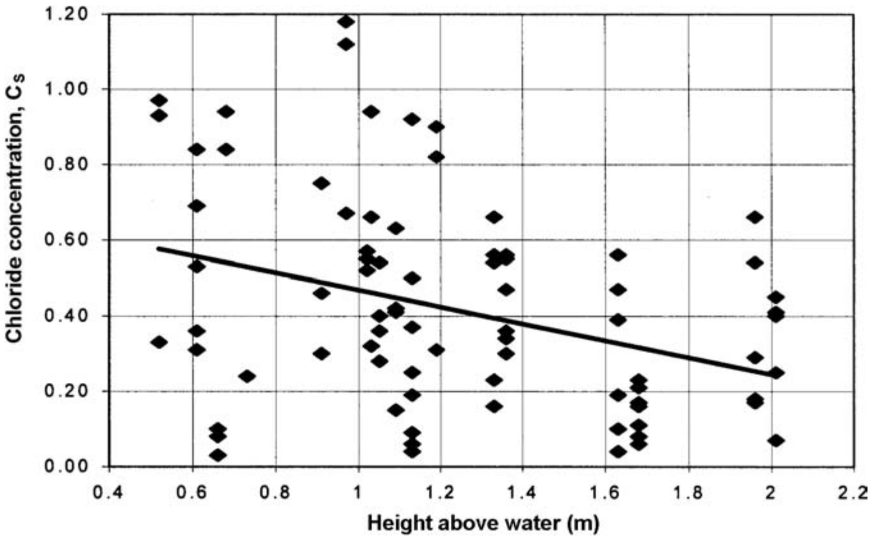


Figure 6.5 Obtained surface chloride concentrations by weight of concrete (C_s) on Norwegian concrete harbour structures (source: Markeset (2004), based on data from Hofsøy *et al.* (1999)).

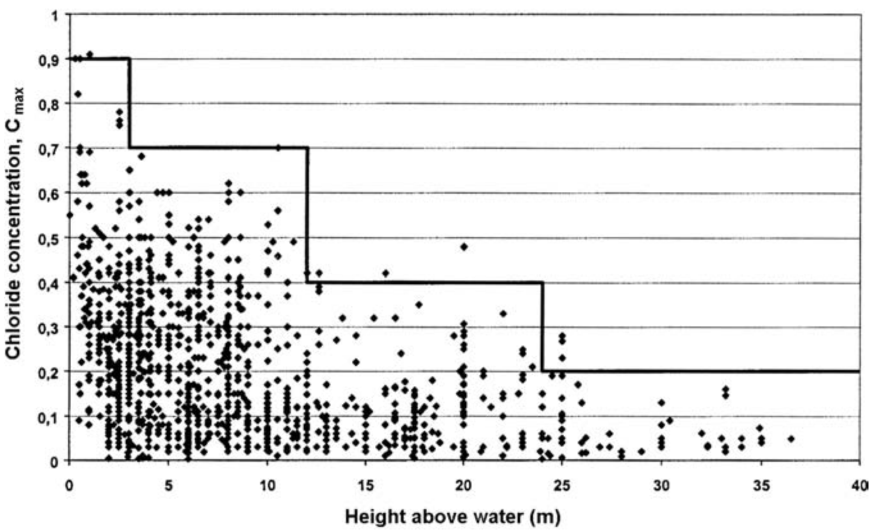


Figure 6.6 Observed maximum surface chloride concentrations by weight of concrete (C_{max}) on Norwegian concrete coastal bridges (source: Fluge (2001)).

chloride load. Primarily based on the extensive field investigations of concrete structures along the Norwegian coastline, some general guidelines for estimation of chloride loads on concrete structures in severe marine environments are given in Table 6.1. For concrete harbour structures with an open concrete deck very close to the sea level, the chloride load may be characterized as 'High', with an average value of 5.5 per cent and a standard deviation of 1.3 per cent by weight of cement, respectively. Since the chloride concentrations are often given in per cent by weight of concrete, a general conversion diagram such as that shown in Figure 6.7 may be used.

Although the chloride loads on concrete bridges in marine environments very much depend on the height above water, even at the same height, the surface chloride concentrations may accumulate very differently from one part of the structure to another. Thus, for the Norwegian concrete coastal bridges shown in Chapter 2, lower surface chloride concentrations were typically observed on those parts of the structure which were the most

Table 6.1 Some general guidelines for estimation of chloride loads (C_S) on concrete structures in severe marine environments

Chloride load	C_S *(% by wt. of cement)*	
	Mean value	*Standard deviation*
High	5.5	1.3
Average	3.5	0.8
Moderate	1.5	0.5

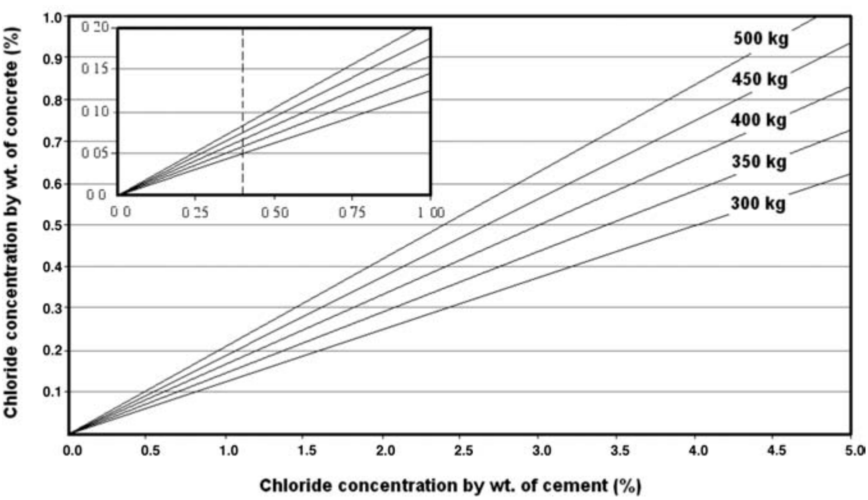


Figure 6.7 Conversion diagram for estimating chloride concentrations in per cent by weight of cement based on per cent by weight of concrete with various cement contents (source: Ferreira (2004)).

exposed to the prevailing winds and salt spray compared to that of the more protected parts (Figures 2.37 and 2.38). In the most exposed parts the rain was intermittently washing off the salt again, while the salt accumulated on the more protected parts.

Temperature, T

For a concrete structure in a given chloride-containing environment, the rate of chloride penetration also very much depends on the temperature, as shown in equation 6.3. Based on local information on the current temperature conditions, data on average annual temperatures may be used as a basis for the selection of this input parameter.

Concrete quality

Chloride diffusivity, D_0

As previously discussed in Chapter 3, the chloride diffusivity (D_0) of the given concrete is a very important concrete property which generally reflects the resistance of the concrete to chloride penetration. Although a low water/binder ratio generally gives a low porosity and hence a high resistance against chloride penetration, it was previously shown and discussed in Chapter 3 how the selection of a proper binder system may be more important for obtaining a high resistance to chloride penetration than selecting a low water/binder ratio. Thus, a reduction of the water/binder ratio from 0.45 to 0.35 for a concrete based on a pure portland cement may only reduce the chloride diffusivity by a factor of approximately 2, while replacing the portland cement with a blast furnace slag cement may reduce the diffusivity by a factor of approximately 50 (Bijen, 1998). If the slag cement is also combined with a pozzolanic material such as silica fume, an extremely low chloride diffusivity, and hence an extremely high resistance to chloride penetration may be obtained, as was shown in Chapter 3.

In the literature, there are several types and definitions of the chloride diffusivity of a given concrete as well as several methods for testing the chloride diffusivity (Schiessl and Lay, 2005). Thus, NORDTEST has standardized three different types of test method, including the steady-state migration method NT Build 355 (NORDTEST, 1989), the immersion test method NT Build 443 (NORDTEST, 1995) and the non-steady-state migration method NT Build 492 (NORDTEST, 1999), respectively. All of these test methods give different values for the chloride diffusivity, but since they all show a strong correlation, any of the established test methods may be used both for quantifying and comparing the resistance against chloride penetration of various types of concrete.

Although all of the above types of test method are accelerated test methods, the duration of the testing is very different. For the non-steady-state migration method, there is no requirement to pre-curing of the concrete and the testing mostly takes 24 hours, while both the steady-state migration method and the immersion test method are based on well-cured concrete specimens, and the testing may take a very long time. Therefore, in order to also be used as a simple and rapid test method for quality control during concrete construction, extensive experience has shown that the non-steady-state migration method or the so-called Rapid Chloride Migration (RCM) method is a very appropriate test method. In particular this is true if this test method is combined with a corresponding testing of the electrical resistivity of the concrete as described and discussed later on for concrete quality control (Chapter 8).

The RCM method which was originally developed by Tang in 1996 (Tang, 1996a, 1996b) was later on the subject of extensive compliance testing with other test methods in the European research project DuraCrete (2000). As a result, a chloride diffusivity based on the RCM method was specified as a basis for the general guidelines for durability design developed in this research project. A very good correlation with the results obtained by the steady-state migration method was also observed by Tong and Gjørv (2001). The strong statistical correlation with the chloride diffusion coefficients obtained by the immersion test may also be seen in Figure 6.8.

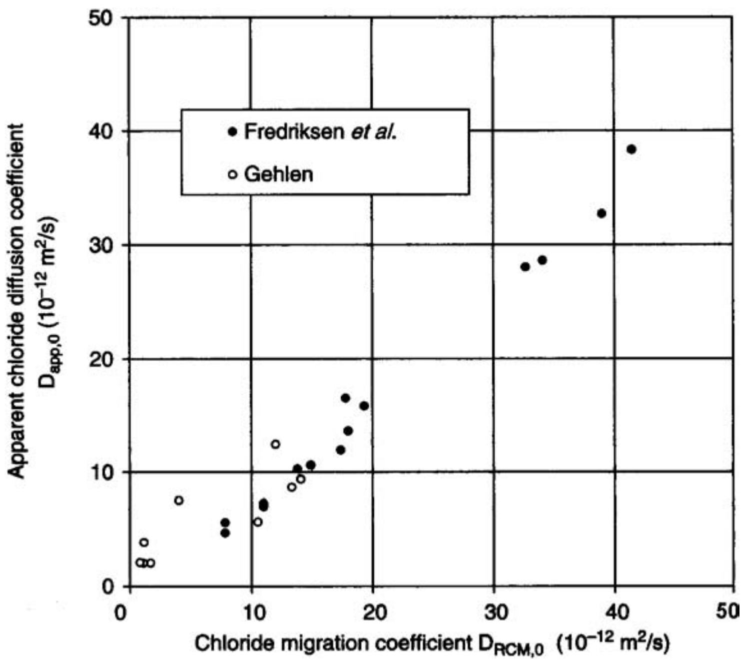


Figure 6.8 Correlation of the effective diffusion coefficients obtained by immersion tests and the RCM method according to Gehlen (2000) and Fredriksen *et al.* (1996) (source: Schiessl and Lay (2005)).

In 2001, a very good documentation of the precision of the RCM method was published by Tang and Sørensen (2001). Based on the European research project ChlorTest (2005), the RCM method also revealed the best precision among all the test methods evaluated (Figure 6.9). Due to its simplicity, rapidity and precision, the RCM method has successively been finding a more international application (Hooton *et al*., 2000; AASHTO, 2003), and the test method is currently also under consideration for further standardization by the European Committee for Standardisation (CEN). In the following, therefore, the chloride diffusivity of the concrete (D_0) is both defined and based on the RCM method, some further details of which are given in Chapter 8.

When the chloride diffusivity is used as an input parameter to the probability analyses for durability design, this parameter is normally based on the value obtained after 28 days of standardized curing conditions (D_{28}). Occasionally, some additional probability analyses may also be carried out on the basis of chloride diffusivities obtained after longer periods of curing. This may be the case if some of the possible concrete qualities are based on binder systems which hydrate more slowly such as those based on fly ash. For regular concrete quality control during concrete construction and documentation of compliance with specified durability, however, the testing is normally based on the 28-day chloride diffusivity (D_{28}) as described in Chapters 8 and 9.

For continued water curing in the laboratory beyond 28 days, the chloride diffusivity is successively reduced over a certain period of time, but, somewhat depending on the type of binder system, it typically tends to level out within a curing period of approximately one year (Chapter 3). Therefore, the obtained chloride diffusivity after one year (D_{365}) is used as a basis for reflecting the potential resistance of the given

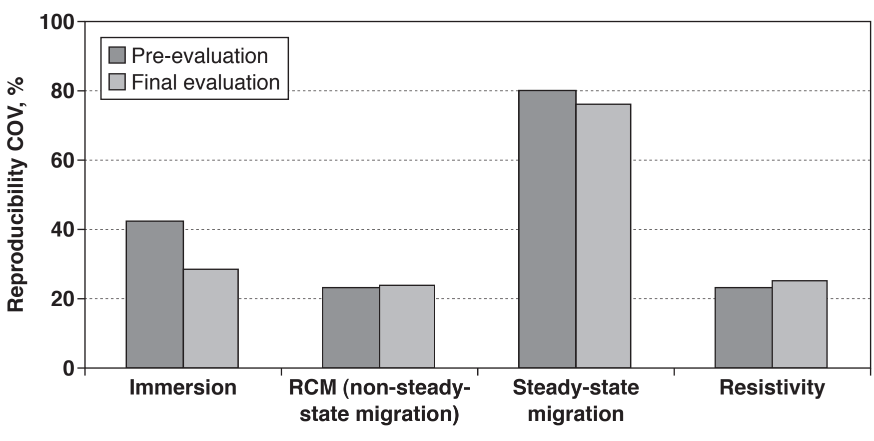


Figure 6.9 Precision among all the test methods for chloride diffusivity evaluated in the European research project ChlorTest (2005).

concrete against chloride penetration as described and discussed in Chapter 9.

As a basis for a general assessment of the resistance to chloride penetration of various types of concrete based on the 28-day chloride diffusivity, some general guidelines are shown in Table 6.2.

Time-dependence factor, α

Since the chloride diffusivity is a time-dependent property of the concrete, this time dependence (α) is also a very important parameter, generally reflecting how the chloride diffusivity of a given concrete in a given environment develops over time. Although the α-value may also be determined by use of laboratory testing, this would be both time consuming and at the same time not properly reflect how the chloride diffusivity of the given concrete in the given environment would develop. In order to reflect more realistic field conditions, therefore, empirical α-values for the given type of concrete in the given type of environment are normally used as input parameters to the durability analyses.

For new concrete structures, therefore, the same problem in selecting a proper α-value exists as that already discussed for the selection of a proper chloride load (C_S). Again, current experience from field investigations of similar concrete structures in similar environments may provide a basis for selecting proper α-values. In addition, information from long-term field tests with similar types of concrete in similar environments may be available from the literature. Based on such information, some general guidelines for selecting a proper α-value are given in Table 6.3. This table shows some observed α-values for various types of concrete based on various binder systems exposed in the tidal and splash zone of marine environments (Mangat and Molloy, 1994; Bamforth, 1999; Thomas and Bamforth, 1999; Thomas *et al.*, 1999; DuraCrete, 2000; fib, 2006). Although combinations of the various types of cement with pozzolanic materials such as silica fume or fly ash will always reduce the chloride diffusivity, current experience indicates that Table 6.3 may still be used as a general basis for the estimation of a proper α-value.

Table 6.2 Resistance to chloride penetration of various types of concrete based on the 28-day chloride diffusivity (Nilsson *et al.*, 1998)

Chloride diffusivity, $D_{28} \times 10^{-12}\, m^2/s$	*Resistance to chloride penetration*
>15	Low
10–15	Moderate
5–10	High
2.5–5	Very high
<2.5	Extremely high

Table 6.3 Some general guidelines for estimation of α-values for tidal and splash zone exposure of concrete structures in marine environments

Concrete based on various types of cement	*α-value*	
	Mean value	*Standard deviation*
Portland cements	0.40	0.08
Blast furnace slag cements	0.50	0.10
Fly ash cements	0.60	0.12

Critical chloride content, C_{CR}

As previously discussed in Chapter 3, a number of factors affect the depassivation of embedded steel in concrete. Depending on all these factors, the critical chloride concentration in the pore solution of a concrete for breaking the passivity may vary within wide limits. In addition, due to the very complex relationship between the total chloride content in the concrete and the passivity of embedded steel, it is not possible to give any general values for critical chloride content. When certain values for the critical chloride content are nevertheless given in existing concrete codes and recommendations, this is only based on empirical information on chloride content which may give a certain risk for the development of corrosion (Table 6.4). However, whether an unacceptable development of corrosion will take place or not also depends on other corrosion parameters such as electrical resistivity of the concrete and oxygen availability as discussed in Chapter 3. Generally, very small chloride concentrations in the pore solution of a concrete are able to destroy the passivity of the steel, but the risk of the development of corrosion may be very low both in very dry concrete due to ohmic control of the corrosion process and in very wet or submerged concrete due to very low availability of oxygen (Figure 6.10).

Based on empirical experience from a wide range of concrete qualities and moisture conditions as shown Figure 6.10, an average value of 0.4 per

Table 6.4 Risk for development of corrosion depending on chloride content

Chloride content (%)		*Risk of corrosion*
By wt. of cement	*By wt. of concrete*[1]	
>2.0	>0.36	Certain
1.0–2.0	0.18–0.36	Probable
0.4–1.0	0.07–0.18	Possible
<0.4	<0.07	Negligible

Source: Brown *et al.* (1980).

Note

1 Based on 440 kg/m^3 of cement.

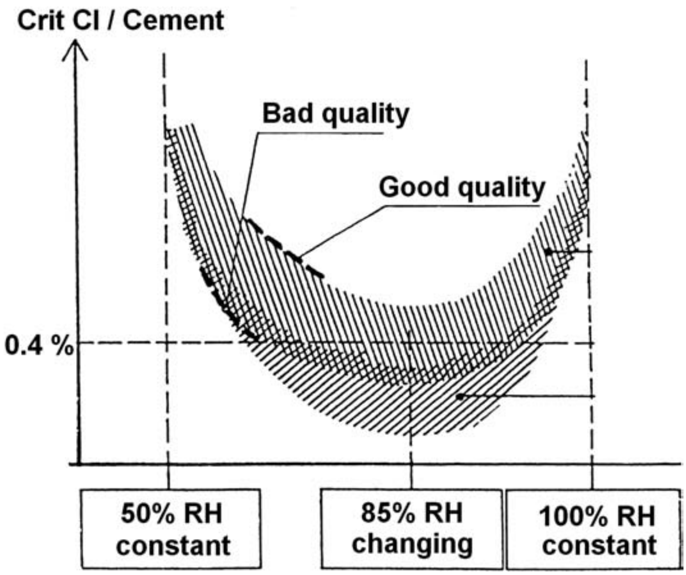


Figure 6.10 Qualitative relationship of critical chloride content (C_{CR}), environmental conditions and quality of concrete (courtesy of International Federation for Structural Concrete, fib) (source: CEB (1992)).

cent by weight of cement is often referred to in codes and specifications. Therefore, if nothing else is known, an average value of 0.4 per cent with a standard deviation of 0.1 per cent by weight of cement may be selected as input parameter to the durability analysis. For more corrosion-sensitive types of steel, an average value of 0.1 per cent with a standard deviation of 0.03 per cent may be selected. For various types of stainless steel, however, critical chloride contents of up to 3.5 per cent or even up to 5.0 per cent by weight of cement may be applied, as discussed later on (Chapter 7).

Concrete cover, X

In current concrete codes, requirements for both minimum concrete cover (X_{min}) and tolerances are provided for the given environment. Thus, the nominal concrete cover (X_N) is always given with a certain value of tolerance (ΔX), and different values for ΔX may be specified. For a tolerance of ±10 mm, the minimum requirement to concrete cover then becomes:

$$X_{min} = X_N - 10 \tag{6.7}$$

Although the specified concrete cover primarily gives the required concrete cover to the structural steel, additional mounting steel for ensuring the position of the structural steel during concrete construction is also often applied. Since the penetrating chlorides do not distinguish between structural and mounting steel, the nominal concrete cover should preferably be specified for all embedded steel including the mounting steel in order to avoid any cracking of the concrete cover due to premature corrosion. During structural design, efforts are always made in order to avoid any cracking of the concrete. Cracking of the concrete cover caused by corroding mounting steel may represent the same type of weakness as that caused by any other types of cracking. Therefore, instead of using mounting steel which can corrode, mounting systems based on non-corroding materials such as those discussed in Chapter 7 should be applied.

If it is assumed that 5 per cent of the reinforcing steel has a concrete cover less than X_{min}, the durability analysis may be based on an average concrete cover of X_N with a standard deviation of $\Delta X/1.645$. Then, the effect of increased concrete cover beyond that required in current concrete codes may be quantified. For the documentation of achieved construction quality as described later on (Chapter 9), however, the durability analyses must always be based on the achieved values both for concrete cover and standard deviation observed from the quality control during concrete construction.

6.6 Durability analysis

General

In order to demonstrate how the above calculation of corrosion probability may be applied as a basis for the durability design of new concrete structures, some results from the durability design of a new concrete harbour structure are shown in the following. As an overall durability requirement to the given structure, a service period of at least 120 years was required before a 10 per cent probability of corrosion would be reached. In addition, the minimum durability requirements according to the current concrete codes should also be fulfilled. As a basis for selecting a proper combination of concrete quality and concrete cover, a number of durability analyses were carried out. Some of these analyses were based on different types of concrete with various chloride diffusivities, while others were carried out in order to evaluate the effect of increased concrete cover beyond the minimum concrete cover required in the current concrete code.

Effect of chloride diffusivity

In order to select a concrete with a proper chloride diffusivity, the 28-day chloride diffusivity of the eight types of concrete previously shown in Figures 3.2 and 3.3 (Chapter 3) were selected as input parameters to the durability analyses. These concrete mixtures were all produced with four different types of commercially available cements with and without the addition of silica fume. The cements included one high performance portland cement of type CEM I 52,5 LA (Type 1), one blended portland cement with approximately 18 per cent fly ash of type CEM II/A V 42,5 R (Type 2), and two types of blast furnace slag cements, one with approximately 34 per cent slag of type CEM II/B-S 42,5 R NA (Type 3) and one with approximately 70 per cent slag of type CEM III/B 42,5 LH HS (Type 4). For the first four concrete mixtures (Types 1–4), 390 kg/m^3 of cement and 39 kg/m^3 of silica fume (10 per cent) were used giving a water/binder ratio of 0.38. For comparison, the other four mixtures (Types 5–8), 420 kg/m^3 of cement giving a water/binder ratio of 0.45 was also used.

Based on the 28-day chloride diffusivities (D_{28}) and estimated data both for the time-dependence of the chloride diffusivities (α) and the critical chloride content (C_{CR}) as shown in Table 6.5, durability analyses were carried out. For all of these analyses, the input parameters for concrete cover and environmental loading were kept constant. In order to meet the requirements for minimum concrete cover and tolerance given in the current Norwegian Concrete Codes NS 3473 and NS 3465 (Standard Norway, 2003a, 2003b), an average value for concrete cover of 70 mm

Table 6.5 Input parameters for analysing the effect of chloride diffusivity

Concrete quality	*Input parameter*		
	D_{28} ($\times 10^{-12}$ m^2/s)	*α*	*C_{CR} (% by weight of binder)*
Type 1 (CEM I 52,5 LA + 10% CSF)	N[1](6.0;0.64)	N(0.40;0.08)	
Type 2 (CEM II/A – V 42,5 R + 10% CSF)	N(7.0;1.09)	N(0.60;0.12)	
Type 3 (CEM II/B – S 42,5 R NA + 10% CSF)	N(1.9;0.08)	N(0.50;0.10)	
Type 4 (CEM III/B 42,5 LH HS + 10% CSF)	N(1.8;0.15)		N(0.40;0.08)
Type 5 (CEM I 52,5 LA)	N(13.3;0.83)	N(0.40;0.08)	
Type 6 (CEM II/A–V 42,5 R)	N(12.8;1.03)	N(0.60;0.12)	
Type 7 (CEM II/B–S 42,5 R NA)	N(7.2;1.16)	N(0.50;0.10)	
Type 8 (CEM III/B 42,5 LH HS)	N(6.7;0.0.75)		

Note
1 Normal distribution.

with assumed normal distribution and standard deviation of 6 mm was selected. For the chloride load (C_S), an average value of 5.5 per cent with assumed normal distribution and standard deviation of 1.3 per cent by weight of binder were adopted (Table 6.1). An annual average temperature (T) of +10°C was also chosen.

For the first four concrete mixtures (Types 1–4), the durability requirements according to the current Norwegian Concrete Code NS-EN 206–1 (Standard Norway, 2003c) both with respect to water/binder ratio (≤0.40) and binder content (≥360 kg/m^3) were fulfilled. In spite of this compliance, however, the results in Figure 6.11 clearly demonstrate how different the obtained probabilities for corrosion for the various types of concrete would be. Thus, for a combination of the pure portland cement with silica fume (Type 1), a service period of only about 25 years would be obtained before the level of 10 per cent probability for corrosion would be reached, while the use of fly ash cement (Type 2) would increase this period to about 40 years. Both types of blast furnace slag cement (Types 3 and 4) would give a service period of more than 120 years.

For the other four concrete mixtures (Types 5–8) which all had a water/binder ratio of 0.45 and binder content of 420 kg/m^3, the durability requirements according to the current European Concrete Code EN 206–1 (CEN, 2000) were also fulfilled. In this case, it may be seen from Figure 6.12 that the pure portland cement (Type 5) would only give a service period of about ten years, while the fly ash cement (Type 6) would increase

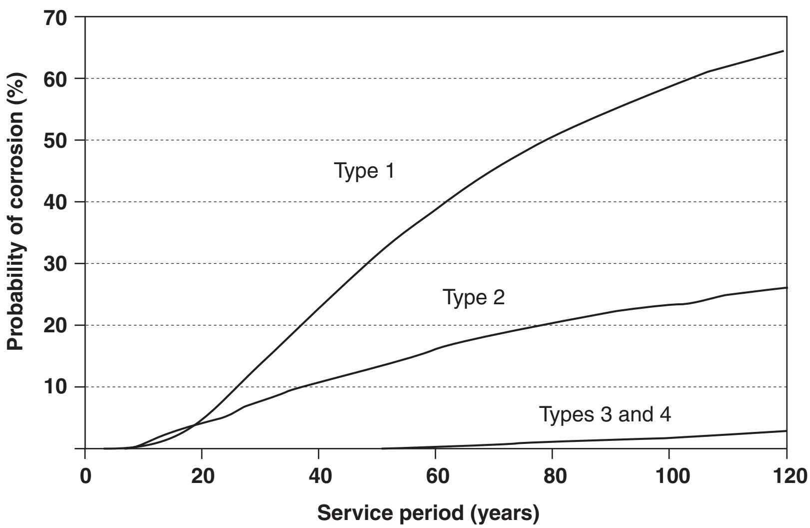


Figure 6.11 Effect of binder system on the probability of corrosion in a relatively dense concrete.

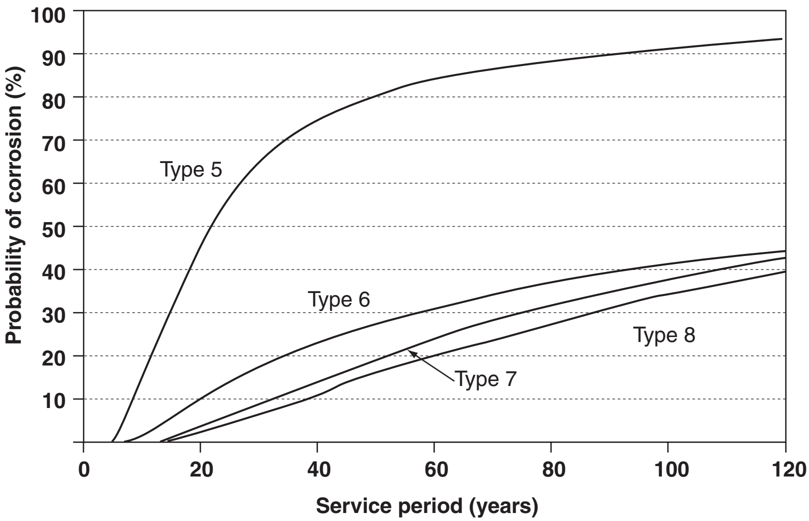


Figure 6.12 Effect of binder system on the probability of corrosion in a relatively porous concrete.

this period to about 20 years before the 10 per cent probability of corrosion would be reached. Both types of blast furnace slag cement (Types 7 and 8) would give a service period of 30 to 35 years.

Since the overall durability requirement of the given structure was a service period of at least 120 years before the probability of 10 per cent for corrosion would be reached, it would only be the concretes of Types 3 and 4 that would meet this requirement. It should be noted, however, that the above durability analyses were only carried out on the basis of the 28-day chloride diffusivities. Although some additional durability analyses based on more mature values for the chloride diffusivity as shown in Figures 3.2 and 3.3 were also carried out, this did not significantly affect the basis for comparison of the various types of concrete. In addition, the above results are in general agreement with current practical experience on the long-term performance of concrete structures in marine environments. As previously discussed in Chapter 3, both blast furnace slag cements and fly ash cements will always give a higher resistance to chloride penetration than that of pure portland cements, and slag cements will always give a higher resistance than that of fly ash cements. While blast furnace slag cements will also give a high early age resistance to chloride penetration, even at very low temperatures, fly ash cements will always give a very low early age resistance, and the lower the temperature, the lower the resistance (Chapter 3). Therefore, concretes based on fly ash cements will generally be more vulnerable to early age exposure during concrete construction in severe marine environments compared to that of other types of cement.

Effect of concrete cover

In order to evaluate the effect of increased nominal concrete cover beyond the minimum requirement of 70 mm used in the above analyses, a new series of durability analyses based on increased concrete covers of 90 and 120 mm, respectively, were carried out. For these analyses, the above input parameters for the analysis of Type 1 concrete were selected (Table 6.6).

As clearly demonstrated in Figure 6.13, increased concrete cover also has a significant effect on the obtained probability of corrosion. While a nominal cover of 70 mm for a concrete quality of Type 1 would only give a service period of about 25 years, increased concrete covers of up to 90 and

Table 6.6 Input parameters for analysing the effect of increased nominal concrete cover

Input parameter	*Average value*	*Standard deviation*	*Comments*
D_{28}	6.0	0. 64	Chloride diffusivity ($10^{-12} \times m^2/s$)
α	0.40	0.08	Time-dependence factor
C_{CR}	0.40	0.10	Critical chloride content (% by wt. of binder)
C_s	5.5	1.3	Chloride load (% by wt. of binder)
X_C	70	6	Nominal concrete cover (mm)
	90	6	
	120	6	

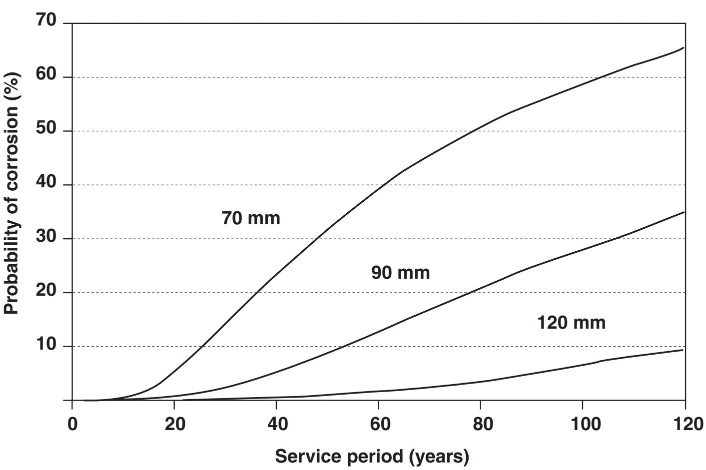


Figure 6.13 Effect of increased nominal concrete cover on the probability of corrosion.

120 mm would increase the service periods to about 50 and 120 years, respectively, before the level of 10 per cent probability for corrosion would be reached.

For certain concrete structures or structural members, it may be difficult to increase the nominal concrete cover very much beyond 90 mm without also increasing the risk of unacceptable crack widths. To a certain extent, this effect may be counteracted by a proper use of synthetic fibres in the concrete. However, by replacing the outer layer of the reinforcement system with a proper quality stainless steel, an increased effective concrete cover to the black steel of both 120 mm and over may be obtained. In this way, the durability analyses may also provide a proper basis for quantifying how much of the black steel needs to be replaced with stainless steel reinforcement in order to meet the required safety level against steel corrosion.

6.7 Evaluation and discussion of obtained results

As shown above, durability analyses may be used in comparing and selecting one of several technical solutions in order to obtain a best possible durability for a given concrete structure in a given environment. For evaluation of the obtained results, however, it is important to be aware of all the simplifications and assumptions made in the above procedures for the calculation of corrosion probability. Although diffusion is a dominating transport mechanism through thick concrete covers in chloride-containing environments, only a very simple diffusion model for the calculation of chloride penetration rates is used. As discussed in Chapter 3, the diffusion behaviour of chloride ions in concrete is a much more complex transport process than that described by Ficks 2. Law of Diffusion. Under more realistic conditions in the field, other transport mechanisms for chloride penetration than pure diffusion exist. The characterization of the resistance of the concrete to chloride penetration is further based on a rapid migration type of testing, where the chloride penetration is very different from that which takes place under more realistic conditions in the field. This type of testing primarily distinguishes the differences in the chloride mobility from one type of concrete to another, and does not properly reflect the differences in the ability of the various binder systems to bind chlorides. The durability analyses are also based on a number of other input parameters, for which there is a lack of reliable data and information. In particular, this is true for the input parameters such as the chloride loads and the ageing factors for the chloride diffusivities. Although a selection of these parameters should preferably be based on current experience from other similar concrete structures in similar environments, such information is not necessarily available. Therefore, the selection of these parameters is normally based on general experience. The temperature is also another important factor, a proper value for which may also be difficult to select.

Based on the above simplifications and assumptions, therefore, the obtained 'service periods' with a probability for corrosion of less than 10 per cent should not be considered as real service periods for the given concrete structure. In addition, based on all the above simplifications, the current procedures for predicting a probability of corrosion should not be used for calculations of service periods of more than 150 years. However, the durability analyses provide a basis for an engineering judgement of the most important factors which are considered relevant for the durability, including the scatter and variability of all factors involved. Hence, a proper basis for comparing and selecting one of several technical solutions in order to obtain a best possible durability for a given concrete structure in a given environment is obtained. As a result, a type of durability requirement may be specified which is also possible to verify and control in such a way that documentation of compliance with the specified durability can be provided, as outlined and discussed in Chapters 8 and 9.

7 Additional protective strategies and measures

7.1 General

Based on local environmental conditions and concrete qualities available, it may not be possible to meet the overall durability specification based on the required service period before a 10 per cent probability for corrosion is reached. In addition, for certain concrete structures, the owner may require a service period of more than 150 years, for which the current procedure for calculation of corrosion probability is considered neither valid nor relevant. In severe marine environments there may be a further risk for early age exposure before the concrete has gained sufficient maturity and density. In all such cases there are several additional strategies and protective measures which may be applied to ensure proper durability, some of which are briefly outlined and discussed in the following. Since all such additional protective measures may have special implications both for the economy of the project and for the future operation of the structure, such measures should always be discussed with the owner before a special strategy and measure is selected.

7.2 Stainless steel reinforcement

Reinforcement based on stainless steel has been on the market already since the early 1930s, and as previously discussed for the Progreso Pier on the Yocatán Coast in Mexico (Chapter 2), the additional costs of stainless steel in this pier later on proved to be an extremely good investment for the owner. For this pier, which was constructed between 1937 and 1941, a stainless steel of grade AISI 304 (W.1.4301) was applied (Knudsen and Skovsgaard, 1999). Traditionally, however, the additional costs of such reinforcement have been so high that it has normally not been considered viable for ordinary concrete structures. During recent years, however, new experience has shown that a more selective use of stainless steel reinforcement can be very attractive for enhancing durability and service life (Knudsen *et al.*, 1998; Materen and Paulsson-Tralla, 2001; Knudsen and Goltermann, 2004). For many years, a galvanic coupling between reinforcing bars based on stainless steel and carbon steel was believed to represent a potential corrosion risk. Both extensive experimental investigations and practical experience have shown, however, that a partial use of stainless steel in coupling with carbon steel in concrete does not increase the risk of corrosion (Bertolini *et al.*, 2004). As a consequence, a partial replacement of the carbon steel with stainless steel in the outer parts of the structure or in the most critical and vulnerable parts of the structure has shown to be a very good technical and cost-effective solution.

There are many different types of stainless steel reinforcement available on the market, but depending on the composition and microstructure of the steel there are basically three different groups:

- ferritic
- austenitic
- austenitic-ferritic (duplex).

The corrosion resistance required for chloride-containing environments mainly depends on the alloying elements such as chromium, molybdenum and nitrogen. Therefore, in order to provide a relative measure and comparable value for the corrosion resistance of the various types of steel, the so-called Pitting Resistance Equivalent Number (PREN) is used. For ordinary reinforcement steel, the PREN value is less than one. Depending on the type of steel, the PREN value may be estimated on the basis of the following expressions:

- austenitic steels: % Cr + 3.3% Mo + 16% N
- duplex steels: % Cr + 3.3% Mo + 30% N.

A classification of all the various types of stainless steel is given both in the European Standard EN 10088–1 (CEN, 1995) and the US Standard AISI, from which some of the most used and relevant types of steel are shown in Table 7.1.

In Table 7.1, some figures for the relative cost of the various types of steel are also indicated. Since the price of the alloying components may change rapidly on the international market, however, any figures regarding costs may also change rapidly. Thus, if the price of nickel becomes very high during a certain period, the lean types of duplex steel such as 1.4162 and 1.4362, which are low in nickel, will also become more cost-effective. Both of these types of steel have high PREN values of 26–27 (Outo Kumpu, 2008). The high corrosion resistance of 1.4362 may also be seen in Figure 7.1, where the pitting potential is close to that of the austenitic types of steel such as 1.4301/4404 and 4571. Reinforcement in the form of stainless steel-clad carbon steel is also available on the market (Rasheeduzzafar *et al.*, 1992; Clemeña, 2002). Although stainless steel cladding may also be quite effective, possible defects in the cladding during bending may reduce the protective effect (Clemeña and Virmani, 2004).

Table 7.1 Some relevant types of stainless steel reinforcement

EN 10088–1	*AISI*	*Diameter*	f_y *(mm)*	f_u *(MPa)*	*PREN (MPa)*	*Relative cost per kg[3] (2003)*
1.4301[1]	304	3–40	>500	600–800	19	5.5 (2.1–2.8)
1.4401[1]	316	–	–	–	24	9.5 (2.5–3.2)
1.4429[1]	316LN	18–40	>500	600–800		
1.4436[1]	316S33	3–16	>550	600–800		
1.4362[2]	2304	3–12	>650	800–1000	27	6.0
1.4362[2]	2304	14–40	>550	700–850		
1.4462[2]	318	3–16	>650	800–1000	35	8.5 (3.5–4.8)
1.4462[2]	318	18–40	>550	700–850	35	8.5

Notes
1 Austenitic.
2 Duplex.
3 2007.
where:

f_y Yield strength (0.2-limit)
f_u Ultimate strength
Relative cost Approximate ratio between materials costs of the stainless steel and ordinary steel per kg. The numbers in brackets indicate the ratio of total costs including handling.

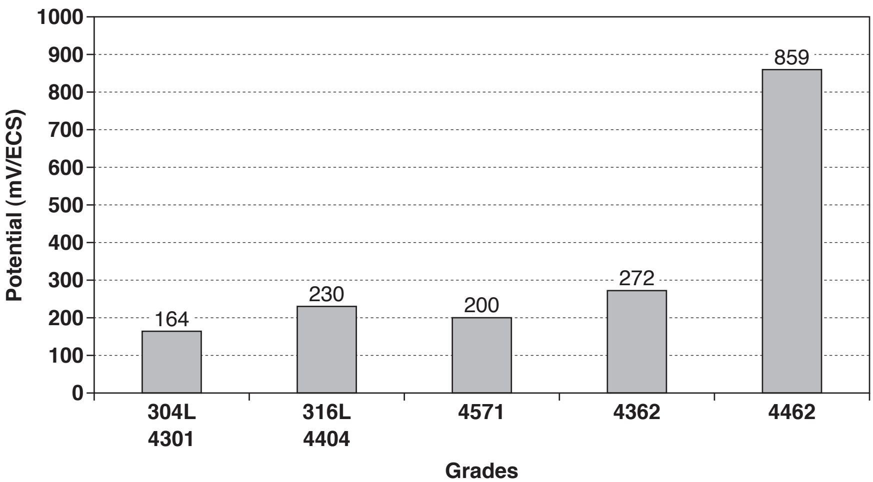


Figure 7.1 Pitting potential (mV/ECS) for various grades of stainless steel in a synthetic medium with pH of 8 and a chloride content of 21 g/l (source: Chauveau and Demelin (2007)).

While traditional carbon steel in concrete may only have a chloride threshold of typical 0.74 kg/m^3 for corrosion, stainless steel of type 1.4301 (304) and type 1.4401 (316) may have threshold values of about 11 and 18 kg/m^3, respectively (Cramer *et al.*, 2002). In Chapter 3, an upper chloride content of 0.40 per cent by weight of cement for traditional carbon steel was indicated. According to Bertolini *et al.* (2004), austenitic steel 1.4307 (304L) may be safely used for chloride content of up to 5 per cent by weight of cement, while both 1.4404 (316L) and duplex stainless steel (1.4462) may be used for even higher chloride content. In the presence of a welding scale on the steel surface, however, a lower chloride content of 3.5 per cent should be assumed (Sørensen *et al.*, 1990). Even in very severe environmental conditions, such high chloride contents are rarely ever reached in the vicinity of the embedded steel. Hence, a partial replacement of the carbon steel even with a simple type of stainless steel in the outer layer of the rebar system can substantially increase the service life of the structure. By using the increased depth of concrete cover to the more vulnerable carbon steel further in as input parameter for the durability analysis, a very good basis is obtained for quantification of how much of the carbon steel it is necessary to replace in order to meet the overall durability requirement (Chapter 6).

By replacing up to 40 per cent of the traditional carbon steel reinforcement with stainless steel in the most vulnerable parts of a typical concrete harbour structure, calculations have shown that the total costs of the structure may not increase by more than about 5 per cent (Isaksen, 2004). In the USA, the Oregon State Department of Transportation has for several years specified the use of austenitic and duplex steel for new concrete bridge construction along the coast of Oregon (Cramer *et al.*, 2002). This specification is based on requirements for yield strength, corrosion threshold and corrosion rate, and the specification is 'allowable' in that it allows the contractor to choose between 316 LN, duplex 2205 or Nitronic 50. In addition to the specification of a concrete with a very high resistance to chloride penetration, stainless steel is being specified both for deck beams and precast, prestressed girders of the bridges. In spite of the high additional costs for the use of stainless steel, the total project costs for three concrete coastal bridge projects increased by only about 10 per cent compared to the equivalent quantity of traditional carbon steel (Table 7.2). At a minimum, it was anticipated that the use of stainless steel in these bridges would double the bridge life while cutting cumulative costs relative to conventional steel reinforcement by 50 per cent over the specified service life of 120+ years for all the bridges.

In addition to be a cost-effective solution, a proper utilization of stainless steel as briefly outlined above has also been shown to be a very simple and robust strategy for achieving a more controlled durability of many important concrete structures in severe environments (Chapter 11). For further information about selection and use of stainless steel reinforcement, reference is made to the more specialized literature referred to in the reference section.

7.3 Cathodic prevention

As already discussed in Chapter 2, extensive experience has demonstrated that cathodic protection is the most efficient way of bringing heavily

Table 7.2 Material costs for three concrete coastal bridge projects in Oregon using stainless steel bars in the most exposed parts of the bridges

Project	*Brush Creek (1998)*	*Smith River (1999)*	*Haynes Inlet (2003)*
Stainless steel bar			
Uses	Deck beams	Precast, prestressed girders[1]	Deck beams
Alloy	316N	316N	316LN
Yield strength (MPa)	414	414	517
Unit price ($/kg)	7.88	262.47/girder-meters	5.02
Quantity (kg)	42,270	2713 girder-meters	320,000
Total stainless cost[2] ($)	333,660	712,080	1,610,000
Equivalent black iron bar cost[2] ($)	107,790	Not available	486,400
Black iron bar			
Unit price ($/kg)	2.55	Not available	1.52
Quantity (kg)	69,550	Not available	600,000
Total black iron bar cost[2] ($)	187,020	390,900	900,000
Project summary			
Total project cost[2] ($)	2,259,380	8,565,080	11,055,400
Stainless cost as % of project cost	14.8%	8.3%	14.5%
Stainless cost premium over black iron bar as % of project cost	10.0%	Not available	10.2%

Source: Cramer *et al.* (2002).

Notes
1 Reinforcing bar cast in girders.
2 1999 US dollars.

chloride-induced corrosion under control. For the design of a proper cathodic protection system, however, the challenge is always to provide the necessary protective current in a reliable, controllable and durable way (Bertolini *et al.*, 2008). For such a protective measure to be effective, a proper electrical continuity within the rebar system must also be provided (CEN, 2000). To provide such continuity during subsequent concrete repair is technically much more difficult and substantially more costly compared to that of providing such continuity already during construction of the new structure. For new concrete structures, therefore, specification of proper electrical continuity right from the beginning is both a cheap and proper strategy for possible applications of cathodic protection later on. For new concrete structures, an even better strategy would not only be to prepare for future cathodic protection, but also to install cathodic protection right from the beginning or to install it at a later stage before the chlorides have reached embedded steel and caused any corrosion damage. By denoting this approach for cathodic prevention, this technique was suggested and introduced by Pedeferri already in the late 1980s (Pedeferri, 1992). Cathodic prevention has successfully been applied to a large number of concrete bridges (Bertolini, 2000; COST, 2003).

In principle, cathodic prevention is based on the fact that the chloride threshold increases as the electrochemical potential of the steel decreases. According to Bertolini *at al.* (2004), very low current densities of less than 2 mA/m^2 are needed in order to bring the potential to values which will suppress initiation of pitting even at high chloride concentrations at the steel surface. For concrete structures submerged in seawater, cathodic prevention will also effectively reduce the rate of chloride penetration by counteracting the chloride diffusion through the concrete cover, as shown in Figure 7.2.

In order to compare cathodic prevention with other protective measures, it may be difficult to estimate the long-term performance and service life of the protective system. In addition to the initial costs, the costs of both operation and maintenance of the electrode system, wires and sensitive electronic equipment must be properly taken into account (Broomfield, 1997; COST, 2003). In addition, in order to install the cathodic prevention at a stage before the chlorides cause any corrosion damage, a very close follow-up of the rates of chloride penetration during operation of the structure must also be carried out. This may represent a challenge,

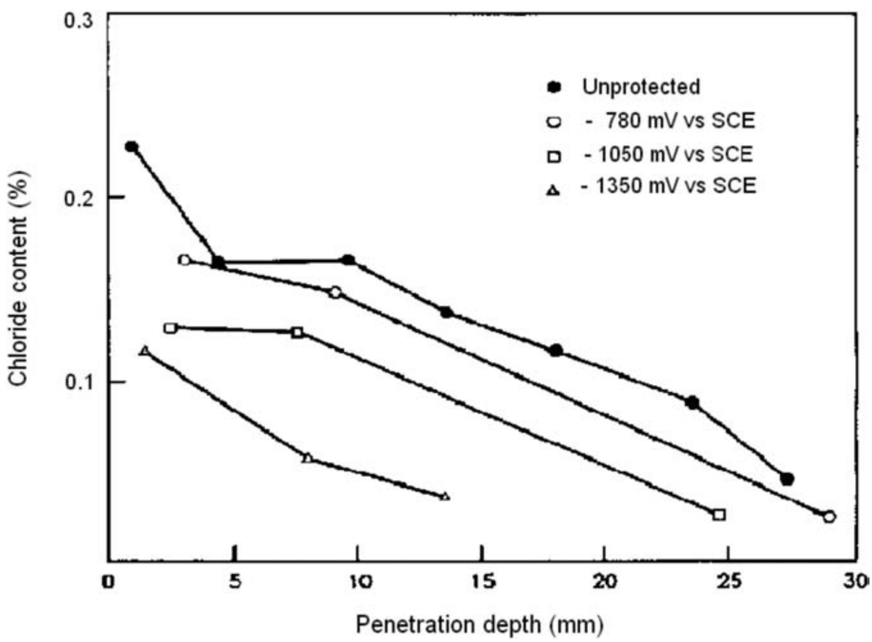


Figure 7.2 Effect of protection potential on chloride penetration into submerged concrete structures (source: Gjørv and Vennesland (1987)).

however, both due to a high variability in the environmental loads and the achieved construction quality of the structure (Chapter 2).

7.4 Non-metallic reinforcement

In recent years, applications of fibre-reinforced polymer composites (FRP) as reinforcement for concrete structures in severe environments have been growing rapidly. Although most of the experience and durability data of FRP composite installations come from the aerospace, marine and corrosion-resistance industries, FRP composites have already been used as a construction material for several decades and for products to reinforce concrete structures since the mid-1950s (ACMA MDA, 2006). A major development of FRP for civil engineering was the application of externally bonded FRP for the rehabilitation and strengthening of existing concrete structures. During the late 1970s and early 1980s, however, a variety of new applications of composite reinforcing products were demonstrated, and already in 1986, the world's first highway bridge using composite tendons was built in Germany.

Since the FRP composites have very high mechanical properties (Table 7.3), such products also represent a viable alternative to conventionally prestressed steel tendons. For a long time, suitable anchorage systems was a problem, but in recent years, new types of anchorage systems have also been developed for practical applications of prestressed systems based on FRP tendons (Gaubinger *et al.*, 2002).

Until recently, glass fibres have been the most predominant type of reinforcing fibre, mostly in the form of E-glass formulation, but also in the form of alkali resistance glass (AR glass). Due both to the rapid increase in production capacity and to new production methods, the costs of both carbon and aramid fibres have also made such fibres more easily available and attractive. More recently, however, basalt fibres with improved properties compared to those of glass fibres have become available (BlackBull, 2008). These fibres do not have any durability problems in the highly alkaline environment of concrete. In addition, the cost of these fibres is equivalent to that of glass fibres and approximately 1/10 compared to that of carbon fibres. Embedded in a proper type of a polymer-based matrix, any of the above types of fibres are currently available in various qualities and dimensions as reinforcing bars for concrete reinforcement.

In principle, the fundamental design methodologies for FRP products are similar to those of conventional steel-reinforced concrete. Cross-sectional equilibrium, strain compatibility and constitutive material behaviour form the basis of all approaches to the design of reinforced concrete structures, regardless of the reinforcing material. For the structural design, the non-ductile and anisotropic nature of FRP reinforcing products needs to be specially addressed, but in current guidelines and recommendations for structural design, this is properly taken into account (ACI, 2007).

Table 7.3 Mechanical properties of advanced composite fibres

	Armid fibre (Twaron HM)	*Glass fibre (E-glass)*	*Basalt fibre*	*Carbon fibre (HT)*	*CFRP wire*	*Steel strand (St 1570/1770)*
Tensile strength (MPa)	2600	2300	3200	3500/7000	2800	>1770
Young's modulus (GPa)	125	74	90	230/650	160	205
Ultimate strain (%)	2.3	3.3	3.0	0.6/2.4	1.6	7
Density (g/cm^3)	1.45	2.54	2.6	1.8	1.5	7.85

Sources: Noisternig *et al.* (1998); BlackBull (2008).

7.5 Corrosion inhibitors

Corrosion inhibitors both for the prevention and delay of corrosion initiation have been on the market for a long time. A number of different inhibitors for addition to the fresh concrete exist, but the protecting mechanisms may be quite different from one type of product to another (Büchler, 2005). Of the various types of product, calcium nitrite is probably the most extensively and widely tested corrosion inhibitor applied so far. Extensive investigations have shown that a proper addition of this inhibitor is capable of both preventing corrosion and decreasing the corrosion rate (Hinatsu *et al.*, 1990). In order to be efficient, however, a critical ratio in the range of 0.5 to 1.0 between nitrite and chloride has to be present (Andrade, 1986; Gaidis and Rosenberg, 1987). In addition, since corrosion activation will consume the substance over time, while the concentration of chlorides will increase, it may be difficult to predict and guarantee the long-term effects of such a protective measure (Hinatsu *et al.*, 1990). If the concentration of the calcium nitrite over time becomes too low, corrosion acceleration may be expected (Nürnberger, 1988; Ngala *et al.*, 2002).

7.6 Concrete surface protection

As discussed in Chapter 2, those offshore concrete platforms that had a thick epoxy coating applied to the concrete surface during concrete construction have subsequently been very effectively protected from chloride penetration (Aarstein *et al.*, 1998). As this surface coating was continuously applied to the concrete surface during slip forming when the young concrete still had an under-pressure and suction ability, a very strong bond was achieved between the concrete substrate and the coating.

In recent years, a number of new surface protection products have been introduced either for retarding or preventing chloride penetration into concrete structures. In principle, the effect of such products may either be to make the concrete surface less permeable to chloride penetration or to reduce the moisture content in the concrete, although many products combine these effects.

Different types of surface protection products may be grouped into the following four classes as schematically shown in Figure 7.3, where (a) shows organic coatings that form a continuous film on the concrete surface, (b) shows hydrophobic treatments that line the interior surface of the concrete pores, (c) shows treatments that fill the capillary pores, and (d) shows a thick and dense cementitious layer. While surface treatments open to water vapour may have a longer service life with no or slower loss of bond, the effectiveness of such surface treatments may generally be less than that of dense protective systems. It appears, therefore, to be a tradeoff between good barrier properties and good long-term effects of the surface treatment (Bertolini *et al.*, 2004).

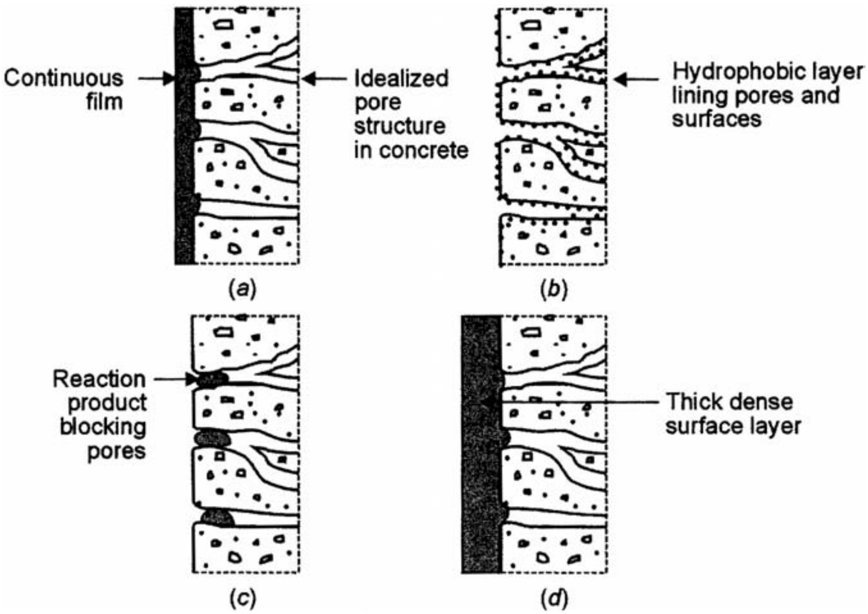


Figure 7.3 Schematic representation of different types of concrete surface protection: (a) organic coating, (b) pore-lining treatments, (c) pore-blocking treatments, (d) thick cementitious coating, shotcrete or rendering (source: Bijen (1989)).

In recent years, surface hydrophobation of concrete structures has been widely adopted as a protective measure for concrete structures in severe environments. Since the protective efficiency of such a polymer-based surface treatment may be reduced over time due to weathering, most types of such treatments need to be re-applied from time to time in order to ensure proper long-term protection. If the surface hydrophobation is applied at a later stage where the chlorides have already reached a certain depth, further chloride penetration may still take place for some time due to a redistribution of the chloride content (Arntsen, 2001; Årskog *et al.*, 2004a). However, in severe marine environments, a surface hydrophobation may primarily be applied during concrete construction in order to protect the concrete against early age exposure before the concrete has gained sufficient maturity and density.

In order to investigate the protective effects of a surface hydrophobation against early age chloride exposure, an experimental programme based on concrete with a water/cement ratio of 0.45 was carried out (Liu *et al.*, 2005). After seven days of concrete curing either at a temperature of 20° or 5°C and a relative humidity of 50 per cent, 95 per cent or 100 per cent RH, concrete specimens were surface treated with a hydrophobic gel

Table 7.4 Effect of temperature and moisture conditions on penetration depth of hydrophobic agent

Code	*Penetration depth (mm)*
T-20–50	9.6 ± 1.2
T-20–100	2.0 ± 2.8
T-05–95	2.8 ± 0.3
T-05–100	<0.1

consisting mainly of an isobutyl triethoxy type of silane. Before exposure, a penetration depth of the hydrophobic agent as shown in Table 7.4 was observed, and after six weeks of intermittent spraying and drying to a salt solution, the protective efficiency was evaluated on the basis of both depth of chloride penetration (C_x), surface chloride concentration (C_s) and apparent chloride diffusivity (D_a) calculated on the basis of Ficks 2. Law of Diffusion. In addition, the chloride penetration rate (V) was also calculated as the total amount of penetrated chlorides divided by the area of exposed surface and time of exposure (g/m^2/s).

For the curing conditions at 20°C and 50 per cent RH, it may be seen from Table 7.5 that the surface treatment reduced the depth of chloride penetration, apparent chloride diffusivity and chloride penetration rate from 7.8 to 1.6 mm, from 6.5 to 0.3×10^{-12} m^2/s and from 3.0 to 0.9×10^{-5} g/m^2/s, respectively. For increased relative humidity to 100 per cent, however, a depth of chloride penetration, apparent chloride diffusivity and chloride penetration rate of 5.1 mm, 4.4×10^{-12} m^2/s and 1.7×10^{-5} g/m^2/s, respectively, were observed. For a combination of increased humidity to 100 per cent RH and reduced temperature to 5°C, the corresponding numbers were 7.7 mm, 8.3×10^{-12} m^2/s and 1.7×10^{-5} g/m^2/s, respectively. Although the surface chloride concentration did not reflect the efficiency of

Table 7.5 Chloride penetration in untreated (U) and treated (T) concrete surface after early age exposure

Code	*Penetration depth (C_x) (mm)*	*Surface concentration (C_s) (% concrete weight)*	*Apparent chloride diffusion coefficient (D_a) (10^{-12} m^2/s)*	*Chloride penetration rate (V) (10^{-5} g/m^2/s)*
U-20–50	7.8 ± 1.0	0.64	6.50	2.96
T-20–50	1.6 ± 0.5	0.41 ± 0.02	0.29 ± 0.03	0.87 ± 0.01
U-20–100	6.9 ± 1.0	0.35	6.10	1.50
T-20–100	5.1 ± 1.1	0.40 ± 0.10	4.42 ± 0.74	1.68 ± 0.30
U-05–95	10.0 ± 0.8	0.62	9.80	3.33
T-05–95	5.4 ± 1.4	0.19 ± 0.02	12.80 ± 2.97	1.16 ± 0.02
U-05–100	9.1 ± 0.5	0.61	6.74	3.06
T-05–100	7.7 ± 0.1	0.34 ± 0.02	8.29 ± 0.05	1.72 ± 0.07

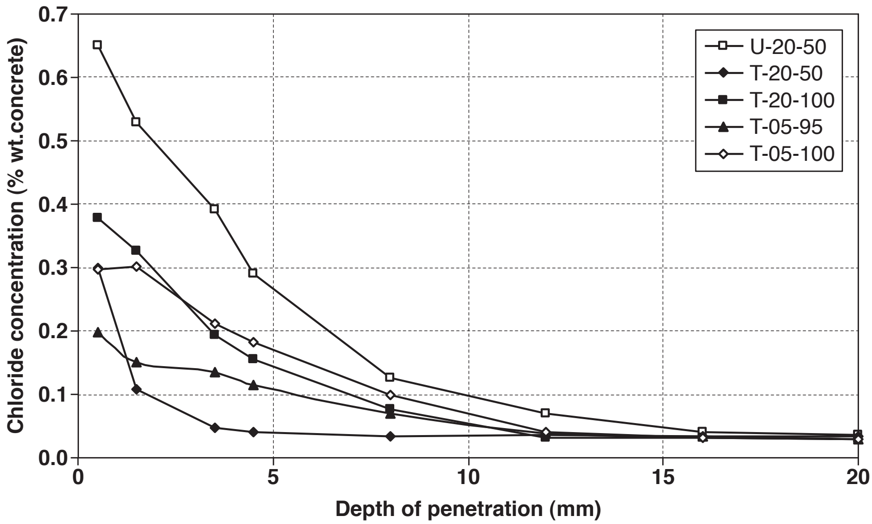


Figure 7.4 Protective effect of a surface hydrophobation against early age chloride exposure (source: Liu *et al.* (2005)).

the surface treatment, there was a good correlation between the penetration depth of the hydrophobic agent (Table 7.6) and the protective efficiency of the surface treatment. As may be seen from both Table 7.5 and Figure 7.4, however, the protective efficiency of the surface treatment was not very good on the moist concrete substrate. For the moist substrate, a low temperature was not very important for the efficiency of the surface treatment.

For many concrete structures in severe environments, the moisture content in the surface layer of the concrete may be quite high (Chapter 2). Therefore, in order to investigate the protective efficiency of a surface hydrophobation under more realistic conditions in the field, some tests on a new concrete harbour structure were carried out shortly after concrete construction (Liu, 2006). The surface treatments included two types of gel-based products, one of which consisted mainly of an isooctyl trietoxy type of silan (T-A), while the other was based on an isobutyl trietoxy silan (T-B). Both products were partly applied in one layer with achieved thickness of approximately 0.25 mm and partly in two layers with achieved thickness of approximately 0.5 mm. The moisture content in the concrete substrate was so high that none of the surface treatments gave any observed depth for penetration of the hydrophobic agent. After four years of exposure to the splashing of seawater, however, it may be seen from Figure 7.5 that all the applied protective systems had provided about the same good protection against chloride penetration. Compared to the observed chloride penetration in the unprotected parts of the concrete deck, the surface

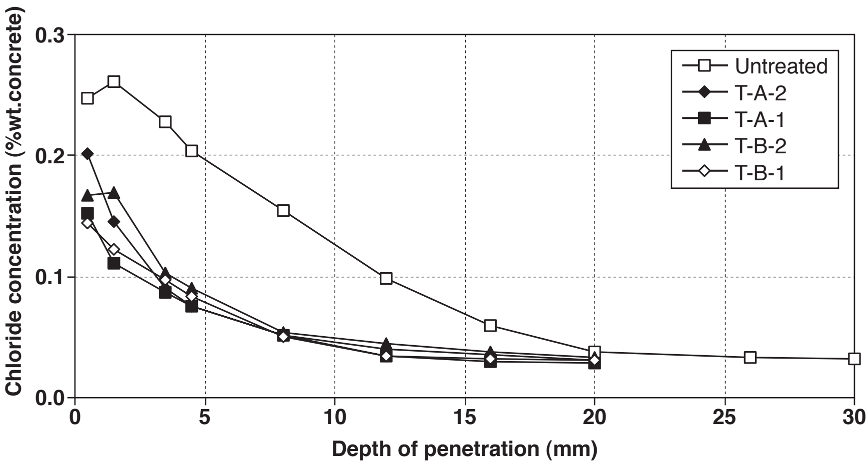


Figure 7.5 Effect of hydrophobic surface treatment for the early age protection of a concrete harbour structure against chloride penetration (source: Liu (2006)).

hydrophobation had reduced the surface chloride concentration, apparent chloride diffusivity and chloride penetration rate by a factor of 0.5, 0.7 and 0.5, respectively.

In the literature, extensive experience with various types of protective surface systems applied to concrete structures in severe environments has been reported, good reviews of which are given both in COST Action 521 (Cost, 2003), Bertolini *et al.* (2004) and Raupach and Rößler (2005). The proceedings from the international conferences on water repellent treatment of building materials also reflect much of the current knowledge and experience with this type of protective surface system, the last proceedings of which were published from the fourth and fifth conferences organized in Stockholm in 2005 (Silfwerbrand, 2005) and in Brussels in 2008 (De Clereq and Charola, 2008), respectively.

By use of certain admixtures to the fresh concrete, it is also possible to make the whole mass of the concrete hydrophobic (Evonic, 2008). Such approach may be appropriate for certain critical parts of the structure.

7.7 Prefabricated structural elements

For many construction projects in severe environments, there may be a number of reasons for applying prefabricated structural elements. From a durability point of view this may also be a good construction strategy. For prefabricated structural elements a more controlled construction quality may be achieved, and in severe marine environments the risk for early age exposure during concrete construction may also be reduced or avoided.

Possible surface protection systems may also be applied to the most exposed concrete surfaces under more controlled and optimum conditions before the structural elements are installed.

For many concrete harbour structures, prefabricated concrete elements are often applied both for formwork and deck slabs. For many concrete bridges, prefabricated deck beams and girders as well as large structural elements are also often used. For some bridges, the prefabricated structural elements may be very large. Thus, for the Northumberland Strait Bridge Project on the east coast of Canada (Tromposch *et al.*, 1998), both the pier shafts and the cantilever girders of up to 190 m length were prefabricated on shore before they were installed by use of a heavy floating crane (Figures 7.6–7.9).

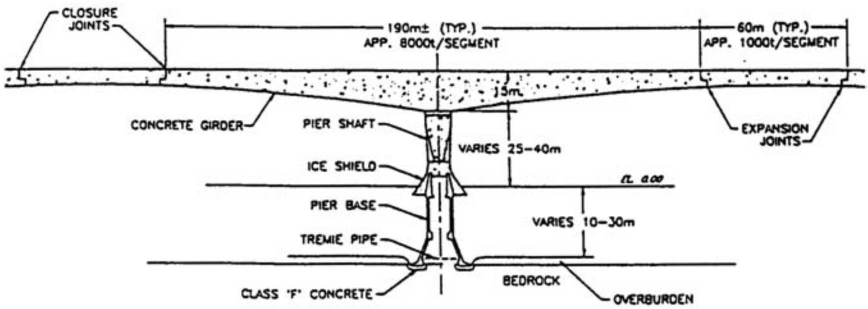


Figure 7.6 Typical foundation, pier shaft and span for the Northumberland Strait Bridge (1997) on the east coast of Canada.

Figure 7.7 Precast cantilever girder for the Northumberland Strait Bridge.

Figure 7.8 Precast cantilever girders for the Northumberland Strait Bridge.

Figure 7.9 Installation of cantilever girders for the Northumberland Strait Bridge (source: Malhotra (1996)).

8 Concrete quality control

8.1 General

As clearly shown and discussed in Chapter 2, extensive experience demonstrates that the durability of concrete structures is closely related not only to design and material but also to construction issues. Although to a certain extent a probability-based approach to the durability design can take the wide variability of construction quality into account, a numerical approach to the durability design alone is not sufficient for ensuring a proper durability. For concrete structures in severe environments, construction quality and variability is a key issue which must be firmly grasped before a more rational approach to a more controlled durability can be achieved (Sommerville, 2000).

Even before the concrete is placed in the formwork, the quality of concrete may show a high scatter and variability. Depending on a number of factors during concrete construction, the achieved quality of the finely placed concrete normally shows an even higher scatter and variability. In Chapter 2, the high scatter of achieved chloride diffusivity in the deck of a new concrete harbour structure according to the RCM method was shown (Figure 2.24). Even for the offshore concrete platforms in the North Sea, where a very high quality of both concrete production and concrete construction was carried out, the observed chloride penetration and extent of steel corrosion also reflect a high scatter and variability of achieved construction quality (Chapter 2). This may be seen both in Figures 2.56–2.58 for the Brent B Platform and in Figure 2.60 for the Brent C Platform. For the Brent B Platform, a very high scatter and variability of achieved chloride diffusivity in the utility shaft obtained by the RCM method was also observed (Figure 2.61). For both of these concrete platforms as well as for the above concrete harbour structure, the regular quality control of compressive strength during concrete construction only revealed a low scatter and variability. These results indicate, therefore, that the observed variations in compressive strength during concrete construction probably translate to substantially higher variations in the resistance of the concrete to chloride penetration.

For production of air-entrained concrete, the problem with large variations in the air void characteristics during concrete construction was discussed in Chapter 4. During production, handling and placing of air-entrained concrete, the air void characteristics may vary within wide limits, and this problem is enhanced when the concrete is produced using cements blended with fly ash of a variable carbon content. As a result, not only the frost resistance but also the resistance of the concrete against chloride penetration may be significantly affected.

Probably the best-known and well-documented quality issue of concrete structures is the failure to meet specified requirements for concrete cover. The observations of achieved concrete cover in the Norwegian bridge previously shown in Figure 2.40 reflect a general problem for concrete structures in many countries. Thus, in Figure 8.1, the data from this bridge have been plotted together with similar data from a Japanese bridge (Ohta *et al.*, 1992) and the average of data from more than 100 concrete structures in the Gulf Region (Matta, 1993). In recent years, therefore, improved codes and procedures for achieving the specified concrete cover

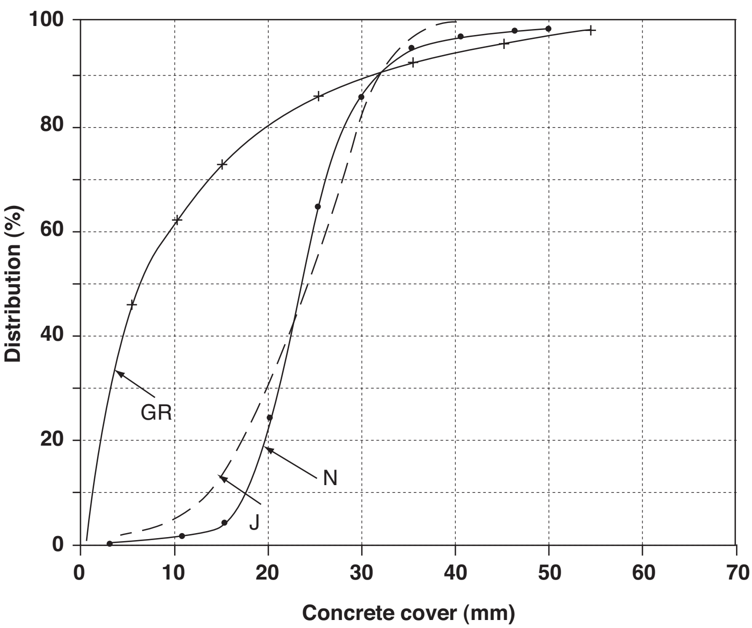


Figure 8.1 Variation in achieved concrete cover (mm) in the Gimsøystraumen Bridge (N) previously shown in Figure 2.40 compared to that of a Japanese bridge (J) and more than 100 concrete structures in the Gulf Region (GR) (source: Kompen (1998)).

with more confidence have been introduced (Chapter 5). However, the variability of concrete cover continues to be a very difficult problem. Although the specified concrete cover is normally carefully checked and controlled before placing the concrete, experience demonstrates that significant deviations may still occur during concrete construction. The loads during placing the concrete may occasionally be too high compared to the stiffness of the rebar system, or the spacers may occasionally have been insufficiently or wrongly placed. Even during the sophisticated slip-forming work of the offshore concrete platforms for the North Sea, the installed spacers occasionally had to be removed during some critical stages of the slip forming in order to allow the slip-forming work to continue.

In order to comply with the overall durability requirement of the given structure as described in Chapter 6, a proper quality control of both the specified chloride diffusivity and the concrete cover must be carried out during concrete construction. For both of these durability parameters, average values and standard deviations must be obtained. If cathodic prevention or preparation for such a protective measure has also been specified, regular quality control of the electrical continuity within the rebar system must also be carried out during concrete construction. In the following, the necessary test methods and procedures for measurements and control of the above durability parameters are outlined and discussed.

8.2 Chloride diffusivity

General

As already described and discussed in Chapter 6, all measurements of chloride diffusivity are based on the Rapid Chloride Migration (RCM) method (NORDTEST, 1999). Although the duration of such measurements may take only a couple of days, this is not good enough for regular quality control during concrete construction. For all porous materials, however, the Nernst–Einstein equation expresses the following general relationship between the ion diffusivity and the electrical resistivity of the material (Atkins and De Paula, 2006):

$$D_i = \frac{R \cdot T}{Z^2 \cdot F^2} \cdot \frac{t_i}{\gamma_i \cdot c_i \cdot \rho} \tag{8.1}$$

where:

D_i = diffusivity for ion i
R = gas constant
T = absolute temperature
Z = ionic valence

F = Faraday constant
t_i = transfer number of ion i
γ_i = activity coefficient for ion i
c_i = concentration of ion i in the pore water
ρ = electrical resistivity.

Since most of the factors in equation 8.1 are physical constants, the above relationship for a given concrete with given temperature and moisture conditions may be simplified to:

$$D = k \cdot \frac{1}{\rho} \tag{8.2}$$

where D is the chloride diffusivity, k is a constant and ρ is the electrical resistivity of the concrete. Since the electrical resistivity of the concrete can be measured in a more rapid and simple way than the chloride diffusivity, it is primarily regular quality control of the electrical resistivity of the concrete which provides the basis for indirect quality control of the chloride diffusivity during concrete construction (Gjørv, 2003). Therefore, as soon as the type of concrete is given, the above relationship between chloride diffusivity and electrical resistivity must be established. This is done by producing a certain number of concrete specimens, on which parallel testing of both chloride diffusivity and electrical resistivity at different periods of water curing are carried out. After the relationship between the chloride diffusivity and the electrical resistivity has been established, this relationship is subsequently used as a calibration curve for an indirect control of the chloride diffusivity based on regular measurements of the electrical resistivity during concrete construction (Figure 8.2). Since the testing of electrical resistivity is a non-destructive type of test, these measurements are carried out on the same concrete specimens as that being used for regular quality control of the 28-day compressive strength during concrete construction.

In order to establish the above calibration curve, the measurements of chloride diffusivity are carried out on three 50 mm-thick specimens cut from Ø 100 × 200 mm concrete cylinders after approximately 7, 14, 28 and 60 days of water curing. These measurements are carried out on water-saturated specimens according to an established water-saturation procedure. In parallel, the corresponding measurements of the electrical resistivity are carried out on three concrete specimens of the same type as that being used for the regular quality control of the compressive strength. These measurements are carried out on moist concrete specimens after the various periods of water curing.

Based on the established calibration curve, the specified chloride diffusivity is indirectly controlled by the electrical resistivity measurements on all the concrete specimens used for the 28-day compressive strength during

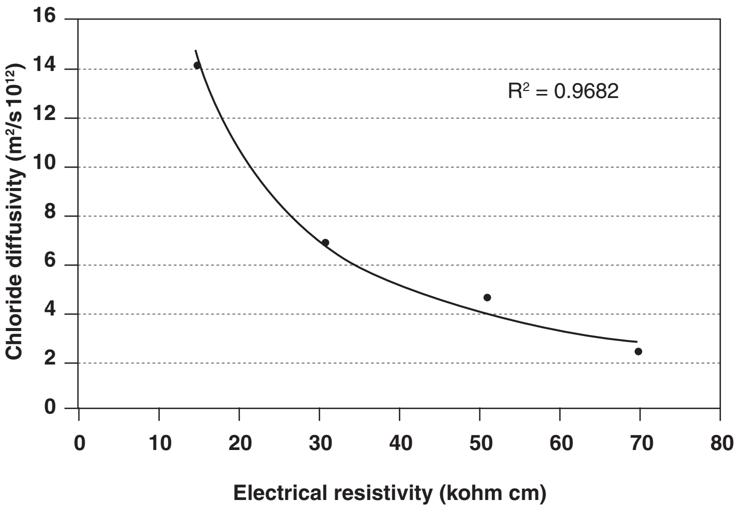


Figure 8.2 A typical calibration curve for control of chloride diffusivity based on electrical resistivity measurements.

concrete construction. Of all these control data, no individual value should exceed 30 per cent by that of the specified chloride diffusivity.

It is not the purpose here to describe all the details of the measurements of chloride diffusivity based on the RCM method. Such measurements require special testing equipment and qualified experience which are only available in professional testing laboratories. However, for a better evaluation and application of the obtained results, a brief outline of the test method is given in the following.

Test specimens

The testing is normally based on 3 × 50 mm-cut slices of a concrete cylinder with diameter Ø 100 mm. These slices may either be cut from a cast concrete cylinder or from a concrete core with the same diameter.

Testing procedure

Immediately before testing, the test specimens are preconditioned according to a standard water-saturation procedure. The specimens are then mounted into rubber sleeves and placed in a container with a 10 per cent NaCl solution (as shown in Figure 8.3), while the insides of the sleeves are filled with a 0.3 N NaOH solution. A set of concrete specimens under testing using the RCM method is shown in Figure 8.4.

Through the use of the separate electrodes placed on each side of the

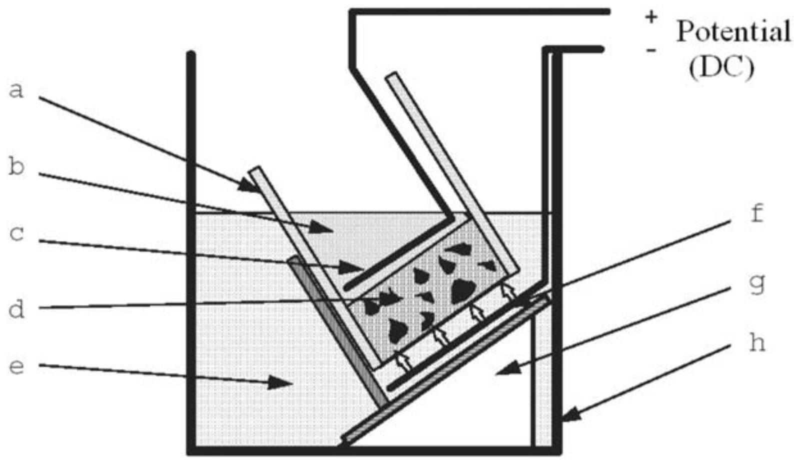


Figure 8.3 Experimental set up for the RCM testing of chloride diffusivity (D_0), where (a): rubber sleeve, (b): anolyte, (c): anode, (d): concrete specimen, (e): catholyte, (f): cathode, (g): plastic support and (h): plastic box (source: NORDTEST (1999)).

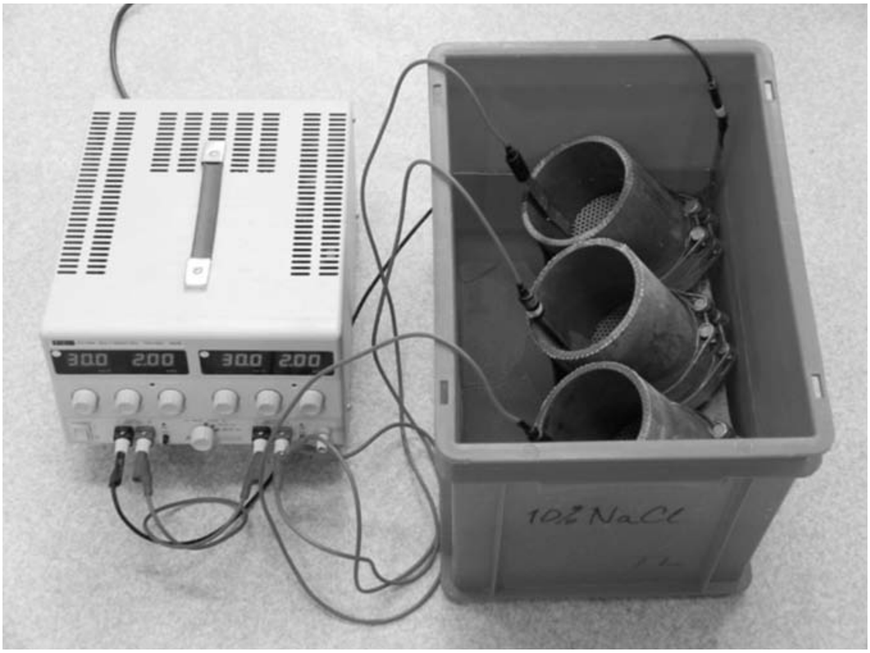


Figure 8.4 Concrete specimens under testing with the RCM method (source: Tang (2008)).

specimens, an electrical voltage gradient is applied and the electrical current passing through the specimens observed. Depending on the level of the observed current which reflects the resistance of the concrete to chloride penetration, the applied potential is adjusted in order to obtain a proper duration of the testing. By an applied DC potential which may vary from 10 to 60 V, the chloride ions are forced into the concrete specimens over a relatively short period of time. For a normal dense concrete a test duration of 24 hours may be typical, while for a more dense concrete more time may be needed.

Immediately after the above accelerated exposure to the chloride solution, the test specimens are split into two halves, from which the average depth of chloride penetration is observed on the split surface by use of a colorimetric technique (Figure 8.5). Based on the observed depth of penetration and the applied testing conditions, the concrete diffusivity is calculated according to an established procedure. As a result, the chloride diffusivity (D_0) of the concrete is obtained as an average value and standard deviation from the testing of three specimens.

Evaluation of obtained results

When the resistance of the concrete to chloride penetration is characterized by the RCM method as described above, it should be very clear that the results obtained on the basis of such an accelerated chloride penetration are very different from what is taking place under more realistic and normal conditions in the field. Therefore, this type of testing primarily distinguishes the difference in chloride mobility in the concrete from one type of concrete to another, and does not properly reflect the difference in the ability of the various types of binder system to bind the penetrating chlorides. Hence, all results obtained by the above accelerated test method should only be considered as a simplified way of comparing the resistance to chloride penetration of various types of concrete. For equal concrete

Figure 8.5 Observed depth of chloride penetration on the split concrete surface after a spraying of the surface with a standard $AgNO_3$ solution.

compositions, however, it was previously shown in Figure 6.8 how the results obtained by the above test method showed a strong statistical correlation with the apparent chloride diffusion coefficients obtained by the more time-consuming immersion test NT Build 443 (NORDTEST, 1995).

Since the RCM method requires only a short duration of testing, this is the only test method available for control of chloride diffusivity at any stage of hydration for the given concrete. For severe marine environments, the early age resistance against chloride penetration may also be an important property of the concrete as previously discussed in both Chapters 2 and 3.

Although the 28-day chloride diffusivity is normally used as a general parameter for characterizing the resistance of the given concrete against chloride penetration (Table 6.2), this parameter is occasionally also being used as a durability parameter for characterizing the general durability properties of a given concrete, not only for chloride-containing environments but also for other severe and aggressive environments. Thus, regular testing of chloride diffusivity in the same way as outlined above was also applied as part of regular concrete quality control and documentation of achieved construction quality of several concrete structures produced for a new sewage and water treatment plant in Trondheim (Rindal *et al.*, 2000).

For a more complete evaluation and comparison of the durability properties of various types of concrete, it is not only the 28-day chloride diffusivity which should be considered. As was also shown and discussed in Chapter 3, it is the more complete development curve of chloride diffusivity from an early age and upward which more properly reflects the durability properties and, hence, the different resistance to chloride penetration of various types of concrete.

8.3 Electrical resistivity

General

Several experimental techniques and test methods exist for the electrical resistivity measurements of concrete and, when applied to the same type of concrete, all these test methods also give different test results (Gjørv *et al.*, 1977; Polder *et al.*, 2000; Polder, 2001). Basically, however, there are two different types of test method which appear suitable for regular quality control during concrete construction, one of which is the two-electrode method as schematically shown in Figure 8.6.

For the two-electrode method, the applied current flows through the whole concrete specimen which is placed between two steel plates, and the resistance is observed by use of a resistance meter of a certain frequency. In order to ensure proper electrical connection between the concrete and

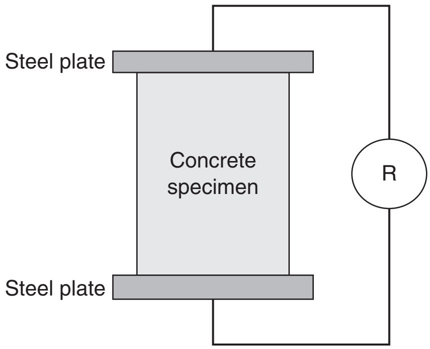


Figure 8.6 The two-electrode method for electrical resistivity measurements of concrete.

the steel plates, proper precautions have to be taken. The resistivity of the concrete (ρ) may then be calculated using the following equation:

$$\rho = R\frac{A}{t} \qquad (8.3)$$

where R is the resistance, A is the surface area of the specimen and t is the height of the concrete specimen.

The other test method is the four-electrode or the so-called Wenner method as schematically shown in Figure 8.7. For this method, the measurements are based on a low-frequency alternating electrical current passing through the concrete between the two outer electrodes, while the voltage drop between the two inner electrodes is observed. The electrical resistivity of the concrete (ρ) may then be calculated using the following equation:

$$\rho = 2\pi aV/I \qquad (8.4)$$

where a is the electrode spacing, V is the voltage drop and I is the current.

When the Wenner method is applied to different types of concrete specimens, the resulting current flow through the concrete is substantially affected by both the electrode spacing and the geometry of the concrete specimen. In principle, therefore, this test method does not have as well-defined testing conditions as that of the two-electrode method.

However, since the Wenner method was introduced by Stratfull for condition surveying of existing concrete structures already in the early 1950s (Chapter 2), this method is both widely adopted in the construction industry and easily available in the form of several commercial units. As shown in Figure 8.8, the Wenner method is also very easy to apply. Thus, the Wenner device is simply pressed against the concrete surface and the apparent resis-

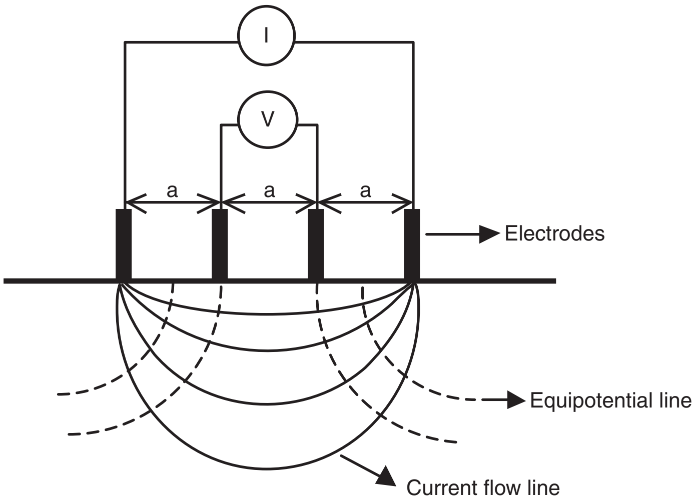


Figure 8.7 The four-electrode method (Wenner method) for electrical resistivity measurements of concrete.

tivity is directly observed on a display for the given electrode distance. For certain commercial units, the electrode distance is also adjustable.

In order to test the suitability of the Wenner method for rapid and convenient quality control during concrete construction, an experimental comparison with the two-electrode method was carried out (Sengul and Gjørv, 2008). When the Wenner method was applied to a given type of concrete specimen, different probe spacings significantly affected the observed resis-

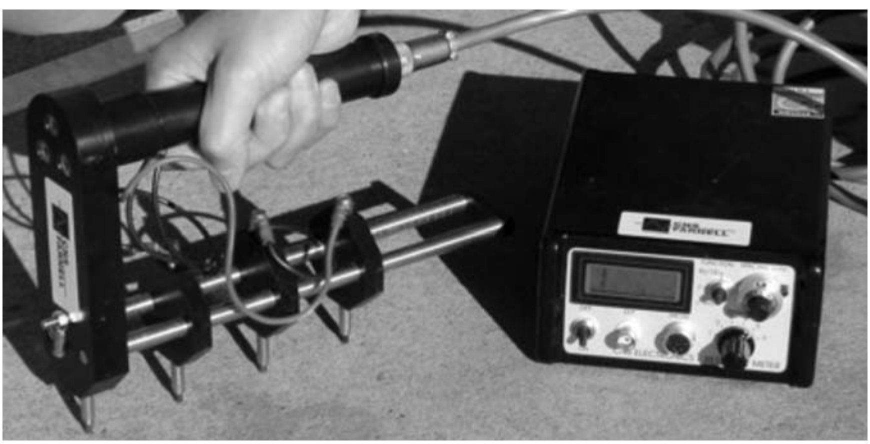


Figure 8.8 Measurement of electrical resistivity by use of a Wenner device (source: CNS Farnell (2008)).

tivity, and the resistivity increased for increased probe spacing. For different types of concrete specimen, different resistivities were also observed. However, for a given probe spacing on a given type of specimen, there was a good correlation between the results obtained by the Wenner method and those obtained by the two-electrode method (Figure 8.9). As a result, it was concluded that for a given type of concrete specimen with given moisture and temperature conditions, the Wenner method may be used as a rapid and reliable test method for regular quality control during concrete construction. For such use, however, it is essential to maintain the same conditions and established procedure throughout the testing.

Testing procedure

Since both the moisture and temperature conditions are also important factors affecting the electrical resistivity of the concrete, all measurements of the electrical resistivity must be carried out under controlled conditions in the laboratory. Immediately before testing, all surface water on the concrete surface must be carefully wiped off before the Wenner device is pressed against the concrete surface. In order to ensure a good electrical connection between the electrodes and the concrete surface, the electrodes must also be kept moist during all measurements. In order to avoid any current drain during measurements, the specimens must further be placed on a dry electrically insulated base plate and any touching of the concrete specimen by hand avoided.

If the regular quality control of compressive strength during concrete construction is based on the testing of concrete cubes, the electrical resistivity

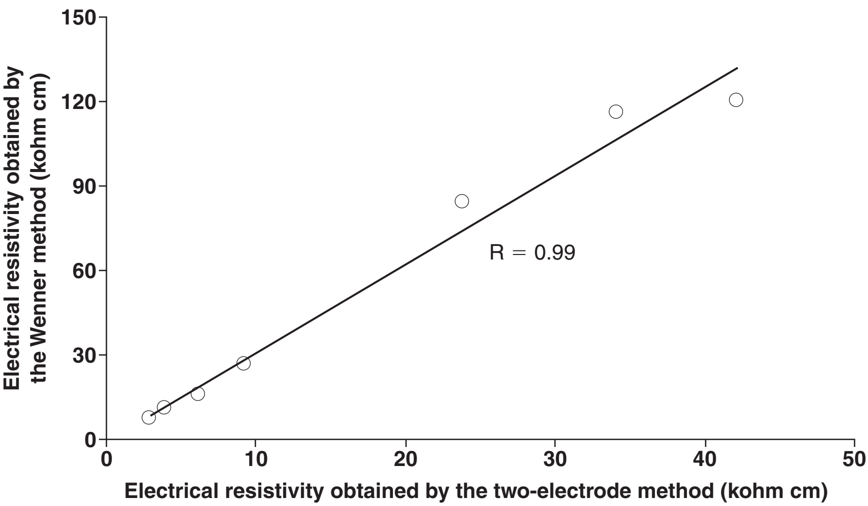


Figure 8.9 Relationship between the resistivities obtained by the Wenner method and the two-electrode method (source: Sengul and Gjørv (2008)).

is observed from two diagonal readings on two opposite sides of each cube (Figure 8.10), but the top surface of the cube should never be used for testing.

If the regular quality control of compressive strength is based on the testing of concrete cylinders, the resistivity is observed from three longitudinal readings along each cylinder, as is shown in Figure 8.11.

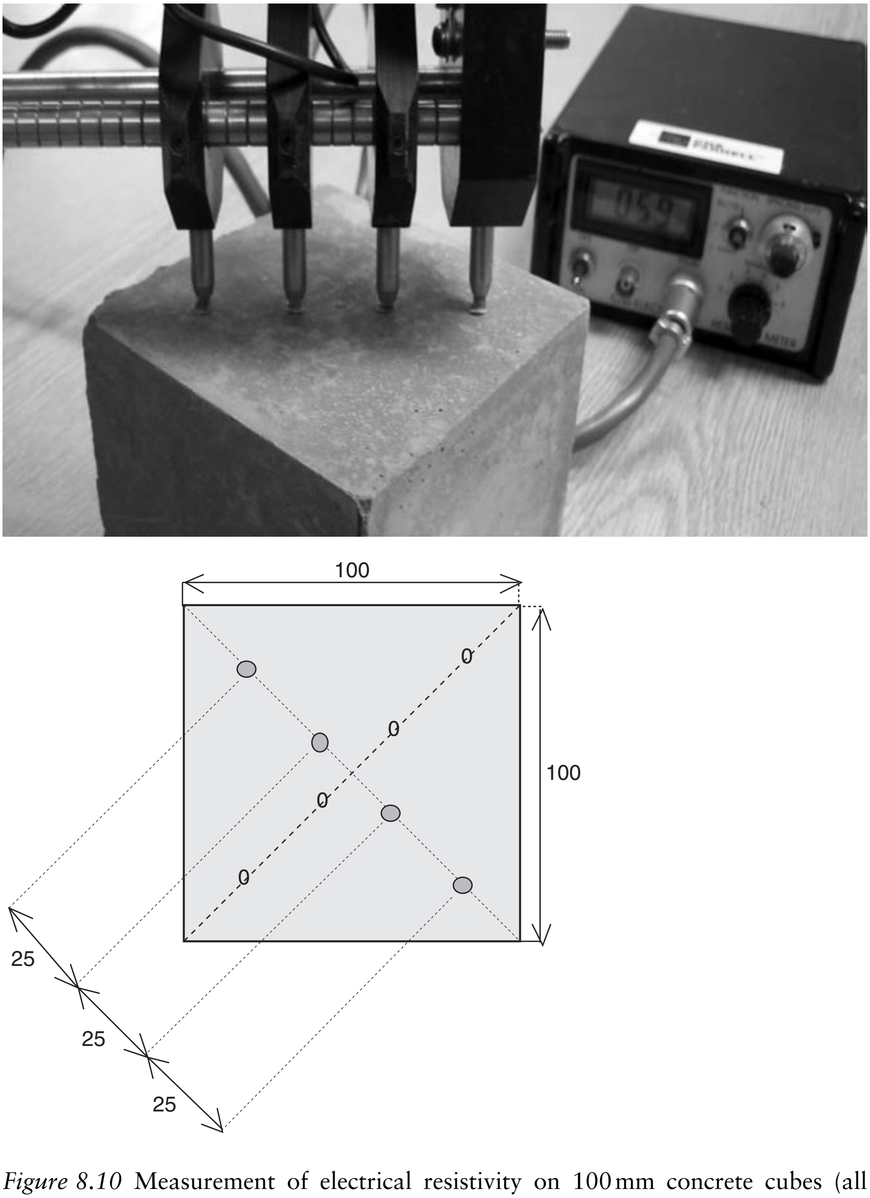


Figure 8.10 Measurement of electrical resistivity on 100 mm concrete cubes (all measures in mm).

Evaluation of obtained results

Although the above two test methods show somewhat different resistivities, the test methods show a good statistical correlation. In addition, both test methods are only used for indirect testing of chloride diffusivity based on the established calibration curve. Therefore, both types of test method can be applied for the regular quality control of electrical resistivity during concrete construction. However, the same type of test method and testing procedure as that used for the calibration curve must subsequently be applied for the regular quality control.

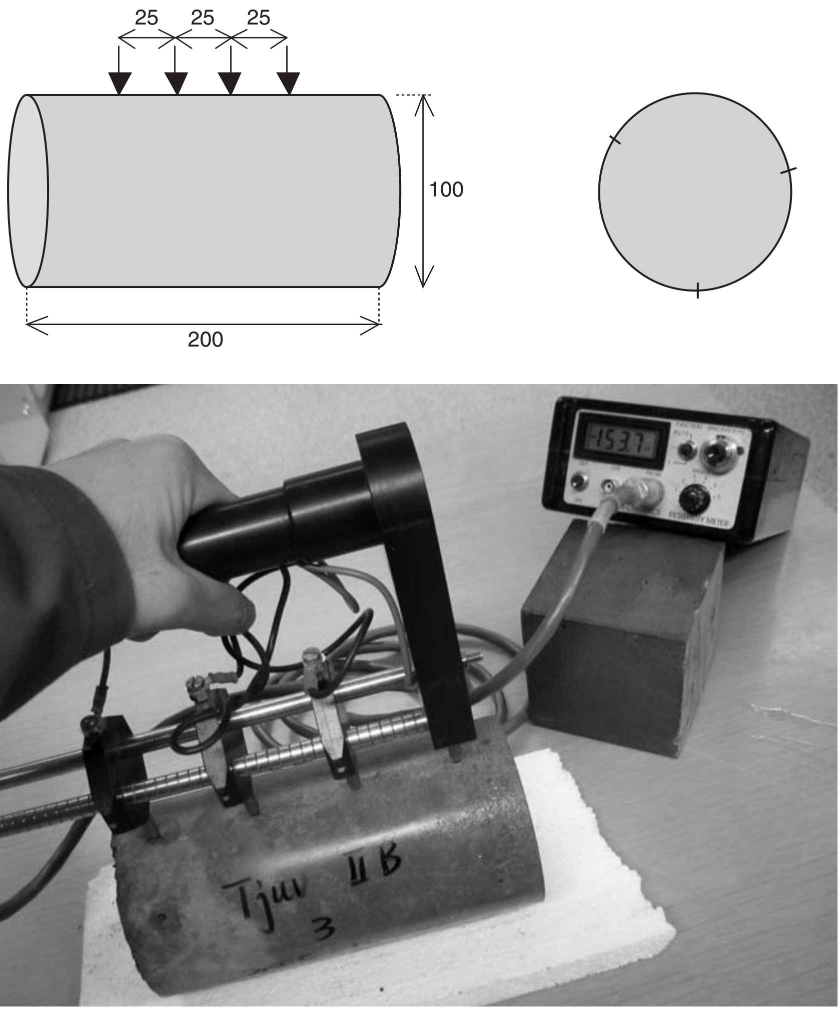


Figure 8.11 Measurement of electrical resistivity on Ø 100 × 200 mm concrete cylinders (all measures in mm).

From the quality control carried out on a number of large construction projects, the Wenner method has proved to be both a simple, rapid and reliable test method for the regular quality control during concrete construction (Chapter 11). However, as soon as the calibration curve for the control of chloride diffusivity is established, it is essential to maintain the same established testing conditions and procedures for the resistivity measurements throughout the quality control.

8.4 Concrete cover

For concrete structures in severe environments, the specified concrete cover is normally very thick and the reinforcement system may also be very congested. For such structures, therefore, it may be difficult to obtain reliable control data on achieved concrete cover based on the use of conventional cover meters. It may also be difficult to apply conventional cover meters for control of concrete cover if the reinforcement is based on stainless steel, which does not respond to magnetic measurements. However, cover meters based on a pulse-induction technique may also be used for the control of concrete cover for stainless steel reinforcement (Figure 8.12). Extensive experience has shown, however, that straight manual readings of the concrete cover on protruding bars in casting joints during concrete construction may also provide a proper basis for the regular quality control of the

Figure 8.12 Measurements of concrete cover based on pulse-induction technique (source: Germann (2008)).

achieved concrete cover. However, the extent of measurements must be sufficient to give reliable average values and standard deviations.

8.5 Electrical continuity

General

If cathodic prevention or preparation for such a protective measure is specified, requirements of the electrical continuity within the reinforcement system are given in the European standard for cathodic protection of concrete structures EN 12696 (CEN, 2000). For heavily reinforced concrete structures, there may already be good electrical continuity without any additional precautions, but very often, both welding or special electrical connections are needed between various parts of the reinforcement system. For each step of the concrete construction work, the specified electrical continuity of the reinforcement must be controlled. Such control is often carried out and quality assured by professional people from the established business of cathodic protection.

Figure 8.13 Equipment for control of electrical continuity within the reinforcement system (source: Protector (2008)).

Testing procedure

In principle, electrical continuity is controlled by measurements of the ohmic resistance between various parts of the reinforcement system. In order to avoid the uncertainty of measurements when a traditional multimeter is applied, the measurements should preferably be carried out by the use of a relatively high current (1 A) between the various parts of the reinforcement. Commercial equipment for such measurements is available, where both the ohmic resistance and the rest voltage between the measuring parts 0.1 seconds after interruption of current are observed (Figure 8.13).

9 Achieved construction quality

9.1 General

From the performance-based concrete quality control described in Chapter 8, average values and standard deviation of both chloride diffusivity and concrete cover are obtained. Upon completion of the concrete construction, therefore, these data are used as input parameters to a new durability analysis which provides the documentation of compliance with the specified durability (Chapter 6).

Since the specified chloride diffusivity is only based on the testing of small, separately produced concrete specimens water cured in the laboratory for 28 days, such a chloride diffusivity may be quite different from that achieved on the construction site. During concrete construction, therefore, some additional documentation on the achieved chloride diffusivity on the construction site must also be provided. At the end of concrete construction, such a chloride diffusivity in combination with the achieved concrete cover provide the basis for the documentation of achieved durability on the construction site during concrete construction.

Since neither the 28-day chloride diffusivity from small concrete specimens nor the achieved chloride diffusivity on the construction site during concrete construction reflects the potential chloride diffusivity of the given concrete, further documentation on the long-term chloride diffusivity of the given concrete must also be provided. Such chloride diffusivity in combination with the achieved concrete cover provide the basis for documentation of the potential durability of the given concrete structure.

For the owner of the structure, proper documentation of achieved construction quality may have implications both for the future operation and expected service life of the concrete structure. In the following, therefore, some procedures for providing such documentation are briefly outlined and discussed.

9.2 Compliance with specified durability

As a result of the durability design, an overall durability requirement based on a required service period with a probability for corrosion of less than 10 per cent has been specified (Chapter 6). In order to show compliance with such a durability requirement, a new durability analysis has to be carried out based on the average values and standard deviations of both the chloride diffusivity and the concrete cover obtained from quality control during concrete construction. Although it may have been difficult to select proper data for several of the input parameters to the original durability analysis, these input parameters are now the same for the new durability analysis. Therefore, the new durability analysis primarily reflects the achieved values for chloride diffusivity and concrete cover during concrete construction, including the observed scatter and variability. Hence, the new durability analysis provides a basis for documentation of compliance with the specified durability.

9.3 Durability on the construction site

In principle, any documentation of achieved chloride diffusivity on the construction site should be based on the testing of a number of concrete cores removed from the concrete structure under construction. In order not to weaken the structure, however, one or more un-reinforced concrete elements should be separately produced on the construction site, from which much of the concrete coring can take place during the construction period. In addition, some coring from the real concrete structure must also be carried out, but only from locations where the coring will not weaken the concrete structure.

All separately produced concrete elements, which may be either a wall or slab type of element or both, should be produced and cured as representative as possible of the real concrete structure or various parts of the concrete structure. From these elements, which are produced at an early stage of the concrete construction work, a number of Ø 100 mm concrete cores are removed later on at various ages, and immediately upon removal are sent to the laboratory for testing of achieved chloride diffusivity. In order to obtain a proper curve for the development of chloride diffusivity on the construction site, the cores should be removed and tested after periods of approximately 14, 28, 60, 90, 180 and up to at least 365 days. In addition, supplementary data on achieved chloride diffusivity are also obtained from testing a certain number of concrete cores removed from the real concrete structure during concrete construction.

Depending somewhat on the type of binder system, the obtained development of chloride diffusivity on the construction site often tends to level out after a period of approximately one year. As an example, the observed development of achieved chloride diffusivity from one particular construction site is shown in Figure 9.1. In this figure, the development of chloride diffusivity

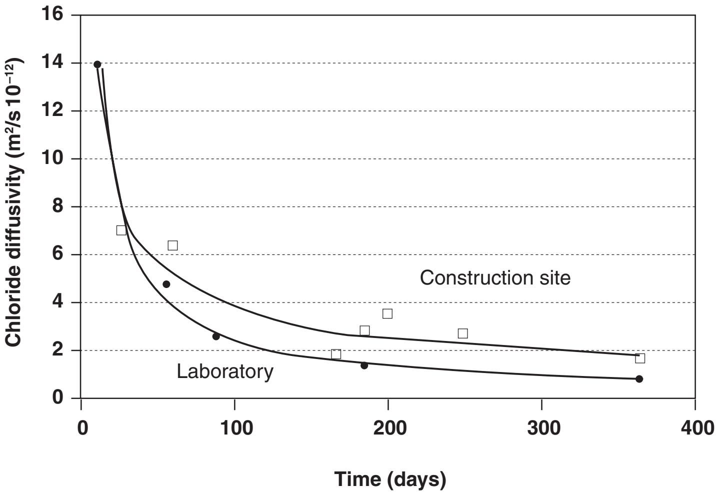


Figure 9.1 Developments of chloride diffusivity on the construction site and in the laboratory during the construction period.

for the same concrete based on separately cast and cured concrete specimens in the laboratory is also shown. Based on the achieved chloride diffusivity on the construction site after approximately one year, a new durability analysis is carried out. In combination with the achieved data on concrete cover, this new durability analysis provides the basis for the documentation of achieved durability on the construction site during concrete construction.

9.4 Potential durability

In order to establish the calibration curve as described in Chapter 8, the chloride diffusivity is determined on separately cast concrete specimens after periods of water curing in the laboratory of approximately 7, 14, 28 and 60 days. Through continued testing of the chloride diffusivity on a few additional specimens after curing periods of approximately 90, 180 and up to at least 365 days, a development of chloride diffusivity (as shown in Figure 9.1) is obtained. Although it may take some time before a final value of the chloride diffusivity is reached, this development curve for most types of concrete tends to level out after approximately one year. Hence, the observed chloride diffusivity after one year of water curing in the laboratory is used as an input to a new durability analysis. In combination with the achieved data on concrete cover, this analysis provides a basis for documentation of the potential durability of the given concrete structure. Also, for this new analysis, the other input parameters are the same as that used in the original durability analysis.

10 Condition assessment and preventive maintenance

10.1 General

For most concrete structures, the typical situation during operation of the structures is that maintenance and repairs are mostly reactive, and the need for taking appropriate measures is usually realized at a very advanced stage of deterioration. For chloride-induced corrosion, repairs at such a stage are then both technically difficult and disproportionately expensive compared to carrying out regular condition assessments and preventive maintenance. Therefore, for all concrete structures where high safety, performance and service life are of special importance, regular condition assessments and preventive maintenance should be carried out.

In recent years, a rapid international development on general systems for Life Cycle Management (LCM) of important infrastructure facilities has taken place (Hudson *et al.*, 1987; Grigg, 1988; O'Connor and Hyman, 1989; BRIME, 1997; RIMES, 1997). Depending on the number of facilities to be included, established LCM systems for both network and project level are commercially available. In many countries, national authorities have also developed their own LCM systems. For many important concrete structures, therefore, general condition assessments and preventive maintenance are already part of the established LCM systems.

For all important concrete structures in chloride-containing environments, however, special procedures for monitoring and control of chloride penetration during operation of the structures are needed. The establishment of such procedures should always be an important and integral part of the durability design (Eri *et al.*, 1998; Tromposch *et al.*, 1998; Gjørv, 2002). In the following, some procedures for regular monitoring and control of chloride penetration as a basis for the preventive maintenance are described and discussed in more detail.

10.2 Control of chloride penetration

Even if the strongest requirements for both concrete quality and concrete cover have been specified and achieved during concrete construction, extensive experience demonstrates that for concrete structures in severe chloride-containing environments a certain rate of chloride penetration will always take place during operation of the structure. As already discussed in previous chapters, the achieved construction quality of concrete structures will always show a high scatter and variability. For concrete construction work in severe marine environments, a certain rate of chloride penetration may also take place during construction before the concrete has gained sufficient curing and maturity (Chapter 2). For concrete structures where this is likely to occur, early control of chloride penetration should always be required before the structure is handed over to the owner. However, early control of chloride penetration should always be carried out in order to provide a reference level for the future control of chloride penetration, and hence a proper basis for evaluation of the future rates of chloride penetration.

For the regular control of chloride penetration during operation of the structure, it is very important to have a detailed plan for the given structure showing the selected locations in which the future control of chloride penetration should take place. These locations, which should be as representative as possible for the most exposed and critical parts of the structure, provide a basis for the assessment of the future rates of chloride penetration. For the plan in question, it should also be important to decide whether future control of chloride penetration should include any automatic readings from embedded probes or only be based on manually observed chloride penetrations.

Already at an early stage of the development of offshore concrete structures for the North Sea, special probes to be embedded in the concrete during concrete construction were developed (Figure 10.1). In combination with other types of probes and instrumentation for remote data acquisition, a general condition assessment of the concrete structure including a control of current chloride penetration rates could then be carried out (Gjørv and Vennesland, 1982). Later on, several new versions of such probes have been developed and are now widely adopted for monitoring chloride penetration in new concrete structures (Figures 10.2 and 10.3). From such probes, not only rates of current chloride penetration can be monitored, but additional information about the local corrosion conditions is also obtained. In particular, such instrumentation may be appropriate for control and condition assessment of special or critical parts of new concrete structures which may not be easily accessible for inspection and manual control later on.

Since embedded probes primarily provide information on how fast the chloride front moves into the concrete, further information about the

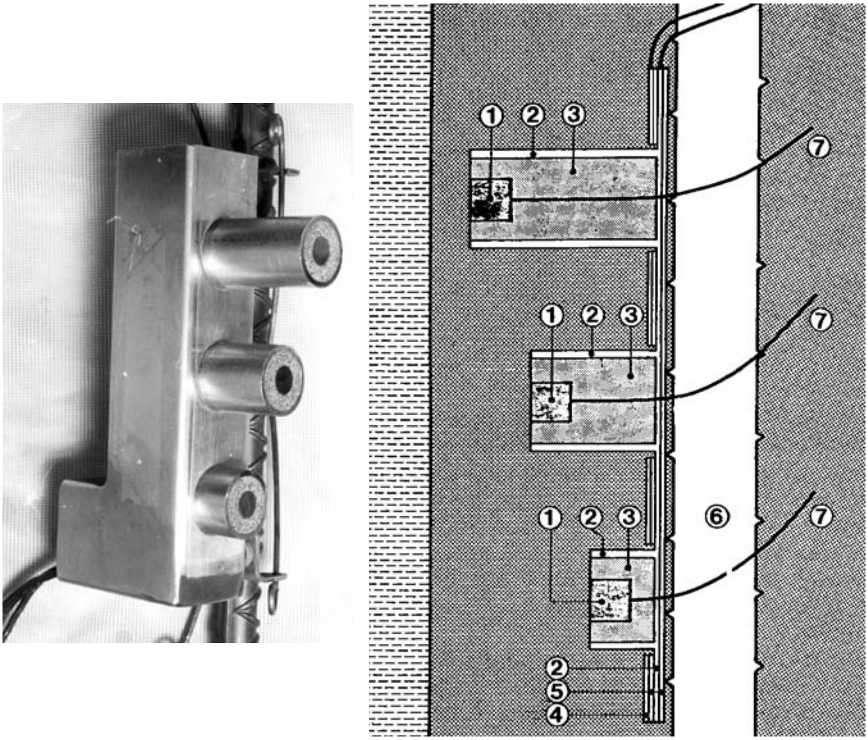


Figure 10.1 Probe for automatic monitoring of chloride penetration into concrete structures: (1) electrode, (2) casing of stainless steel, (3) insulation, (4) reference electrode, (5) insulation, (6) rebar system, (7) shielded leads (source: Gjørv and Vennesland (1982)).

apparent chloride diffusivity (D_a) should also be provided at certain intervals during the service period. In order to establish data on this apparent chloride diffusivity, manual measurements of the ongoing chloride penetration should be carried out. Based on either drilling of dust samples from various depths below the concrete surface or concrete coring, from which dust samples are ground later on, chemical analyses of the chloride contents are carried out and a more complete chloride penetration curve established. Based on a regression analysis of all the observed data on the chloride penetration and a curve fitting to Ficks 2. Law of Diffusion, a chloride penetration curve as shown in Figure 10.4 is obtained. From this regression analysis, information on both the apparent chloride diffusivity (D_a) and the surface chloride concentration (C_s) is obtained. Normally, proper software for this analysis is applied (DURACON, 2004).

If the monitoring of chloride penetration is based on embedded probes, a proper basis is obtained in order to decide how frequently the apparent

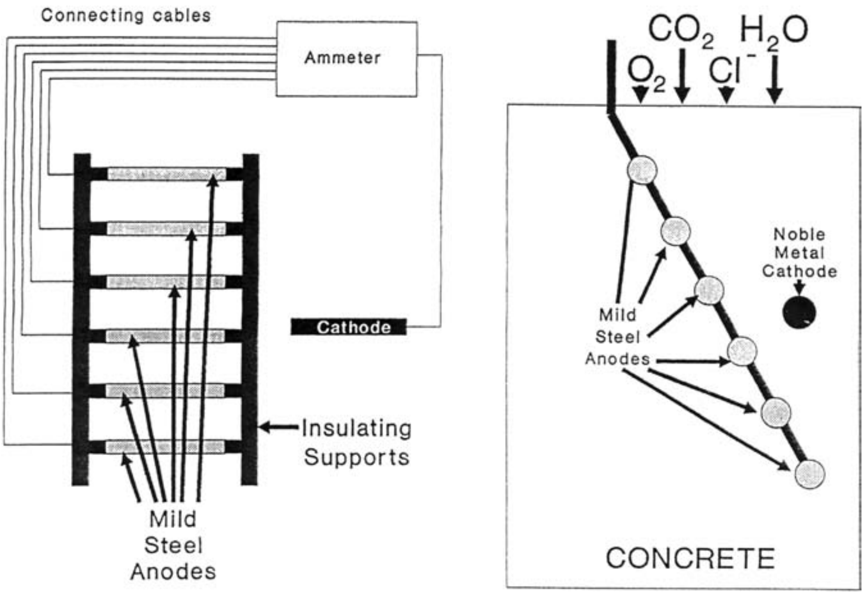


Figure 10.2 Probe for automatic monitoring of chloride penetration into concrete structures: Anode ladder (source: Raupach and Schiessl (1997)).

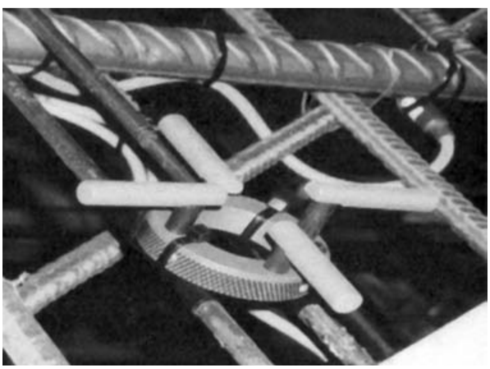


Figure 10.3 Probe for automatic monitoring of chloride penetration into concrete structures: CorroWatch (source: Germann (2008)).

chloride diffusivity should be determined. If the monitoring of the chloride penetration is only based on a manual control, the frequency of control measurements must therefore be based on established experience. Since the rate of chloride penetration is always faster at an early stage of exposure than later on, the control of chloride penetration should generally be

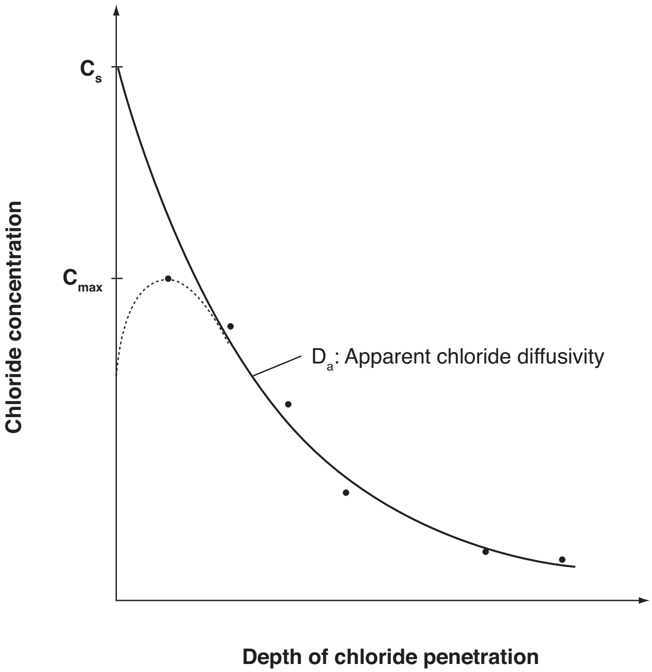


Figure 10.4 The apparent chloride diffusivity (D_a) is the result of a regression analysis of all observed data on chloride penetration at the given time of observation.

carried out more frequently at an early stage of the service period. Based on the reference level obtained shortly after completion of the structure, the next control should normally be carried out after a service period of, for example, five to ten years. Later on, regular control measurements must be carried out, but the frequency of such controls may depend on the observed rate of chloride penetration.

From each control measurement, a new apparent chloride diffusivity (D_a) is obtained, and the more new values for this parameter that become available, a better basis for establishing a more reliable α-value for the time dependence of this apparent chloride diffusivity is obtained. As soon as two or three values for the apparent chloride diffusivity become available, a good basis for establishing a reliable α-value for the given concrete structure in the given environment is obtained. In order to establish this α-value, proper software is normally applied (DURACON, 2004).

Based on the obtained values both for the surface chloride concentration (C_s), the apparent chloride diffusivity (D_a) and the time-dependence factor for the chloride diffusivity (α), a new durability analysis is carried out (Chapter 6). In combination with the previously observed data on concrete cover from the concrete quality control (Chapter 8), this analysis provides a basis for predicting the future probability of corrosion.

10.3 Prediction of corrosion probability

In principle, the prediction of future corrosion probability may be carried out at any early stage of condition assessment for the given concrete structure. For such calculations, however, only α-values based on general experience may then be applied. Nevertheless, such calculations have occasionally been carried out as part of the general condition assessment of several concrete structures (Årskog *et al.*, 2004b; Ferreira *et al.*, 2004; Sengul and Gjørv, 2007). Thus, as part of the condition assessment of the eight-year-old 'Turistskipskaia' (1993) in Trondheim Harbour discussed in Chapter 2 (Figure 2.20), the calculated probability of corrosion for two of the deck beams is shown in Figure 10.5. For these calculations, both the surface chloride concentration (C_s) and the apparent chloride diffusivity (D_a) for the two deck beams in question were obtained on the basis of the observed data on chloride penetration (Table 10.1). As input parameters for the concrete cover, a number of concrete cover measurements were carried out. As input parameters for the time-dependence factor (α), however, only an empirical α-value based on current experience from similar concrete structures in similar environments was selected. For both analyses, it may be seen from Figure 10.5 that the 10 per cent probability of corrosion was exceeded already during a service period of approxi-

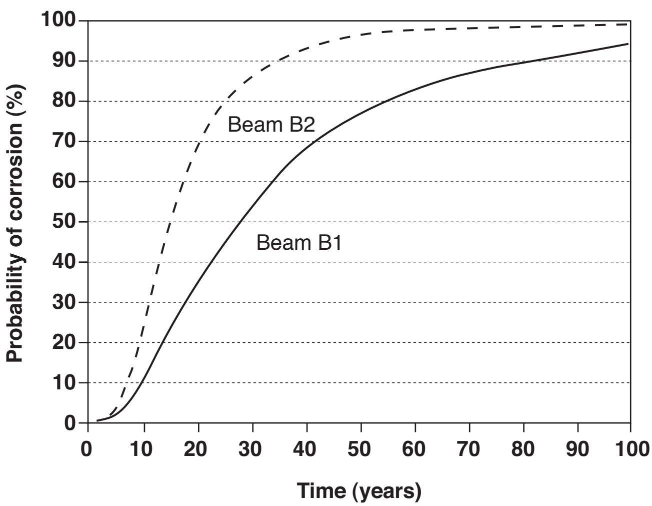


Figure 10.5 Calculated probability of corrosion for two of the deck beams in the eight-year-old 'Turistskipskaia' in Trondheim Harbour (1993) (source: Årskog *et al.* (2004)).

Table 10.1 Input parameters for the durability analyses of the investigated concrete harbour structure

Beam	*B1*	*B2*
Chloride diffusivity, D_a ($\times 10^{-12}$ m^2/s)	N(0.95; 0.17)	N(1.1; 0.27)
Chloride load, C_S (% by weight of cement)	N(3.2; 0.74)	N(4.1; 0.17)
Concrete cover, X_C (mm)	N(50.0; 10.0)	N(48.7; 5.5)
Time-dependence factor (α)	N(0.37; 0.07)	
Critical chloride content, C_{CR} (% by weight of cement)	N(0.40; 0.01)	
Age of concrete, t_0 (years)	8	
Service period (years)	50	

Source: Årskog *et al.* (2004).

mately ten years. For this particular concrete structure, an early stage of corrosion in several of the deck beams had already taken place.

For any calculation of the future corrosion probability based on empirical α-values as shown above, however, it should be noted that the results of such calculations may be very unreliable and should therefore be used with great care. As more new data on the apparent chloride diffusivity (D_a) from the given locations in the given concrete structure with the given exposure conditions become available, however, more reliable values for the time-dependence factor (α) may be obtained, as is discussed above. Eventually, more reliable calculations for the future corrosion probability may also be carried out.

10.4 Protective measures

Before the future probability of corrosion becomes too high, proper protective measures should be considered and selected as discussed in Chapter 7. Depending on the type of protective measure, the observed rate of chloride penetration may either be reduced or completely stopped. If the chlorides have not penetrated too deeply through the concrete cover, a proper surface treatment or coating may slow down the further rate of chloride penetration. If the chlorides have already reached too deep, however, a cathodic prevention is the only protective measure that can stop further chloride penetration and thus avoid the development of steel corrosion.

11 Practical applications

11.1 General

In recent years a number of new important concrete structures in Norwegian marine environments have been built, and for most of them the specified durability has mainly been based on the minimum requirements according to the current concrete codes (Standard Norway, 2003a, 2003b, 2003c). In order to obtain some further information about the quality of the concrete typically applied to all these new concrete structures, samples of the concrete from some of these construction sites were collected in order to test the development of chloride diffusivity based on the RCM method (NORDTEST, 1999). Although all of these types of concrete fulfilled the specified durability requirements for a severe marine environment according to current concrete codes, it may be seen from Table 11.1 that

Table 11.1 Observed chloride diffusivity of the concrete used for some important concrete structures recently constructed in Norwegian marine environments

Construction site	*Chloride diffusivity ($\times 10^{-12}$ m²/s)*									
	Time (days)									
	14	*28*	*60*	*90*	*180*	*365*	*400*	*460*	*620*	*730*
'Nye Filipstadkaia', Oslo (2002)	13.5	6.0	4.4	3.8	3.0	–	–	–	–	–
'Harbour Terminal', Aukra (2005)	17.6	6.8	4.3	2.3	–	–	1.5	–	–	–
'Eiksund Bridge', Eiksund (2005)	14.1	4.4	3.8	3.4	3.1	–	–	3.0	–	–
'New Container Harbour', Oslo (2007)	14.0	6.9	4.6	2.4	1.2	0.7	–	–	–	0.7
'Nye Tjuvholmen', Oslo (2005–)	4.7	1.6	0.4	0.4	0.3	0.2	–	–	0.16	–

the chloride diffusivity or the resistance to chloride penetration of the various types of concrete varied within wide limits. All this testing was carried out on separately cast concrete specimens from the various construction sites and then water cured in the laboratory until the time of testing.

In order to obtain a more controlled durability and service life, some of the above concrete structures were also subjected to a durability design and concrete quality control as described and discussed in the previous chapters. In the following, a brief outline of the experience gained with these practical applications is given. All of these concrete structures have recently been completed in the harbour region of Oslo City. One of the structures was the first part of the new container harbour in Oslo, while the others are part of a new city development type of project currently under construction. As a reference project, the achieved construction quality of a concrete structure in Oslo Harbour ('Nye Filipstadkaia') is also briefly outlined, for which the durability requirements were only based on the current concrete codes.

11.2 'Nye Filipstadkaia', Oslo (2002)

'Nye Filipstadkaia' is a typical Norwegian concrete harbour structure with an open beam-and-slab type of deck on top of driven steel tubes filled with concrete (Figure 2.27). The structure, which has a waterfront of 144 m, was constructed in two steps and completed in 2002. This particular concrete structure was designed before the current procedures for durability design were available. Therefore, the specified durability for the required 100-year service life was only based on the use of a concrete quality of type 'C 45 MA' according to the current Norwegian concrete codes (Standard Norway, 1986, 1989), with the following additional requirements:

- $W/(C + k{\cdot}S)$: ≤0.40 ± 0.03
- minimum cement content (C): 370 kg/m^3
- silica fume (S): 6–8 per cent by C
- air content: 5.0 ± 1.5 per cent.

For the above water/binder ratio, k is an empirical 'efficiency factor' of 2 for the use of silica fume. A nominal concrete cover to the structural steel of 75 ± 15 mm was also specified (Standard Norway, 1989).

Although no probability-based durability design was carried out, Oslo Harbour KF as the owner of the structure wanted to obtain the best possible documentation of achieved construction quality and durability during the construction period. Therefore, shortly after the concrete construction started, a type of concrete quality control similar to that described in Chapters 8 and 9 was carried out.

Achieved durability

In order to provide the best possible basis for the testing and evaluation of the given concrete, a wall type of concrete element with dimensions of 1.0 × 0.3 × 2.0 m was separately produced on the construction site at an early stage of the concrete construction work. From this concrete element which was produced and cured as representative as possible for the real concrete structure, a number of Ø 100 mm concrete cores were later removed at various ages during the construction period, and immediately upon removal, sent to the laboratory for testing of achieved chloride diffusivity according to the RCM method. In addition, a number of concrete cores were also removed from the real concrete structure where possible without weakening the structure. Altogether, this testing included concrete cores from 13 different stages of the concrete construction period.

From the same concrete batch which was applied for production of the above concrete element, a number of Ø 100 × 200 mm concrete cylinders were also produced on the construction site. The following day, these specimens were also sent to the laboratory, where they were kept in water until the time of testing. This testing was carried out in order to determine the development of chloride diffusivity under more controlled conditions in the laboratory.

Based on the above measurements, the development of achieved chloride diffusivity both on the construction site and in the laboratory for a period of up to approximately six months is shown in Figure 11.1. During

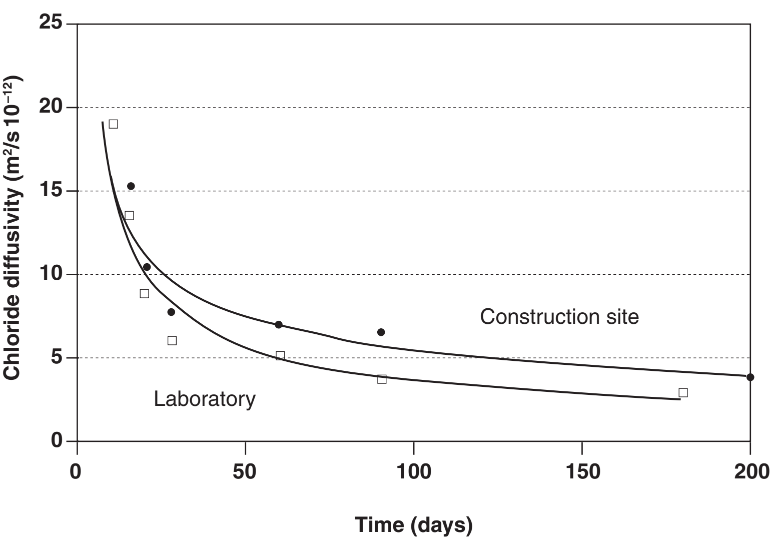


Figure 11.1 Development of chloride diffusivity on the construction site and in the laboratory.

the construction period, a regular control of the achieved concrete cover was also carried out, and based on a total of 153 individual control measurements, an average of 65 mm with a standard deviation of 7 mm was obtained.

Based on the separately cast and water-cured concrete specimens in the laboratory, a 28-day chloride diffusivity (D_{28}) of $6.0 \times 10^{-12}\,m^2/s$ was obtained. Combined with the achieved concrete cover from the quality control, a durability analysis was carried out in order to get an indication of achieved service period before a 10 per cent probability of corrosion would be reached. All the necessary input parameters to this analysis are shown in Table 11.2. The chloride load (C_S) was based on long-term experience from existing concrete structures in Oslo Harbour after service periods of up to 80 years. Since the structure in question was severely exposed to high tides already at an early stage of the construction period before the concrete had gained sufficient maturity and density (Figure 2.28), a time for chloride exposure (t') of 28 days was selected. As part of the environmental loading, an annual average temperature (T) of 10°C was also selected. Since the given concrete was based on a portland cement of type CEM I 52.5 LA in combination with 6 per cent silica fume by weight of cement, a time-dependence factor for the chloride diffusivity (α) of 0.40 was adopted. A critical chloride content (C_{CR}) of 0.40 per cent by weight of binder was also selected. As may be seen from Figure 11.2, a service period of approximately 30 years was achieved before a 10 per cent probability of corrosion would be reached.

Durability on the construction site

Based on an achieved value for chloride diffusivity after 200 days on the construction site of $3.9 \times 10^{-12}\,m^2/s$ and the achieved concrete cover from quality control, a new durability analysis was carried out. With all the other input parameters kept the same as that shown in Table 11.2, the new

Table 11.2 Input parameters for the analysis of achieved durability

Input parameter	
Chloride load, C_S (% by weight of binder)	N[1](3.8; 0.9)
Age of concrete at exposure, t' (days)	28
Temperature, T (°C)	10
Chloride diffusivity, D_{28} ($\times 10^{-12}\,m^2/s$)	N(6.0; 0.6)
Age of concrete at testing, t (days)	28
Time-dependence factor, α	N(0.40; 0.08)
Critical chloride content, C_{CR} (% by weight of binder)	N(0.40; 0.08)
Concrete cover, X (mm)	N(65; 7)
Service period (years)	100

Note
1 Normal distribution with average value and standard deviation.

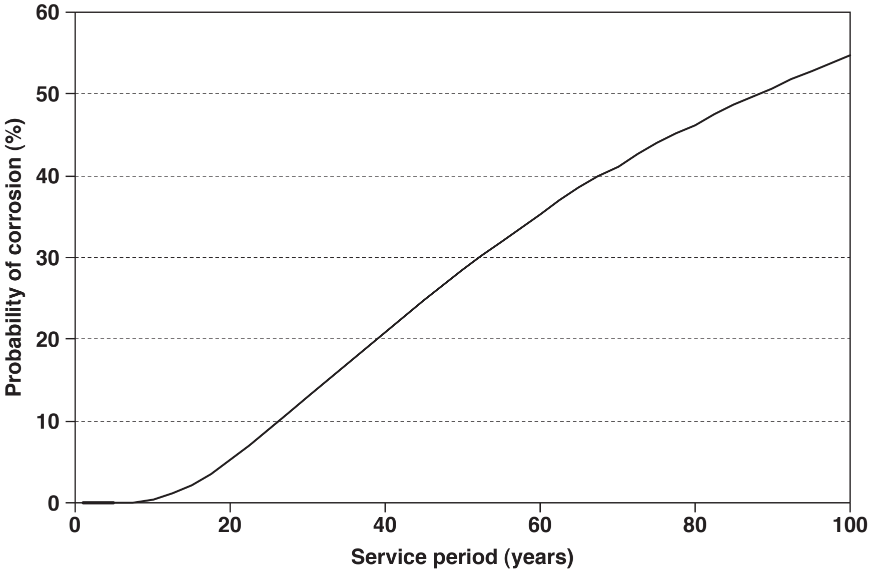


Figure 11.2 Probability of corrosion versus time for 'Nye Filipstadkaia' (2002) in Oslo Harbour.

analysis showed that a probability for corrosion of approximately 80 per cent during a service period of 100 years would be reached.

Potential durability

Based on the water-cured concrete specimens in the laboratory for a period of up to 180 days, a chloride diffusivity of $3.0 \times 10^{-12}\,m^2/s$ was obtained. Combined with the achieved data on concrete cover, a final durability analysis was carried out in order to provide a documentation of the potential durability of the structure. With all the other input parameters as shown in Table 11.2 kept constant, the new analysis showed that a probability of corrosion of approximately 60 per cent during a service period of 100 years would be reached.

Based on the above results for achieved construction quality and durability, current experience indicates that a relatively high level of maintenance should be expected in order to maintain a proper serviceability of the given structure during a service period of 100 years.

11.3 'New Container Harbour', Oslo (2007)

During the period from January 2005 to June 2007, the first part of the new container harbour on Sjursøya in Oslo was constructed. This structure has a waterfront of 650 m and consists of an open concrete deck on top of solid columns consisting of driven steel tubes filled with concrete. The major part of the 22,900 m^2 concrete deck is a beam-and-slab type of deck with deep girders. For this part of the deck, a large number of concrete slab elements were prefabricated and installed in between the in-situ cast concrete girders as a formwork for the later concreting of the deck. The rest of the concrete deck was a flat-slab type of deck constructed on top of an old existing concrete harbour structure, where the old deck was only used as a formwork for the new deck.

According to the current Norwegian concrete codes (Standard Norway, 2003a, 2003b), the durability requirements for a 100-year service life primarily included a maximum water/binder ratio of 0.40, a minimum binder content of 330 kg/m^3 and a minimum concrete cover of 60 mm. For ensuring proper frost resistance, a total air content of 4 to 6 per cent was also specified. In order to obtain increased and more controlled safety against steel corrosion, however, Oslo Harbour KF as the owner of the structure decided to apply the current procedures both for durability design and concrete quality control as described in previous chapters. At that time, these procedures had just been adopted by The Norwegian Association for Harbour Engineers as recommendations and guidelines for durability design and concrete quality control of new concrete structures in Norwegian harbours.

Specified durability

At an early stage of the design, the owner decided that the new structure should have a service period of at least 100 years before 10 per cent probability of corrosion would be reached. In order to satisfy such a durability requirement, an initial durability analysis with input parameters as shown in Table 11.3 was carried out. This provided the basis for establishing the necessary requirements for concrete quality and concrete cover.

Table 11.3 Input parameters for the initial durability analysis

Input parameter	
Chloride load, C_S (% by weight of binder)	N[1](3.8; 0.9)
Age of concrete at exposure, t' (days)	28
Temperature, T (°C)	10
Chloride diffusivity, D_{28} ($\times 10^{-12}$ m^2/s)	N(5.0; 1.0)
Age of concrete at testing, t (days)	28
Time-dependence factor, α	N(0.60; 0.12)
Critical chloride content, C_{CR} (% by weight of binder)	N(0.40; 0.08)
Concrete cover, X (mm)	N(90; 11)
Service period (years)	100

Note
1 Normal distribution with average value and standard deviation.

With a location very close to the previous structure 'Nye Filipstadkaia', the same data for environmental loads as those selected for this structure were adopted. Since previous experience had also shown that an early age exposure during the construction period could take place, a time for chloride exposure (t') of 28 days was also adopted. Based on current experience, a concrete quality with a 28-day chloride diffusivity of $5.0 \times 10^{-12}\,m^2/s$ with a time-dependence factor (α) of 0.60 and a critical chloride content (C_{CR}) of 0.40 per cent were selected.

Based on the above durability analysis, it was shown that a chloride diffusivity of $D_{28} = 5.0 \times 10^{-12}\,m^2/s$ in combination with a concrete cover of 90 ± 15 mm would satisfy the overall durability requirement with a proper margin. These results, therefore, were selected as a basis for specifying the necessary requirements for chloride diffusivity and concrete cover.

In order to establish a concrete mixture which would meet the specified chloride diffusivity, some preliminary tests were carried out by the contractor. Based on a pure portland cement of type CEM I 52.5 LA and 4 per cent silica fume by weight of cement, three different concrete mixtures with 20, 40 and 60 per cent replacements of the portland cement by a low calcium-containing fly ash (FA) were produced. Based on these tests, the mixture based on 60 per cent FA was selected by the contractor. Although this type of concrete showed a somewhat higher 28-day value of the chloride diffusivity (D_{28}) than that specified, the further development of the chloride diffusivity showed very good results. Since the documentation of frost resistance of this concrete also was very good, this type of concrete was accepted for the further production of concrete.

Compliance with specified durability

Shortly after the start of the concrete construction work, a solid concrete test slab with dimensions of 2.0 × 2.0 × 0.5 m without any reinforcement was produced on the construction site. From this test slab, which was produced and cured as representative as possible for the real concrete structure, a number of Ø 100 mm concrete cores were later removed at various ages in order to observe the development of chloride diffusivity on the construction site.

From the same concrete batch as that used for the production of the above test slab, a number of 100 mm concrete cubes and Ø 100 × 200 mm concrete cylinders were also produced. The following day, these test specimens were sent to the laboratory for the testing and establishment of the necessary calibration curve. This calibration curve would later be needed for the regular concrete quality control of the chloride diffusivity (D_{28}) based on electrical resistivity measurements during concrete construction (Chapter 8). All electrical resistivity measurements were carried out by the use of the Wenner method, while the parallel testing of the chloride diffusivity was based on the RCM method. With parallel measurements of chloride diffusivity and electrical resistivity after curing periods of approximately 14, 28, 56 and 90 days (Table 11.4), a calibration curve as shown in Figure 11.3 was established. This calibration curve was later used for the indirect quality control of chloride diffusivity (D_{28}) carried out on all concrete cubes for testing the 28-day compressive strength during concrete construction.

During the whole period of concrete construction, a total of 344 individual measurements of electrical resistivity were carried out, and this quality control showed an acceptable level of chloride diffusivity throughout the concrete construction without large deviations. As a result, an average value of $7.9 \times 10^{-12}\,m^2/s$ for chloride diffusivity with a standard deviation of 3.2 was obtained (Table 11.5).

Throughout the period of concrete construction, a number of control measurements of achieved concrete cover were also carried out. Due to the

Table 11.4 Test results for establishing the necessary calibration curve

Testing age (days)	*Chloride diffusivity*[1] *($\times 10^{-12}\,m^2/s$)*	*Electrical resistivity*[1] *(ohm m)*
14	14.0;1.9	146;9
28	6.9;0.1	307;21
56	4.6;0.6	508;41
90	2.4;0.5	699;131

Note
1 Average value and standard deviation.

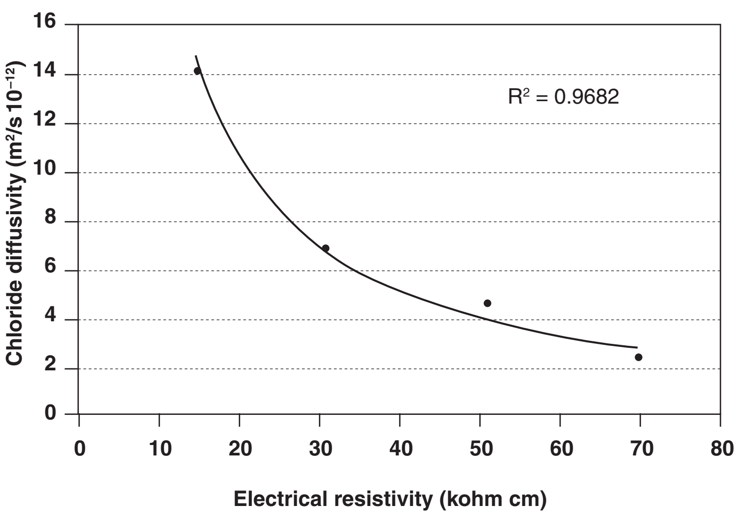


Figure 11.3 Calibration curve for quality control of chloride diffusivity based on electrical resistivity measurements.

Table 11.5 Obtained chloride diffusivity (D_{28}) based on electrical resistivity measurements during concrete construction

Testing age (days)	*Electrical resistivity*[1] *(ohm m)*	*Chloride diffusivity*[1] *($\times 10^{-12}$ m^2/s)*
28	260;102	7.9;3.2

Note
1 Average value and standard deviation.

very thick concrete cover and congested reinforcement as shown in Figures 11.4 and 11.5, it was not easy to obtain reliable measurements based on a conventional cover meter. Therefore, most of the control measurements were based on manually observed concrete covers on protruding bars in concrete joints during the construction period. Based on a total of 68 individual measurements, an average concrete cover of 99 mm with a standard deviation of 11 mm was obtained.

Figure 11.4 Protruding bars in the joints of the concrete deck.

Figure 11.5 The concrete deck had very congested reinforcement.

Upon completion of the concrete construction work, a new durability analysis based on the input parameters, as shown in Table 11.6, was carried out. An obtained probability of approximately 5 per cent for corrosion over a 100-year service period showed that the specified durability had been achieved with a good margin.

Table 11.6 Input parameters for the control of compliance with the specified durability

Input parameter	
Chloride loading, C_S (% by weight of binder)	N[1](3.8; 0.9)
Age of concrete at exposure, t' (days)	28
Temperature, T (°C)	10
Chloride diffusivity, D_{28} ($\times 10^{-12}\,m^2/s$)	N(7.9; 3.2)
Age of concrete at testing, t (days)	28
Time-dependence factor, α	N(0.60; 0.12)
Critical chloride content, C_{CR} (% by weight of binder)	N(0.40; 0.08)
Concrete cover, X (mm)	N(99; 11)
Service period (years)	100

Note
1 Normal distribution with average value and standard deviation.

Durability on the construction site

In order to provide some additional documentation of achieved durability on the construction site, a number of Ø 100 mm concrete cores were removed from the test slab on the construction site. These cores were removed at various ages of up to approximately one year, and immediately upon removal the cores were sent to the laboratory for testing of achieved chloride diffusivity. In addition, achieved chloride diffusivity was also tested on a number of concrete cores removed from the real concrete structure during the construction period. All these test results are plotted in Figure 11.6, where the development of chloride diffusivity based on the water-cured specimens in the laboratory is also shown.

Based on the achieved chloride diffusivity on the construction site after one year of $1.5 \times 10^{-12}\,m^2/s$ with a standard deviation of 0.54 in combination with the achieved concrete cover, a new durability analysis was carried out. As a result, a probability of corrosion of 0.6 per cent after a service period of 100 years was obtained. Such a result indicates that the achieved durability on the construction site was very good.

Potential durability

Based on the achieved chloride diffusivity after one year of water-cured specimens in the laboratory in combination with the achieved concrete cover, a further durability analysis was carried out. For a chloride diffusivity of $0.7 \times 10^{-12}\,m^2/s$ with standard deviation of 0.02, this durability

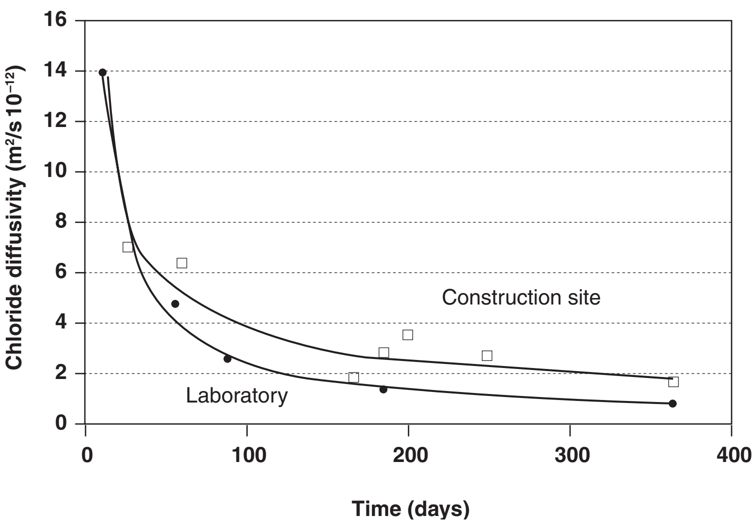


Figure 11.6 Development of chloride diffusivity on the construction site and in the laboratory.

analysis showed a probability for corrosion of 0.01 per cent after a service period of 100 years. This result indicates that the potential durability of the given concrete structure was also very good.

11.4 'Nye Tjuvholmen', Oslo (2005–)

In 2005, a new city development project ('Nye Tjuvholmen') was started in the Oslo harbour region. This project includes a number of concrete substructures located in various water depths of up to 20 m, on top of which a number of business and apartment buildings are being constructed, as is shown in Figure 11.7. Most of the concrete substructures which are still under construction include large submerged parking areas. Most of the substructures in the more shallow water include a solid concrete bottom slab on the sea bed surrounded by concrete walls partly protected by stone fillings and partly freely exposed to the tidal zone. In the deeper water, some of the structures include an open concrete deck on top of solid columns consisting of driven steel pipes filled with concrete. For the deepest water, three large concrete caissons are currently under construction in a dry dock of a shipyard nearby and will upon completion be moved into position and submerged in water depths of up to 20 m. These prefabricated concrete units will provide large submerged parking areas in four levels (Figure 11.8).

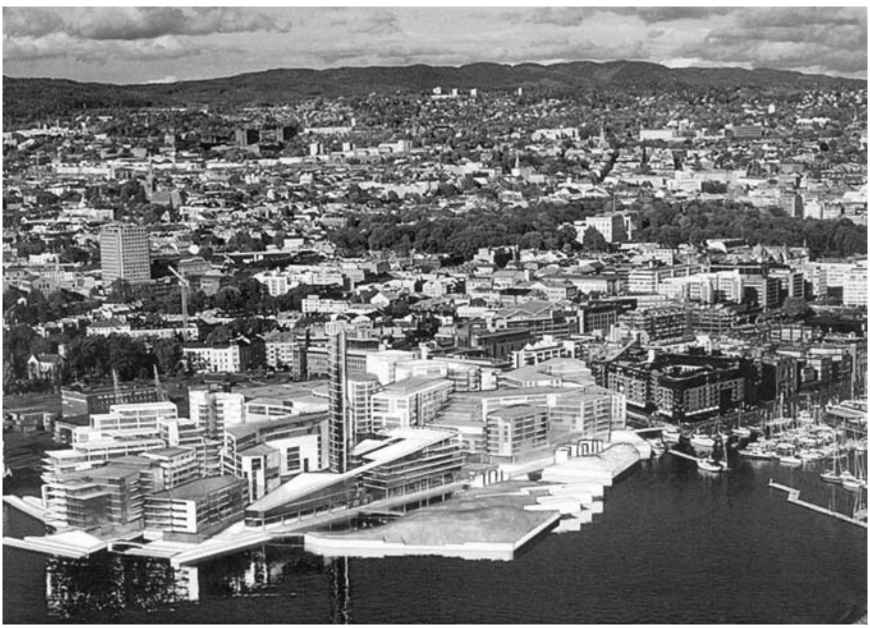


Figure 11.7 A new city development currently under construction on Tjuvholmen in the harbour region of Oslo City (source: Courtesy of Tjuvholmen KS).

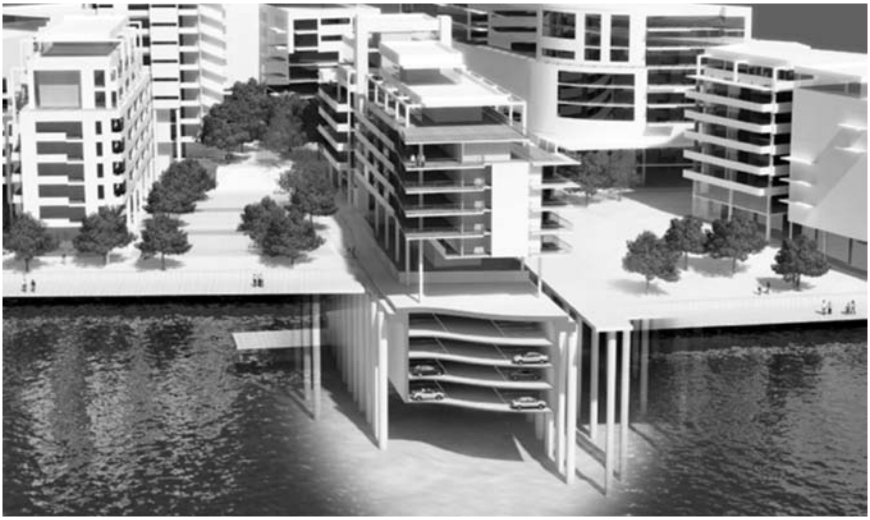


Figure 11.8 A model section showing how the prefabricated concrete caissons after installation will provide large submerged parking areas in four levels (source: Courtesy of Skanska).

At an early stage of planning, Tjuvholmen KS as the developer of the project would like to achieve the best possible defence against the corrosion of embedded steel for all the concrete substructures in the project. These concrete substructures should later on form the basis for a large number of buildings representing huge investments. Based on the new recommendations and guidelines for durability design which had shortly before been published by The Norwegian Association for Harbour Engineers, the developer would like to specify a service period of 300 years.

Specified durability

As discussed in Chapter 6, the current basis for calculating a probability of corrosion for a service period of more than 150 years is neither considered valid nor relevant. As a basis for the durability design, therefore, the durability analyses were carried out in order to obtain a combination of concrete quality and concrete cover which would give a probability of corrosion as low as possible and not exceeding 10 per cent within a service period of 150 years. In order to further ensure a proper long-term performance of the given concrete structures, some additional strategies and protective measures as discussed in Chapter 7 would be needed.

As is shown in Chapter 3, a concrete based on blast furnace slag cement in combination with silica fume would give a very low chloride diffusivity. Based on current experience, therefore, a value for the 28-day chloride diffusivity (D_{28}) of $2.0 \times 10^{-12}\,m^2/s$ was selected as a basis for the initial durability analysis. For this analysis, a nominal concrete cover of 100 ± 10 mm was also selected, while all the other necessary input parameters were based on current experience, as shown in Table 11.7. As a result, a probability for corrosion of less than 0.3 per cent after a service period of 150 years for the most exposed parts of the structures would be obtained. Therefore, the above values both for the 28-day chloride diffusivity and the concrete cover were specified. In order to reduce the risk of cracking for a nominal concrete cover of 100 ± 10 mm, a proper dosage of synthetic fibres to the concrete was also specified.

As an additional protective measure for all exposed walls of the first concrete structure, preparation for later installation of cathodic prevention in combination with embedded instrumentation for chloride monitoring was specified. For the solid concrete slab on the sea bed, however, any additional protective measure was not considered necessary due to the very low oxygen availability for this part of the structure.

For some of the concrete structures produced later, other combinations of various types of concrete and special protective measures were also applied. As experience was gained, however, a combination of the above type of concrete based on blast furnace slag cement and silica fume in combination with stainless steel was soon adopted as a very good and robust technical solution. Even on a short-term basis, this solution also proved to be competitive from an economic point of view. In the following, therefore, some of the obtained results from the application of the above type of concrete in combination with stainless steel are briefly outlined.

For all exposed concrete walls, a stainless steel of type W.1.4301 or an equivalent steel quality was specified (Chapter 7). For all exposed walls, replacement of the black steel with stainless steel in the outer layer of the rebar system would increase the effective concrete cover to the black steel further in to more than 150 mm. Therefore, the specified nominal concrete

Table 11.7 Input parameters for the initial durability analysis

Input parameter	
Chloride load, C_S (% by weight of binder)	N[1](3.8; 0.9)
Age of concrete at exposure, t' (days)	28
Temperature, T (°C)	10
Chloride diffusivity, D_{28} ($\times 10^{-12}\,m^2/s$)	N(2.0; 0.4)
Age of concrete at testing, t (days)	28
Time-dependence factor, α	N(0.50; 0.1)
Critical chloride content, C_{CR} (% by weight of binder)	N(0.40; 0.08)
Concrete cover, X (mm)	N(100; 7)
Service period (years)	150

Note

1 Normal distribution with average value and standard deviation.

cover to the stainless steel was reduced to 85 ± 10 mm. Hence, any addition of fibres to the concrete for these concrete walls was no longer needed. For all the solid bottom slabs, however, both black steel with a nominal concrete cover of 100 ± 10 mm and a concrete with synthetic fibres were still specified.

Compliance with specified durability

In order to meet the specified chloride diffusivity of $D_{28} = 2.0 \times 10^{-12}\,m^2/s$, several trial mixtures were produced by the contractor. Finally, a concrete mixture based on a blast furnace slag cement of type CEM III/B 42.5 LH HS and 10 per cent silica fume by weight of cement similar to the 'GGBS2' type of concrete shown in Figure 3.3 was established. As a result, a 28-day value for the chloride diffusivity of $1.6 \times 10^{-12}\,m^2/s$ was obtained (Table 13.1), while the addition of fibres did not much affect the observed chloride diffusivity. Since a very good frost resistance was also obtained (Chapter 4), this type of concrete both with and without synthetic fibres was accepted for further concrete production.

For each new concrete structure, two new test elements were produced on the construction site at an early stage of concrete construction. These elements included both a wall element and a slab element produced and cured as representative as possible for the real concrete structure (Figures 11.9, 11.10(a) and 11.10(b)). From these elements, a number of Ø 100 mm

Figure 11.9 Production of a test wall element on the construction site.

(a)

(b)

Figures 11.10(a) and (b) Production of a test slab element on the construction site.

concrete cores were later removed at various ages in order to establish the development of chloride diffusivity on the construction site.

From the same concrete batch as that used for production of the test elements, a number of 100 mm concrete cubes and Ø 100 × 200 mm concrete cylinders were also produced. The following day, these test specimens were sent to the laboratory for establishing the necessary calibration curve, which was needed later for regular quality control of the chloride diffusivity based on the electrical resistivity measurements. While all chloride diffusivity measurements were based on the RCM method, all electrical resistivity measurements were based on the Wenner method. Based on the combined testing of chloride diffusivity and electrical resistivity after curing periods of approximately 14, 28, 56 and 90 days, a calibration curve as typically shown in Figure 11.11 was established.

Based on the measurements of electrical resistivity and the above calibration curve, regular quality control of the 28-day chloride diffusivity was carried out on all concrete cubes used for testing the 28-day compressive strength. During the construction period, regular control measurements of the achieved concrete cover were also carried out. Due partly to the thick concrete covers and partly to the use of stainless steel which does

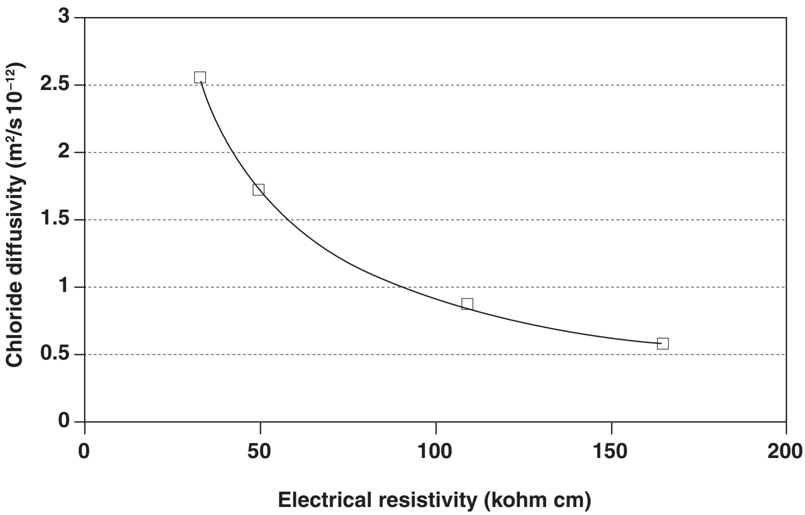


Figure 11.11 Calibration curve for control of chloride diffusivity based on electrical resistivity measurements.

not respond to magnetic measurements, all control measurements of the concrete cover were carried out manually on protruding steel in the various construction joints during concrete construction.

For the concrete used in the exposed walls, a typical 28-day chloride diffusivity of $2.1 \times 10^{-12}\,m^2/s$ with a standard deviation of 0.4 was obtained. A typical value for the achieved concrete cover of 83 mm with a standard deviation of 6 mm was also obtained. Together with a conservative value for the critical chloride content of 2 per cent by weight of binder for the stainless steel and the same input parameters as previously shown in Table 11.7, a probability of less than 0.1 per cent for corrosion after a service period of 150 years was obtained. Thus, the specified durability was achieved with a good margin.

Durability on the construction site

For each concrete structure, a further documentation of achieved construction quality on the construction site was also provided. This documentation was based on a diffusivity testing of all the Ø 100 mm concrete cores received from the construction site, partly from the test slabs and partly from the real concrete structures. Based on all this testing, a development of chloride diffusivity on the construction site as shown in Figure 11.12 was typically obtained. This figure also shows the development of chloride diffusivity based on water-cured specimens in the laboratory.

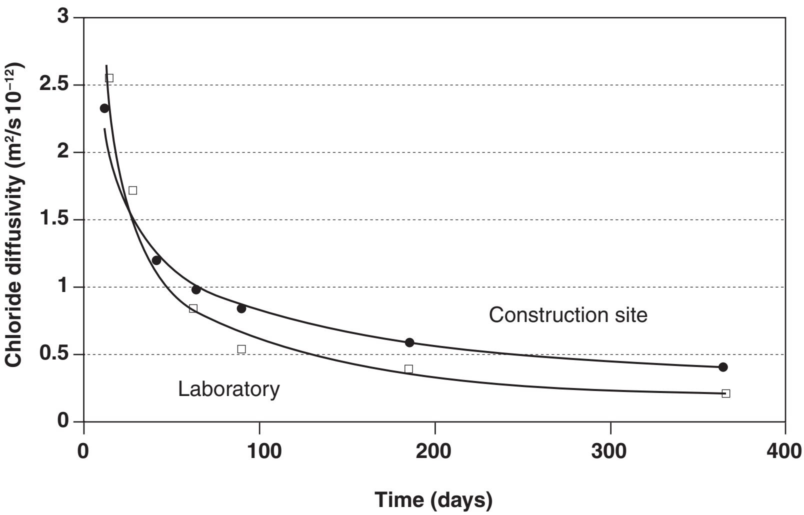


Figure 11.12 Development of chloride diffusivity on the construction site and in the laboratory.

Based on the achieved chloride diffusivity on the construction site after approximately one year of $0.45 \times 10^{-12}\,m^2/s$ with a standard deviation of 0.12 and the same input parameters as previously used for the analysis of the durability compliance, the new durability analysis showed a probability for corrosion of less than 0.001 per cent after 150 years. Based on current experience, such a result indicates that the achieved durability on the construction site was extremely good.

Potential durability

Figure 11.12 also shows the development of chloride diffusivity for the given concrete based on water-cured concrete specimens in the laboratory. With an obtained chloride diffusivity after approximately one year of $0.2 \times 10^{-12}\,m^2/s$ with a standard deviation of 0.06, a new durability analysis was carried out. With all the other input parameters from the previous analyses kept constant, the new analysis also showed a probability for corrosion after 150 years of less than 0.001 per cent. Based on current experience, this result indicates that the potential durability of the concrete structures was also extremely good.

11.5 Evaluation and discussion of obtained results

For a more general evaluation and interpretation of the results obtained, reference is made to the previous discussion in section 6.7 of Chapter 6. As already emphasized here, the above 'service periods' with a given probability of corrosion should not be considered as real service periods for the given concrete structures. However, the durability analyses are primarily carried out in order to provide the basis for an engineering judgement of the most important factors which are considered relevant for the durability, including the scatter and variability of all factors involved. Hence, a proper basis for comparing and selecting one of several technical solutions in order to obtain a best possible durability is obtained. As a result, a type of durability requirement is specified which can also be verified and controlled in such a way that documentation of compliance with the specified durability can be provided. In addition, a certain documentation of achieved construction quality can also be provided. On all the above construction sites, it was also observed that the increased focus and attention to quality control in itself had a beneficial effect on the workmanship and hence, the achieved construction quality.

For the first concrete structure ('Nye Filipstadkaia') which was based only on durability requirements according to the current concrete codes, a service period of approximately 30 years was obtained before a 10 per cent probability of corrosion would be reached. Based on the achieved data from the construction site and from the obtained chloride diffusivity in the laboratory after approximately 180 days, probabilities of about 80 per cent and 60 per cent for corrosion to occur after a 100-year service period were also obtained, respectively.

For the second concrete structure ('New Container Harbour'), an overall durability requirement of at least a 100-year service period had been specified before a 10 per cent probability of corrosion would be reached. Upon completion of this concrete structure, a probability for corrosion of approximately 5 per cent after a service period of 100 years was obtained, showing that the specified durability had been achieved with a proper margin. Based on the achieved data from the construction site and from the obtained chloride diffusivity in the laboratory after approximately one year, probabilities of 0.6 per cent and 0.02 per cent for corrosion to occur after a 100-year service period were also obtained, respectively. These results indicate that both the achieved durability on the construction site and the potential durability of the structure were very good.

For all the concrete structures at 'Nye Tjuvholmen', an overall durability requirement of a 300-year service period was specified. Since the current basis for the calculation of corrosion probability after such a long service period is not considered valid, a probability of corrosion as low as possible and not exceeding 10 per cent within a service period of 150 years was specified. In order to further ensure a proper long-term performance of the given concrete structures, a partial use of stainless steel reinforcement was also typically specified.

Upon completion of the concrete structures at ‘Nye Tjuvholmen’, a probability for corrosion of less than 0.1 per cent after a service period of 150 years was typically obtained, showing that the specified durability had been achieved with a very good margin. Based on the achieved data both from the construction site and from the obtained chloride diffusivity in the laboratory after approximately one year, probabilities of less than 0.001 per cent for corrosion to occur after a 150-year service period were also obtained. These results indicate that both the achieved durability on the construction site and the potential durability of the concrete structures were extremely good. The use of stainless steel proved to be a very simple and robust technical solution for ensuring the specified durability. Even on a short-term basis, this solution also proved to be competitive from an economic point of view.

For all the above concrete structures, a service manual for proper condition assessments and preventive maintenance of the structures was also an important part of the durability design (Chapter 10). During operation of the concrete structures, it is only a monitoring of the real chloride penetration and evaluation of the future corrosion probability in combination with proper protective measures that provide the ultimate basis for achieving a more controlled durability and service life of the concrete structures.

For the durability design of new concrete structures as outlined and discussed above, the objective was primarily to compare and select one of several technical solutions in order to obtain the best possible durability. However, such a durability design is not complete without also taking into account the various costs and environmental impacts of the different technical solutions. These aspects are further outlined and discussed in Chapters 12 and 13, respectively.

12 Life cycle costs

12.1 General

Calculations or estimations of costs against benefits can be carried out in different ways by considering various types of cost or benefit, and this is often referred to in terms of 'whole life cycle costing', 'cost–benefit analysis' or 'cost–benefit–risk analysis'. Life cycle costs (LCC) may be used as a valuable tool for the assessment of the 'cost-effectiveness' of various technical solutions for an optimal durability design. It may also be used for the assessment of various technical solutions for condition assessment, maintenance and repair strategies during operation of the structure.

As a basis for the life cycle costs of a concrete structure up to the time t_N, the following expression may be used:

$$\mathrm{LCC}(t_N) = C_I + C_{QA} + \sum_{i=1}^{t_N} \frac{C_{IN}(t_i) + C_M(t_i) + C_R(t_i) + \sum_{LS=1}^{M} p_{f_{LS}}(t_i) \cdot C_{f_{LS}}}{(1+r)^{t_i}} \tag{12.1}$$

where:

C_I = design and construction cost
C_{QA} = cost of quality assurance and quality control
$C_{IN}(t)$ = expected cost of inspections
$C_M(t)$ = expected maintenance costs
$C_R(t)$ = expected repair costs
M = number of limit states, LS
$p_{fLS}(t)$ = annual probability of failure for each limit state
C_{fLS} = failure costs associated with the occurrence of each limit state
r = discount rate.

However, the above calculation of life cycle costs does not account for the advantage of designing or maintaining the structure for achieving a longer service life. Therefore, it may be more meaningful to compare all costs on an annual-equivalent basis by distributing all life cycle costs over the whole lifetime of the structure. This may be done by using an annuity factor which expresses the annuity or annual costs. The average annuity cost (C_A) during the service life of a structure (n years) may then be expressed as:

$$C_A(t_N) = \sum_{j=1}^{n} \frac{p_f(t_j) \cdot [C_1 + C_{QA} + C_{IN}(t_j) + C_M(t_j) + C_R(t_j)]}{1-(1+r)^{-t_j}} \quad (1) \qquad (12.2)$$

where $p_f(t_j)$ represents the probability of failure in year (j) and

$$p_f(t_n) = 1 - \sum_{j=1}^{n-1} p_f(t_j) \qquad (12.3)$$

Experience has shown that calculations of annuity costs represent a more appropriate way of expressing increased investment costs for increased durability. Calculations of life cycle costs may also include other costs or benefits such as traffic delays or reduced travel time as well as efficiency of inspections, maintenance and repair strategies and so on. Evidently, the decision analysis should be the subject of a sensitivity analysis in order to ensure that decisions are not unduly influenced by the uncertainties in the various types of costs.

12.2 Case study

In order to demonstrate how an assessment of life cycle costs may provide a further basis for decision-making of various technical solutions for improved durability, a heavily corroding concrete structure in Trondheim Harbour was selected for a case study. The structure, which was constructed in 1964, was a traditional concrete harbour structure with an open concrete deck of 132 × 17 m on top of slender underwater-cast concrete pillars. The concrete deck has three longitudinal main girders and 18 transversal secondary beams with two-way slabs in between. The main girders and the secondary beams have dimensions 90 × 120 cm and 70 × 70 cm, respectively, while the top slab is 25 cm thick including a 6 cm top layer. The mechanical loads on the deck consist mainly of two heavy loading cranes with capacities of 60 and 100 tons, respectively, moving on top of the three longitudinal main girders (Figure 12.1).

After a service period of about 38 years (2002), the general condition of the structure was very poor. A structural assessment confirmed that the load-bearing capacity of the main girders would only be acceptable for continued operation of the heavy loading cranes for a very short period of time. The observed rate of corrosion in the girders was so high that an immediate repair was considered very urgent. The concrete pillars were in fairly good condition, but the deck slabs had already reached such a high degree of deterioration that all other traffic on the concrete deck had been prohibited for some time. Of the various technical solutions for repair considered, one option

Figure 12.1 A heavily corroding concrete harbour structure from 1964 in Trondheim (source: Courtesy of Trondheim Harbour KS).

would simply be to construct a new concrete deck on top of the old deck, using the old deck as a formwork. However, since both the heavy crane facilities and the harbour structure as a whole would not be needed for continued operation of more than a further 15 years, the construction of a new concrete deck would be a too expensive solution. Therefore, a specially designed cathodic protection system was applied in order to reduce the rate of corrosion in the three main girders be a too much as possible and thus extend the service life of the structure for a limited period of time (Vælitalo *et al.*, 2004).

If the given concrete harbour structure had originally been the subject of a proper durability design including assessment of life cycle costs, a more controlled durability and service life could have been obtained. If the objective for such a durability design had been to keep a safe operation of the structure for a service period of approximately 50 years, the following life cycle costs of various technical solutions could have been considered. In order to demonstrate the usefulness of such calculations, a very simple cost comparison of various possible technical solutions was carried out. In order to provide a basis for such calculations, some technical data about the old structure were needed. Although such data were not easily available, some information was provided (Table 12.1).

In order to demonstrate the principles for the cost calculations, the following alternative options were considered:

- Do nothing other than what was originally designed.
- Increase concrete quality from 45 to 70 MPa.

Table 12.1 Basic information about the existing concrete structure

Concrete quality	45 MPa
Concrete cover in beams and girders	75 mm
Concrete cover in slabs	25 mm
Assumed new construction costs	25,000,000 NOK
Amount of concrete	1532 m^3
New costs of concrete (1.200 NOK/m^3)	1,840,000 NOK
Amount of steel	315 tons
New costs of steel (3.650 NOK/t)	1,150,000 NOK
Material costs related to total costs:	
Concrete	7.4%
Steel	4.6%

- Increase concrete cover from 75 to 100 mm in beams and girders.
- Increase concrete quality from 45 to 70 MPa in combination with increased concrete cover from 75 to 100 mm.
- Partly use stainless steel as reinforcement in beams and girders (75 per cent).
- Use stainless steel as reinforcement in beams and girders (100 per cent).
- Use cathodic protection.

In the following, the life cycle costs of all the above options were compared for a service period of 50 years. For convenience, the discount rate was put at zero in all calculations. Annuity costs were therefore calculated by the total costs divided by the expected service life. Other maintenance costs that usually occur during operation of such a structure were not included.

Do nothing

The total life cycle cost for this option was NOK 25,000,000. It was assumed that the service life of the existing structure would end after a continued service period of approximately three years (2005), which means that the annuity costs were calculated to approximately NOK 630,000.

Increase concrete quality

By increasing concrete quality from 45 to 70 MPa based on a pure portland cement, a durability analysis indicated an increased service life of the existing structure by up to ten years. The material costs for such increased quality of concrete would be approximately NOK 2,200,000. Hence, increased costs of NOK 380,000 would increase the total costs by 1.5 per cent to NOK 25,380,000. For an assumed extended service life of approximately ten years, the annuity costs were calculated at approximately NOK 500,000.

Increase concrete cover

Based on a concrete quality of 45 MPa, a durability analysis indicated that increasing concrete cover up to 100 mm in beams and girders would be necessary in order to reach a service life of approximately 50 years. By increasing concrete cover from 75 to 100 mm, the additional material costs would be approximately NOK 70,200 (58.5 m^3 concrete), which would give an increase in total costs of 0.2 per cent to NOK 25,070,200. For an extended service life of approximately ten years, the annuity costs were calculated to approximately NOK 500,000.

Increase concrete quality and concrete cover

By combining the beneficial effects of increasing concrete quality from 45 to 70 MPa and increasing concrete cover from 75 to 100 mm in all beams and girders, an estimated extended service life of approximately 25 years would have been achieved. The material costs for such a solution would be approximately NOK 2,420,000, giving an additional cost of NOK 580,000. This would increase the total costs by 2.3 per cent to NOK 25,580,000. For an assumed extended service life of approximately 25 years, the annuity costs were calculated to approximately NOK 380,000.

Partly use stainless steel reinforcement (75 per cent)

As previously discussed in Chapter 7, stainless steel reinforcement has proved to perform extremely well in marine environments for a very long period of time. In the literature, it is generally assumed that a proper use of stainless steel will increase the service life by a factor of at least two (Cramer *et al.*, 2002). A technical option could therefore have been to select stainless steel reinforcement. By using 75 per cent of stainless steel in all beams and girders with a cost ratio to black steel of 4.5 (Cramer *et al.*, 2002), the material costs for the reinforcement would be approximately NOK 4,170,000, which would increase the total costs by 12.1 per cent to NOK 28,020,000. For an assumed extended service life of at least 40 years, the annuity costs were calculated at less than NOK 350,000.

Use stainless steel reinforcement (100 per cent)

By using 100 per cent stainless steel in all beams and girders, the material costs for the reinforcement would increase to approximately NOK 5,200,000, which again would increase the total costs by 16.1 per cent to NOK 30,200,000. With an assumed extended service life of at least 40 years, the annuity costs were calculated at less than NOK 380,000.

Cathodic protection

After 38 years of service, a specially designed cathodic protection system was developed and installed for the given structure in order to extend the service life of the three main girders by approximately 15 years. Since the cost of this installation was approximately NOK 3,000,000, the annuity costs for this solution were calculated at approximately NOK 540,000.

12.3 Evaluation and discussion of obtained results

A summary of the results from the above cost calculations is shown in Table 12.2, from which it may be seen that a proper utilization of stainless steel in all beams and girders would have generated distinctly lower annuity costs compared to those of the other technical solutions.

The results from the above case study demonstrate that assessment of life cycle costs is a very valuable tool for improved decision-making in the durability design. For the concrete harbour structure in question, a proper utilization of stainless steel would have resulted in a very safe and cost-effective operation compared to that of the traditional design with the corresponding repair and maintenance costs. As already discussed in Chapter 11, such a technical solution would also have given a very simple and robust technical solution for ensuring proper durability.

Table 12.2 Comparison of life cycle costs for various technical solutions for ensuring proper durability of the concrete harbour structure during a service period of 50 years

	Additional service life (years)	*LCC(t_{end}) (%)*	*Annuity costs $C_A(t_n)$ ($\times 10^6$ NOK)*
Doing nothing	0	100.0	0.63
Increased concrete quality	+10	101.5	0.50
Increased concrete cover	+10	100.2	0.50
Increased concrete quality and concrete cover	+25	102.3	0.38
75% stainless steel rebars	>40	112.1	<0.35
100% stainless steel rebars	>40	116.1	<0.38
Cathodic protection	+15	112.0	<0.54

13 Life cycle assessment

13.1 General

In recent years, there has been a rapidly increasing concern about how human activities affect the loss of biodiversity, the thinning of stratospheric ozone, climate changes and the consumption of natural resources. The term 'Sustainable Development' was introduced in the final report of the Brundtland Commission (World Commission on Environment and Development – WCED) in 1987, where 'Sustainable Development' was defined as:

> Development that meets the needs of the present without comprising the ability of future generations to meet their own needs.

On the basis of both weight, volume and money, the construction industry is the largest consumer of materials in our society. Thus, approximately 40 per cent of all materials used are related to the construction industry (Ho *et al.*, 2000). From a production point of view, several of the construction materials have a great impact on both the local and global environment. This is particularly true for concrete as one of the most dominating construction materials. Therefore, an increased environmental consciousness in the form of a better use of concrete as a construction material and the creation of a better harmony and balance with our natural environment represents a great and increasing challenge to the construction industry. This was the conclusion of two international workshops that focused on this problem, the first of which resulted in the 'Hakodate Declaration' of 1996 (Sakai, 1996), and later in the 'Lofoten Declaration' of 1998 (Gjørv and Sakai, 2000), which stated:

> We concrete experts shall direct concrete technology towards a more sustainable development in the 21st century by developing and introducing into practice:
>
> 1 Integrated performance-oriented life cycle design
> 2 More environmental-friendly concrete construction

3 Systems for maintenance, repair and reuse of concrete structures.

In addition, we shall share information on all these issues with technical groups and the general public.

The production of concrete is based on the large consumption of natural resources for concrete production, the production of pure portland cements is based on a very energy-consuming and polluting industrial process. Although distinct improvements in the production technology of cements have been made in recent years, the production of each ton of portland cement still typically requires an energy amount of approximately 4 GJ, while almost 1 ton of CO_2 gas is released into the atmosphere. In addition, a number of other harmful constituents are also released (Table 13.1). On a worldwide basis, cement production comprises about 1.4 billion tons of CO_2 every year, which is about 7 per cent of the total global production of CO_2 (Malhotra, 1999). About half of the CO_2 emissions from cement production is due to the decarbonizing of limestone, while the other half is due to the combustion of fossil fuels. Therefore, a proper durability design and concrete quality control for ensuring a high and more controlled performance and service life of concrete structures are of the greatest importance from an environmental point of view.

As already discussed in Chapter 2, an uncontrolled and premature deterioration in concrete structures has emerged to become one of the most demanding challenges facing the construction industry. Public agencies are spending a significant and rapidly increasing proportion of their available construction budgets on the repair and maintenance of all existing concrete infrastructures.

In years to come, the repair and maintenance of all these concrete structures will be subject to stricter requirements with regard to both economical and environmental constraints. It is therefore imperative also to take the environmental impact and effects into consideration during the design, construction and operation of concrete structures.

Table 13.1 Typical energy consumption and gas emission for production of portland cements

Energy consumption (MJ/kg)		
Electricity	*Fossil fuel*	*Total*
0.56	3.48	4.05

Emission to the atmosphere (g/kg)	
CO_2	867
SO_2	0.29
NOx	1.98
VOC	0.02
Dust	0.17

Source: Gjørv (1999).

In years to come, a rapid development in environmental design will take place. Through the ISO standards 14040–14043, both the framework and methodology for quantifying the ecological effects and impacts of the design, production and maintenance of concrete structures are already available (ISO, 1998–2000). Life cycle assessment includes assessment of both materials and energy consumption, waste generation and emission to the environment as well as health risks. Therefore, life cycle assessment (LCA) provides a valuable tool both for quantifying and comparing the ecological effects of various technical solutions for the environmental design, construction and operation of concrete structures.

The objective of this chapter is primarily to give a brief introduction to the methodological framework of LCA. Although the largest environmental gains can be made by a proper environmental design and construction of new concrete structures, a short case study is included on how the framework may also be applied for the assessment of environmental impacts of various types of repair and maintenance systems during operation of the concrete structures.

13.2 Framework for life cycle assessment

The Institute of Ecosystem Studies defines ecology as:

> The scientific study of the processes influencing the distribution and abundance of organisms, the interaction among organisms, and the interactions between organisms and the transformation and flux of energy and matter.

A practical way of interpreting LCA is to determine the impact on the environment caused by all human activities throughout the whole life cycle of the structure. However, this is a very difficult process, since the relationship between the external environment and the category end-point can be very complex. Normally, the LCA will stop at the step before the category end-point showing only the impact categories which are fairly easy to do, and then interpret the results from the various category indicators. The concept of category end-points is shown in Figure 13.1. Further information about the methodological framework for the assessment of environmental impacts is available in the ISO standards 14040–14043 (ISO, 1998–2000).

In order to demonstrate the concept of category indicators applied to the repair and maintenance of concrete structures, all steps in the process have to be thoroughly evaluated. From the condition assessment and investigation of a given structure, the type of repair or maintenance action is first selected. The selected action depends on the condition of the structure and of the external environment as well as the type of equipment and materials to be used during the process. The next step is to determine

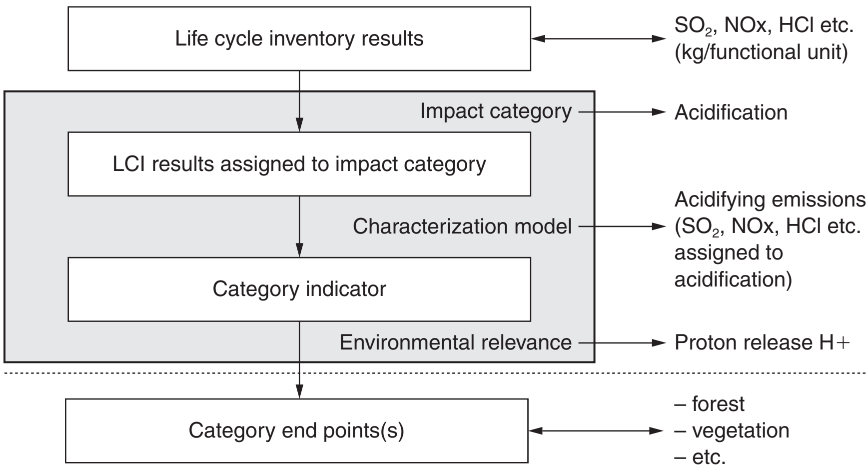


Figure 13.1 Concept of category indicators (source: ISO (2000a)).

the functional unit, which is the reference unit used in the life cycle study (ISO, 1999). All emission, energy and flow of materials occurring during the process are related to this unit. The functional unit which has to be measurable depends on the goal and scope of the analysis. The goal of the life cycle assessment will unambiguously state the intended application and indicate to whom the results will be addressed. Thus, the functional unit for a protective coating may be defined as the unit of concrete surface (m^2) which needs to be protected for a specified period of time.

The life cycle inventory (LCI) phase will then consist of:

1 Quantifying the amount of all raw materials, chemicals and equipment which are necessary to fulfil the repair or maintenance function. This quantification gives the reference flow (ISO, 2000b) by which all inputs and outputs are referred to and is closely connected to the functional unit.
2 Environmental data of consumed raw materials, chemicals and equipment from the suppliers (specific data) or from databases (generic data) or from a LCI carried out at supplier level. All materials used should have an environmental declaration with a 'cradle-to-grave'-type scope. The environmental declaration will include use of resources such as energy (renewable or non-renewable), materials (renewable or non-renewable), water and waste as well as emissions into both air and water.
3 Quantifying and classifying waste from the process such as recycling or disposal (hazardous or non-hazardous).

The calculations of impact categories should then be carried out according to Figure 13.1. The impact categories include global warming, ozone depletion, acidification, photo-oxidant creation and eutrophication. All calculated effects should be potential effects.

The classification is the assignment of LCI results to impact categories. Classification and characterization should be carried out according to ISO 14042, using effect factors from IPCC in the Montreal Protocol (Centrum voor Milieukunde, 1999). Emission of a specific gas may be assigned to more than one category, an example of which is the emission of NO_x, which will be assigned to the categories of both eutrophication and acidification.

The final results may be displayed as impact categories or weighted to an environmental index, where the weighting is the process of converting indicator results of the different impact categories by using numerical factors based on value-choice. This is an optional element in ISO 14042. Thus, factors from value-choices may be based on political targets according to the Kyoto Protocol or other similar preferences. Interpretation of the results based on ISO 14043 will identify, qualify, evaluate and present the findings of significant issues.

13.3 Case study

General

In order to demonstrate how the above methodological framework for the assessment of environmental impacts can be applied to various types of repair and maintenance systems for a concrete structure, two examples of commonly used systems have been the subject of analysis (Årskog *et al.*, 2003), the results of which are briefly outlined in the following. The first system was a patch repair based on shotcreting, where the damage was caused by chloride-induced corrosion of embedded steel. The second system was a hydrophobic surface protection, which is commonly used as a preventive measure for the protection of concrete structures against chloride penetration (Chapter 7).

For both cases, the following common assumptions for the calculation of the ecological impacts were made:

- Same transport distance back and forth (60 km).
- Materials and equipment were transported by truck.
- Fuel consumption (diesel) for the truck was 0.2 kg per ton-km.
- Same functional unit (1 m^2 of repaired or protected concrete surface for a period of ten years).

Patch repair

This analysis was based on the following assumptions:

- Surface area repaired: 30 m^2.
- Rebound of shotcrete: 25 per cent.
- Power supply on construction site based on diesel engines.

The various steps of the process considered:

- Removal of concrete cover to an average depth of 50 mm by high-pressure hydro jetting (1000 bar).
- Cleaning the reinforcing bars by sand blasting.
- Protective coating of the reinforcement.
- Application of the shotcrete layer.
- Curing measures for the applied shotcrete.

The consumption of energy and the ecological impacts of the patch repair are summarized in Table 13.2.

Hydrophobic surface protection

This analysis was based on the following assumptions:

- Hydrophobic agent: iso-octyltriethoxy type of silane in combination with a mineral thickener.
- Surface area treated: 150 m^2.

The various steps of the process considered:

- Preparation of the concrete surface by high-pressure sand blasting (160 bar).
- Application of the hydrophobic agent by use of a high-pressure sprayer to a thickness of 0.25 mm. It was assumed that only 45 per cent of the hydrophobic agent was applied to the concrete surface, which is equivalent to approximately 500 g/m^2, while the rest of the agent (approximately 600 g/m^2) was emission into the air. The iso-octyltriethoxy type of silane is volatile, and ethanol is released into the atmosphere.

The consumption of energy and the ecological impacts of the hydrophobic surface protection are summarized in Table 13.3.

Table 13.2 Energy consumption and ecological impacts of the patch repair

Process	*Impact category*				
	Use of energy (MJ/m^2)	*Global warming (kg CO_2 eq/m^2)*	*Acidification (g SO_2 eq/m^2)*	*Eutrophication (g PO_4 eq/m^2)*	*Photo-oxidant formation (g Ethene eq/m^2)*
Hydro jetting	677	84	75	1330	266
Cleaning reinforcement	296	22	4	350	70
Protective coating on reinforcement	35	1.4	19	2.4	3
Application of shotcrete	59	4.4	19	70	14
Transportation	127	10	8	150	30
Total	1194	122	125	1902	383

Table 13.3 Energy consumption and ecological impacts of the hydrophobic surface protection

Process	*Impact category*				
	Use of energy (MJ/m^2)	*Global warming (kg CO_2 eq/m^2)*	*Acidification (g SO_2 eq/m^2)*	*Eutrophication (g PO_4 eq/m^2)*	*Photo-oxidant formation (g Ethene eq/m^2)*
Production of hydrophobic agent	47	295	0.5	6	2
Surface preparation	17	13	0.4	7	1
Transportation and surface treatment	12	80	0.1	2	66
Long-term degradation	–	2171	–	–	1
Total	76	2559	1	15	70

Table 13.4 Comparison of the ecological impacts caused by patch repair and a protective measure based on hydrophobic surface treatment

Method	*Impact category*				
	Use of energy (MJ/m^2)	*Global warming (kg CO_2 eq/m^2)*	*Acidification (g SO_2 eq/m^2)*	*Eutrophication (g PO_4 eq/m^2)*	*Photo-oxidant formation (g Ethene eq/m^2)*
Hydrophobic treatment	76	2.6	1	15	70
Patch repair	1642	122	125	1902	383

13.4 Evaluation and discussion of obtained results

In order to carry out the above comparison of the ecological impacts caused by the repair and protective measures of the given concrete structure, a number of assumptions had to be made. The results obtained indicate, however, that the hydrophobic surface treatment can be repeated more than five times before the ecological impact in the form of photo-oxidant formation approaches that of patch repair by shotcreting (Table 13.4).

A more complete assessment of all impacts on the environment caused by human activities throughout the whole life cycle of a concrete structure is both very complex and difficult. Based on the methodological framework for life cycle assessments (LCA) as briefly outlined above, however, LCA appears to be a very good tool for the environmental design of new concrete structures (Sakai, 2005). Extensive work is currently being conducted in order to develop international standards and guidelines for more practical applications of LCA (ISO, 2008).

Although the largest environmental benefits can be gained from the proper design and construction of new concrete structures, the above case study demonstrates that LCA also provides a good basis for selecting a proper strategy for the operation of concrete structures. Thus, it appears to be a very good environmental strategy to carry out regular condition assessments and preventive maintenance of concrete structures before a stage is reached where repairs are needed.

14 Recommended job specifications

14.1 General

Although most codes and specifications for the durability of concrete structures have been upgraded and improved in recent years, they are still mostly based on prescriptive requirements of concrete composition and the execution of concrete work, the results of which are neither unique nor easy to verify and control during concrete construction (Chapter 5). Upon completion of a new concrete structure, therefore, the structure is handed over to the owner without proper documentation of achieved construction quality and compliance with the specified durability. After some years, the owner may be faced with serious and costly maintenance problems during operation of the structure.

For many years, when concrete was mostly based on pure portland cements and simple procedures for concrete production, the concept of water/cement ratio was the fundamental basis for characterizing and specifying concrete quality. As already discussed in Chapter 5, a number of different cementitious materials and reactive fillers are increasingly being used for concrete production, and the concrete properties are increasingly being controlled by the various combinations of such materials. In addition, the concrete properties increasingly being controlled by the use of various types of processed concrete aggregate, new concrete admixtures and sophisticated production equipment. Therefore, the old and very simple terms 'water/cement ratio' and 'water/binder ratio' for characterizing and specifying concrete quality have successively lost their meaning. As a consequence, there is a great need for performance-based definition and specification for concrete quality. In particular this is true for characterizing and specifying concrete durability.

As discussed in Chapter 2, it appears that it is not possible to produce concrete structures without a high scatter and variability of achieved construction quality. For concrete structures in severe environments, extensive field experience has shown that any weakness in the concrete structure will soon be revealed, whatever its constituent materials may be. For such concrete structures, therefore, better control of construction quality and variability is a key issue which must be firmly grasped before a more rational approach to more controlled durability can be achieved. Hence, it is very important to carry out a probability-based durability design which will take some of the above scatter and variability into account (Chapter 6). As a result, performance-based concrete quality control during concrete construction can also be carried out (Chapter 8). In this way, a documentation of achieved construction quality and compliance with the specified durability can be provided (Chapter 9).

For all concrete structures, the minimum durability requirements according to current concrete codes must always be fulfilled. However, for all concrete structures in severe environments where high performance and safety are of special importance, probability-based durability design and performance-based concrete quality control during concrete construction should be carried out. For such structures, extensive experience has shown that increased durability beyond what is possible to reach based on current concrete codes appears to be a very good investment (Chapter 12). Such increased durability is not only a technical and economical issue but also an increasingly important environmental and sustainability issue (Chapter 13). Based on the durability design and performance-based concrete quality control as outlined and discussed in previous chapters, the following job specifications are recommended.

14.2 Job specifications

1 Probability of steel corrosion

As an overall durability specification to the given concrete structure in the given environment, the structure shall have a certain required service period before a probability of 10 per cent for steel corrosion is reached.

Based on the durability design as outlined in Chapter 6, a combination of concrete quality and concrete cover for a service period of up to 150 years without exceeding a 10 per cent probability for corrosion may be required. For all important concrete structures where high performance and safety are of special significance, a service period of at least 100 years should be required.

Due to a number of simplifications and assumptions made in the calculation of the above ‘service period’, it should be noted that such a ‘service period’ cannot be considered to be a real service period for the given structure. As discussed in Chapter 6, however, the above service period is the result of an engineering approach and judgement of the most important factors relevant for the durability including the scatter and variability of all factors involved. By comparing the obtained service periods for various combinations of concrete quality and concrete cover, a good engineering basis is obtained for selecting one of several technical solutions in order to obtain the best possible durability for a given concrete structure in a given environment. As a result, proper requirements for both chloride diffusivity and concrete cover can then be specified.

2 Additional strategies and protective measures

If it should be difficult or not possible to meet the required service period as defined above, or if a service period of more than 150 years should be required, additional strategies and protective measures shall be applied.

Based on local conditions and availability of a proper concrete quality, it may not always be possible to meet the required service period. For certain concrete structures, a service period of more than 150 years may also be required. In severe marine environments, the risk for early age exposure during concrete construction before the concrete has gained sufficient maturity and density may also be high. In all such cases, one or more of the additional protective strategies and measures as outlined in Chapter 7 shall be applied.

3 Achieved construction quality

During concrete construction, performance-based concrete quality control shall be carried out. Upon completion of the concrete construction work, this shall provide documentation for compliance with the specified durability. Additional documentation on both achieved durability on the construction site and potential durability of the structure shall also be provided.

Extensive experience has shown that much of the durability problems which develop after some time may be related to an absence of proper quality control as well as specific problems during concrete construction. It is very important, therefore, to carry out performance-based concrete quality control during concrete construction (Chapter 8). In addition to the documentation of compliance with the specified durability, it is also important to provide some additional documentation on both achieved durability on the construction site and potential durability of the structure.

4 Condition assessment and preventive maintenance

As part of the durability design, a service manual for monitoring the real chloride penetration during operation of the concrete structure and recommendations for how to control this chloride penetration shall be produced.

Even if the strongest requirements for both concrete quality and concrete cover have been specified and achieved during concrete construction, extensive experience demonstrates that for all concrete structures in severe chloride-containing environments, a certain rate of chloride penetration will always take place during operation of the structure (Chapter 2). Before the new concrete structure is handed over to the owner, therefore, it is very important to provide the owner with a service manual for regular condition assessment and preventive maintenance of the structure (Chapter 10). For concrete structures in severe chloride-containing environments, it is only regular monitoring of the real chloride penetration during operation of the structure and evaluation of the future corrosion probability in combination with protective measures that provide the ultimate basis for achieving a more controlled service life of the given concrete structure in the given environment.

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