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Durability of Concrete Structures

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Abstract Extensive experience demonstrates that the durability of concrete structures is related not only to design and material but also to construction issues. Upon completion of new concrete structures, the achieved construction quality always shows a high scatter and variability, and in severe environments, any weaknesses in the concrete structures will soon be revealed whatever specifications and constituent materials have been applied. In order to better take all this variability into account, a probability-based durability design should be applied. Since many durability problems also can be related to poor 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. Therefore, a performance-based concrete quality control during concrete construction with documentation of achieved construction quality and compliance with the specified durability should also be carried out. When the concrete structure is completed, the owner should further be provided with a proper service manual for the future operation of the structure. Only such a service manual for condition assessment and preventive maintenance can provide the ultimate basis for achieving a more controlled durability and service life of concrete structures in severe environments. In the present paper, current experience with probability-based durability design and performance-based concrete quality control is briefly outlined and discussed.

Keywords Service life · Durability · Durability design · Probability analysis · Durability requirements · Construction quality · Quality assurance · Condition assessment · Preventive maintenance

الخلاصة

تبين الخبرات الواسعة أن ديمومة المنشآت الخرسانية لا تتعلق فقط بالتصميم والمواد ولكن أيضا بقضايا الإنشاء. وعند الانتهاء من المنشآت الخرسانية الجديدة دائما ما تظهر جودة الإنشاء المنجزة درجة عالية من البعثرة والتغير، وفي الظروف القاسية فإن أي ضعف في المنشآت الخرسانية سيكشف سريعا مهما طبقت المواصفات والمواد المكونة لهذه الخرسانة.
ومن أجل أخذ أفضل هذه الاختلافات في الاعتبار، فإنه يجب تطبيق تصميم الاحتمالية – المثانة، نظرا لمشاكل الديمومة الكثيرة . أيضا نستطيع القول إنهما بسبب ضعف مراقبة الجودة بالإضافة إلى مشاكل خاصة خلال عملية الإنشاء . لذا فإن تقنية الجودة وتغير البناء يجب أن تؤخذ بحزم قبل أي توجه عقلاني للحصول على أعلى درجة من التحكم ، ولذلك يجب تطبيق التحكم في جودة أداء الخرسانة خلال عملية البناء مع الديمومة المحددة. وعند انتهاء البناء الخرساني يجب أن يزود المالك بدليل الخدمة المناسب لعملية الصيانة المستقبلية . وهو دليل لتقييم الحالة والصيانة الوقائية التي توفر الأساس في نهاية المطاف لتحقيق مزيدٍ من الديمومة وعمر الخدمة للمنشآت الخرسانية في الظروف القاسية. لقد تم - في هذه الورقة - عرض الخبرات الحالية ومناقشتها مع تصميم احتمالية – ديمومة الخرسانة وأدائها في التحكم في الجودة.

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1 Introduction

For many years, extensive field investigations of a large number of important concrete structures have been carried out. The results of all these investigations have been reported in comprehensive reports and published in numerous papers in various journals and proceedings from various international conferences over a long period of time [1]. As early as 1924, Atwood and Johnson [2] had compiled a list of about 3000 references on concrete durability. Even in the most severe environments, extensive literature has shown that it is not the disintegration of the concrete itself but rather electrochemical corrosion of embedded steel which poses the greatest threat to the durability and long-term performance of concrete structures in severe environments [3,4]. As early as 1917, this was pointed out by Wig and Ferguson [5] after a comprehensive survey of concrete structures in US waters.

Later on, the use of de-icing salt on highway bridges created a new challenge to the durability and long-term performance of these bridges. Thus, in 1986 it was estimated that the cost of correcting corroding concrete bridges due to de-icing salt in the USA was US $24 billion, with an annual increase of US $500 million [6]. Later on, annual costs of repair and replacement of US bridges were estimated to be up to about US $8.3 billion by Yunovich et al. [7] and up to US $9.4 billion for the next 20 years by the American Society of Civil Engineers [8]. In 1998, annual costs of US $5 billion for concrete structures in Western Europe were estimated [9], and similar durability problems have also been reported with corresponding costs from a large number of other countries.

Internationally, deterioration of concrete infrastructure has emerged as one of the most severe and demanding challenges facing the construction industry [10]. Although corrosion of embedded steel represents the predominant type of deterioration, freezing and thawing and alkali aggregate reactions also represent big challenges to the durability and long-term performance of many concrete structures. Based on current codes and practice, however, it appears to be easier to control such durability problems compared to that of steel corrosion [11]. Thus, for concrete structures in severe environments, it may be difficult to avoid steel corrosion within typical service periods of 15–20 years [1].

Since concrete structures make up a very large and important part of the national infrastructure in many countries, the condition and performance of these structures are very important for the productivity of society [12]. Since there is a growing amount of deteriorating concrete structures, however, not only is the productivity of the society affected, but it also has a great impact on resources, the environment and human safety. The operation, maintenance and repair of concrete structures are consuming much energy and resources and are producing a heavy environmental burden and large quantities of waste. Thus, poor durability and premature service life of many concrete structures represent not only technical and economic problems; this is poor utilization of natural resources and hence also presents sustainability and ecological problems [13].

In order to obtain an increased and more controlled durability and service life of important concrete infrastructure, a rapid international development on both probability-based durability design and performance-based concrete quality control has taken place, a brief outline of which is presented and discussed in the following. A more complete durability design should also take the various costs (LCC) and environmental impacts (LCA) into account [1], but these aspects are not included and discussed in the present paper.

2 Durability Design

2.1 General

In 1989, the European Union produced a Construction Products Directive [14], requiring documentation of achieved durability of buildings and structures. This document was primarily based on the general technical basis for service life design of buildings and structures developed by the ISO. It then was up to the various industrial sectors to come up with more detailed technical procedures and specifications for such documentation. For concrete structures, both RILEM and CEN have put much work into the development of a better basis for service life design of concrete structures [15,16]. It was not until the European research project DuraCrete [17] was completed in 2000, however, that more general guidelines for a probability-based service life design became available. Since then, a model-code for probability-based service life design of concrete structures has been produced by fib [18]. Such a service life or durability design requires, however, that a mathematical model for the given deteriorating process exists, and that the input parameters to the model also can easily be determined. It is further necessary to have some information on both the average values and natural scatter of the various input parameters to the model.

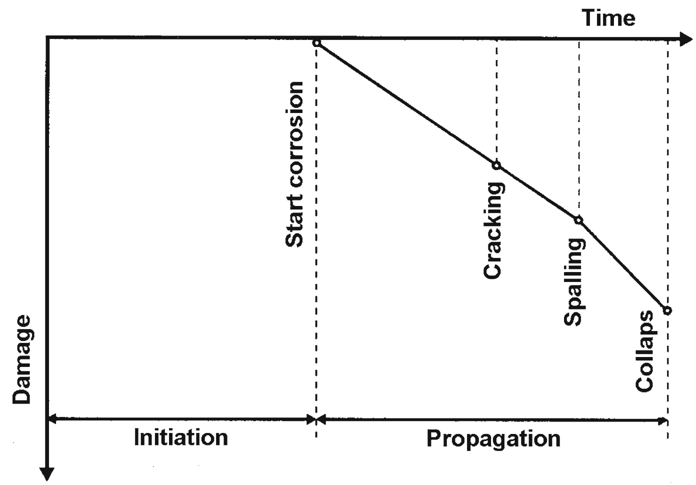

Fig. 1 Deterioration of a concrete structure due to steel corrosion [20]

Of the various deteriorating processes which can cause problems to concrete structures, proper mathematical models for the above design are currently available only for corrosion of embedded steel, due either to chloride penetration or concrete carbonation. For most new concrete structures, however, it may not be disintegration of the concrete itself but rather electrochemical corrosion of the embedded steel which poses the most critical threat to the safety, durability and service life of the structures. In particular, this is true for concrete structures in chloride containing environments.

2.2 Chloride-Induced Corrosion

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 appears in the form of cracks and rust staining, but it may take a long time before the load carrying capacity of the structure is severely reduced. Schematically, the deterioration process takes place as shown in Fig. 1. As soon as the corrosion process starts, a very complex system of galvanic cell activities in the concrete structure develops [19]. 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 cathodic areas act as catchment areas for oxygen. Although larger portions of the rebar system eventually become depassivated, not all of these areas will necessarily corrode. Generally, the first and most active corroding parts of the structure will act as sacrificial anodes and thus cathodically protect the other parts of the structure. Since both the structural shape and 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 for developing such a mathematical model have been made, it appears that no reliable mathematical model for this very complex deterioration process currently exists. In the early 1970s, however, Collepardi et al. [21,22] came up with a relatively simple mathematical model for estimating the time necessary for chlorides to reach embedded steel through concrete of a given quality and thickness.

Although it is possible to estimate the time likely to elapse until 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 in an early stage represents only a maintenance and cost problem, but later on develops into a safety problem that is more difficult to control. As a basis for the durability design, therefore, efforts should be made to obtain the best possible control of the chloride penetration during the initial period before corrosion starts. It is in this early stage of the deterioration process that it is both technically easier and much cheaper to take necessary precautions and to select proper protective measures to control subsequent deterioration.

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 as introduced in DuraCrete [17]. In this way, it is possible to take some of this scatter and variability into account and thus to calculate the probability of chlorides reaching the embedded steel during a certain service period.

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

Recent years have seen a rapid international development of various procedures for probability-based durability design of concrete structures [17,18,23–25], and in many countries, such durability design has been applied to a number of important concrete structures [26–29]. 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 [11,30]. In the beginning, this design was primarily based on the results and guidelines from DuraCrete [17], but as practical experience with such design was gained, the basis for the design was successively simplified and further developed for more practical applications. Thus in 2004, this design was adopted by the Norwegian Association for Harbor Engineers as general recommendations and guidelines for durability design of new concrete harbor structures [31,32]. Later on, new and revised editions were issued and, more recently, also adopted by the Norwegian Chapter of PIANC [33,34], which is the international professional organization for maritime infrastructure.

In general, the basic principles for probability-based durability design are more or less the same. In the following, however, a short outline of the Norwegian version of such a design is given, as it has been applied to a number of important concrete structures in recent years [1].

2.3 Calculation of Chloride Penetration

It is well known that the transport mechanisms for penetration of chlorides into concrete are rather complex [35]. In a very simplified form, however, it is possible to estimate the rates of chloride penetration by use of Fick's Second Law of Diffusion according to Collepardi et al. [21,22]. This is combined with the time dependence of the chloride diffusivity according to Takewaka and Mastumoto [36] and Tang and Gulikers [37] as shown in Eqs. 1 and 2:

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

In the above 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.

$$D(t) = \frac{D_0}{1-\alpha}\left[\left(1+\frac{t'}{t}\right)^{1-\alpha} - \left(\frac{t'}{t}\right)^{1-\alpha}\right]\left(\frac{t_0}{t}\right)^{\alpha}\cdot k_{\mathrm{e}} \tag{2}$$

In Eq. 2, 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 [38]:

$$k_{\mathrm{e}} = \exp\left[b_{\mathrm{e}}\left(\frac{1}{293} - \frac{1}{T+273}\right)\right] \tag{3}$$

In Eq. 3, exp is the exponential function, b_{e} is a regression parameter, and T is the temperature. The criterion for steel corrosion then becomes:

$$C(x) = C_{\mathrm{CR}} \tag{4}$$

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.

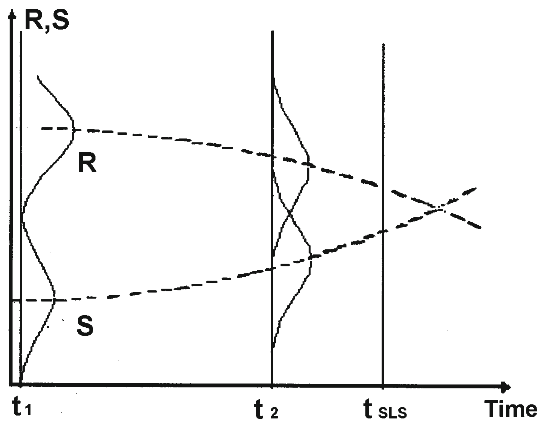

Fig. 2 The principles of a time dependent reliability analysis

2.4 Calculation of Probability

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

$$R \geq S \quad \text{or} \quad R - S \geq 0 \tag{5}$$

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

In principle, durability design takes the same approach as that of structural design. In this case, however, the effect of both loads (S), which is the combined effect of both chloride loads and temperature conditions, and the resistance to withstanding the loads (R), which is the resistance against 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 of "failure" less than a given value is the same.

In Fig. 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 overlap between these two distribution curves, but over time, a gradual overlapping from time t_1 to t_2 takes place. This increasing overlap 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 can be written as:

$$P(\text{failure}) = P_f = P(R - S < 0) < P_0 \tag{6}$$

where P_0 is a measure for failure probability.

According to current codes for reliability of structures, an upper level for probability of failure of 10% in the serviceability limit state is often required. Therefore, an upper probability level of 10% 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)
– Monte Carlo Simulation (MCS)

2.5 Calculation of Corrosion Probability

In principle, the calculation of corrosion probability can be carried out through any of the above mathematical methods with the use of appropriate software. Based on current experience with durability design of concrete structures in chloride containing environments, however, calculation of chloride penetration by use of Eq. 1 in combination with a Monte Carlo Simulation has proved to give a very simple and appropriate basis for calculating the corrosion probability. Although such a combined calculation also can be carried out in different ways, special software (DURACON) for this calculation has been developed [39].

The above calculation of corrosion probability provides the primary 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 the probability of 10% for corrosion is reached. For the given environmental exposure, requirements to both concrete quality (chloride diffusivity) and concrete cover can then be specified. For the later operation of the concrete structures, however, the above calculation also provides the basis for condition assessment and preventive maintenance as briefly outlined later. Based on the observed rates of the real chloride penetration taking place during operation of the structure, calculations of the future corrosion probability then provide the basis for the preventive maintenance of the structure.

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

2.6 Input Parameters

General

In general, the durability design should always be an integral part of the structural design for the given structure. At an early stage of the design, therefore, the overall durability requirement for the structure should be based on the specification of a certain service period before 10% probability of corrosion is reached. For the given environmental exposure, the durability analysis then provides the basis for specifying 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, α
  - Critical chloride content, C_{CR}
- Concrete cover, X

All the above parameters may have different distribution characteristics, but if nothing else is known, 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, the determination and selection of the above input parameters are briefly described and discussed.

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 Fig. 3, which is the result of a regression analysis of observed data on chloride penetration and curve fitting to Fick's Second Law. This surface chloride concentration (C_S) is normally higher than the maximum chloride concentration observed at the concrete surface (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. For the durability analysis, however, it is important to estimate and select a proper value for the surface chloride concentration (C_S), one which is as representative as possible for the most exposed and critical parts of the structure. In some cases, it may also be appropriate to select different chloride

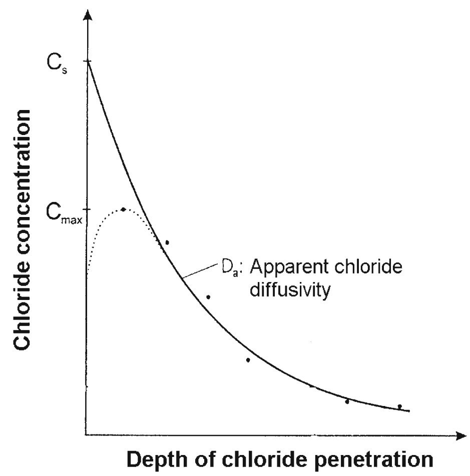

Fig. 3 Definition of the surface chloride concentration (C_S) based on a regression analysis of observed data on chloride penetration and curve fitting to Fick's Second Law of diffusion

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

For a new concrete structure, it may be difficult 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 environments should be applied. In many countries, a large number of both concrete bridges and concrete harbor structures in severe chloride exposure environments have been subjects of extensive field investigations. 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 proper chloride loads [1].

Although the chloride loads on a concrete bridge in a marine environment very much depend on its 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 many concrete coastal bridges, lower surface chloride concentrations are typically observed on those parts of the structures which are the most exposed to prevailing winds and salt spray compared to those of the more protected parts. In the most exposed parts, rain may intermittently wash off the salt, while on the more protected parts, the salt may accumulate.

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 Eq. 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 The chloride diffusivity (D_0) of a given concrete is a very important property which generally reflects the resistance of the concrete against chloride penetration. Although a low water/binder ratio generally gives a low porosity and permeability and, hence, a high resistance against chloride penetration, it is well documented that the selection of a proper cement or binder system may be more important for a high resistance against chloride penetration than selecting a low water/binder ratio. Thus, a water/binder ratio reduced from 0.45 to 0.35 for a concrete based on a pure portland cement may reduce the chloride diffusivity to only a small extent compared to that of replacing the portland cement with another type of cement such as blast furnace slag cement [40]. If the slag cement is also combined with pozzolanic materials such as silica fume, an extremely low chloride diffusivity and, hence, an extremely high resistance against chloride penetration may be obtained [1].

In the literature, there are several types and definitions for the chloride diffusivity of a given concrete, and several methods for testing of the chloride diffusivity [41]. Thus, NORDTEST has standardised three different types of such test methods, including the steady state migration method NT Build 355 [42], the immersion test method NT Build 443 [43] and the non-steady state migration method NT Build 492 [44]. All of these test methods give different values for the chloride diffusivity, but since they all show a good correlation [1], any of the established test methods can be applied both for quantifying and comparing the resistance against chloride penetration of various types of concrete.

Although all the above test methods are accelerated methods, the duration of the testing is very different. For the non-steady state migration method, however, there is no requirement for pre-curing the concrete, and the testing usually takes only a couple of days. Both the steady-state migration method and the immersion test method are based on well cured concrete specimens, and the duration of the testing may take a very long time. Therefore, in order to be applied as a 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 suitable test method [1]. In particular, this is true if this test method also is combined with a corresponding testing of the electrical resistivity of the concrete as described and discussed later on with regard to concrete quality control.

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 standardised 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, however, the testing is normally based on the 28-day chloride diffusivity (D_{28}) in the same way as that for the regular control of compressive strength.

For continued water curing in the laboratory beyond 28 days, the chloride diffusivity is successively reduced over a certain period of time; depending somewhat on type of binder system, it typically tends to level out within a curing period of approximately 1 year. Therefore, the obtained chloride diffusivity after 1 year (D_{365}) is used as a basis for reflecting the potential resistance of the given concrete against chloride penetration, as also discussed later.

Time dependence, 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 also could be determined on the basis of laboratory testing, this would be time consuming and, at the same time, would not properly reflect how the chloride diffusivity of the given concrete in the given environment develops over time. In order to more realistically reflect 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 probability or durability analyses.

For new concrete structures, therefore, the same problem exists in selecting a proper α-value 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. Also, 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 values for selecting a proper α-value are given in Table 1. This table shows some observed α-values for various types of concrete based on various binder systems exposed to the tidal and splash zones of marine environments [17,18,45–48]. Although combinations of the various types of cement with pozzolanic materials such as silica fume or fly ash always will reduce the chloride diffusivity, current experience indicates that Table 1 may still be used as a rough basis for estimating a proper α-value.

Table 1 Some general values 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

Table 2 Risk for development of corrosion depending on chloride content [48]

Chloride content (%)		Risk of corrosion
By wt. of cement	By wt. of concrete[a]	
>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

[a] Based on 440 kg/m^3 of cement

Critical chloride content, C_{CR} It is well known that 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. Also, due to the very complex relationship between the total chloride content in a concrete and the passivity of embedded steel, it is not possible to give any general values for critical chloride contents. When certain values for the critical chloride content nevertheless are given in existing concrete codes and recommendations, this is based only on empirical information on chloride contents which may give a certain risk for development of corrosion (Table 2). However, whether an unacceptable development of corrosion may take place or not also depends on other corrosion parameters such as the electrical resistivity of the concrete and the oxygen availability. Generally, however, very small chloride concentrations in the pore solution of a concrete are able to destroy the passivity of the steel, but the risk for 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 oxygen availability [19].

Based on empirical experience from a wide range of concrete qualities and moisture conditions, an average value of 0.4% by weight of cement is often referred to in current concrete codes and recommendations. Therefore, if nothing else is known, an average value of 0.4% with a standard deviation of 0.1% 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% with a standard deviation of 0.03% may be selected. For various types of corrosion resistant steel, however, critical chloride contents of up to 3.5% or even up to 5.0% by weight of cement may be applied [50].

Concrete Cover, X

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

$$X_{\min} = X_N - 10 \tag{7}$$

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, however, 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. For all structural design, great care are always made to avoid any cracking of the concrete. Cracking of the concrete cover caused by corroding mounting steel may therefore represent the same type of weaknesses as that caused by any other types of concrete cracking. Instead of using mounting steel which can corrode, mounting systems based on non-corroding materials may be applied.

If it is assumed that 5% of the reinforcing steel has a concrete cover less than $X_{\min}$, the durability analysis can 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 can be quantified. For the documentation of achieved construction quality as described later, the durability analyses must always be based on the average observed value of concrete cover with standard deviation from the quality control carried out during concrete construction.

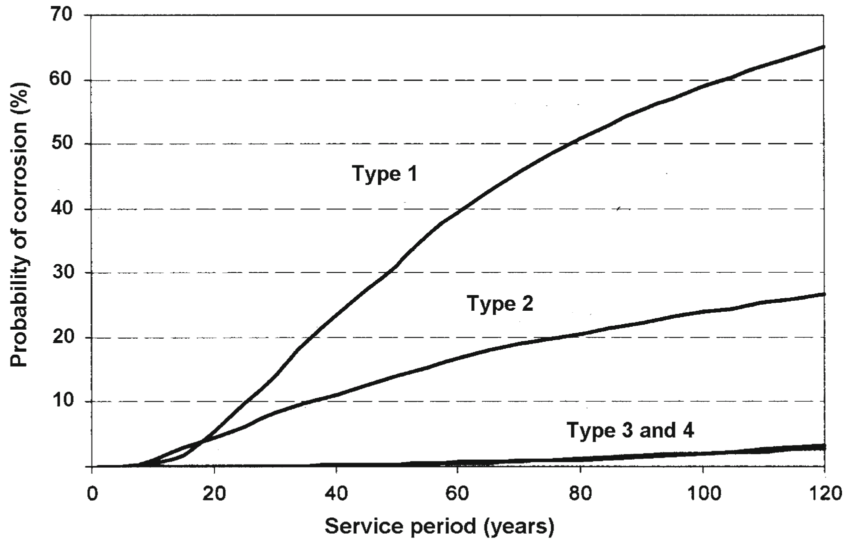

Fig. 4 Effect of binder system on the probability of corrosion

2.7 Durability Analysis

General

A durability design of a given concrete structure in a given environment can be carried out based on the procedures briefly described above. For important concrete structures, a service period of 120 years before 10% probability of corrosion is reached may be specified as an overall durability requirement. Always, however, the minimum durability requirements according to the current concrete codes must be fulfilled. Several durability analyses may be carried out as a basis for selecting a new and improved combination of concrete quality and concrete cover, some typical results of which are briefly demonstrated in the following.

Effect of Chloride Diffusivity

In order to find out how four different types of commercial cements in combination with 10% silica fume would affect the probability of corrosion for a new concrete structure in a typical Norwegian marine environment, four different durability analyses were carried out. These analyses were based on the 28 day chloride diffusivity obtained from the four types of concrete produced with the four different binder systems (Types 1–4). For the analyses, some estimated values were selected for the other input parameters for a severe marine environment as discussed above. Apart from type of binder system, the composition of all the concrete mixtures was the same, and this composition also met all durability requirements according to the current concrete codes for a 100 year service period both with respect to water/binder ratio (≤ 0.40) and binder content ($\geq 360\,\mathrm{kg/m^3}$). A minimum concrete cover of 70 mm was also adopted with a tolerance according to the current concrete code. In spite of this compliance with the current concrete codes, the results in Fig. 4 clearly demonstrate how different the obtained probabilities of corrosion for the various types of concrete would be. Thus, for the pure portland cement type of concrete (Type 1), a service period of only about 25 years would be obtained before 10% probability of corrosion would be reached, while the fly ash type of cement (Type 2) would increase this period up to about 40 years, and the two types of blast furnace slag cement (Types 3 and 4) would increase the service period of up to more than 120 years.

Effect of Concrete Cover

Further durability analyses based on increased concrete covers of up to 90 and 120 mm, respectively, were also carried out in order to evaluate the effect of increased nominal concrete cover beyond the minimum requirement of 70 mm used in the above analyses. For these analyses, all the other input parameters were the same as those used for the above analysis of Type 1 concrete. As clearly demonstrated in Fig. 5, increased concrete

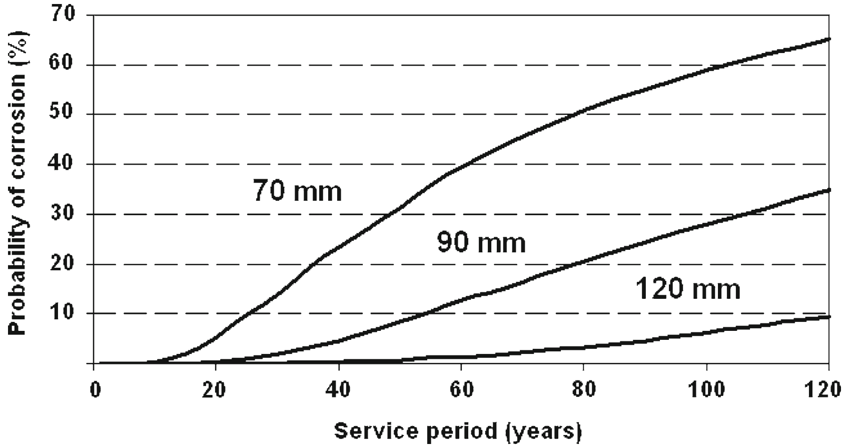

Fig. 5 Effect of increased nominal concrete cover on the probability of corrosion

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 give a service period of only about 25 years, increased concrete covers of up to 90 and 120 mm would increase the service period up to about 50 and 120 years, respectively, before 10% probability of 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 can be counteracted by a proper use of synthetic fibers added to the fresh concrete. However, by replacing the outer layer of the reinforcement system with stainless steel, an effective increase may be obtained equivalent to a concrete cover of both 120 mm and more for the black steel reinforcement. In this way, durability analyses also provide a proper basis for quantifying how much of the black steel reinforcement needs to be replaced by stainless steel in order to meet the required safety level against corrosion.

Results and Discussion of Results

As shown above, durability analyses can be used to compare and select one of several technical solutions in order to obtain the best possible durability for a given concrete structure in a given environment. For evaluation of the obtained results, however, it should be noted that a number of simplifications and assumptions were made in the above procedures for the calculation of corrosion probability. Although diffusion is the predominant transport mechanism through thick concrete covers in chloride containing environments, only a very simple diffusion model for the calculation of chloride penetration rates was applied. It should also be noted that the diffusion behavior of chloride ions in concrete is a much more complex transport process than that which can be described by Fick's Second Law of Diffusion. Under more realistic conditions in the field, other transport mechanisms for chloride penetration than pure diffusion also exist.

The characterization of the resistance of the concrete against chloride penetration was 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. However, the chloride diffusivity obtained by such an accelerated test method should be considered only as a relative index reflecting the ability of the concrete to resist chloride penetration in the same way as the compressive strength also is a relative index reflecting the ability of the concrete to resist mechanical loading. The durability analyses were further 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 (C_S) and the aging factors (α) 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% should not be considered as real service periods for the given concrete structure. Also, based on all the above simplifications, the current procedures for predicting probability of corrosion should never be used for service periods of more than 150 years. However, the above durability analyses provide a proper basis for an engineering judgement of the most important factors considered relevant for durability, including the scatter and variability of all factors involved. In this way one can obtain a proper basis for comparing and 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, a type of durability requirement can be specified which is also possible to verify and control in such a way that documentation of compliance with the specified durability can be obtained as briefly outlined and discussed in the following section.

3 Performance-Based Concrete Quality Control

3.1 General

Although to a certain extent a probability-based approach to durability design takes the great variability of construction quality into account, a numerical approach to durability design alone is not sufficient for ensuring proper durability. For concrete structures in severe environments, construction quality and variability are key issues which must be firmly grasped before a more rational approach to a more controlled durability can be achieved [51].

Even before the concrete is placed in the formwork, the quality of the concrete may show a high scatter and variability. Depending on a number of factors during concrete construction, the achieved quality of the finally placed concrete normally shows an even higher scatter and variability. Even for the offshore concrete platforms in the North Sea, where a very high quality of both concrete production and concrete construction was applied, most of the durability problems which have been experienced later on can be ascribed due to lack of proper quality control and special problems during concrete construction [1,52].

For production of air-entrained concrete, the problem with large variations in the air void characteristics during concrete construction is well known. During production, handling and placing of air-entrained concrete, the air void characteristics may vary within wide limits [53]. This problem may be enhanced when the concrete is produced with cements blended with fly ash having variable carbon content. As a result, not only the frost resistance but also other properties, such as the resistance of the concrete to chloride penetration, may be significantly affected.

Probably the best known and well documented quality issue of concrete structures is the failure to meet the specified requirements for concrete cover. In recent years, therefore, improved codes and procedures have been introduced for achieving the specified concrete cover with more confidence. Still, however, the variability of concrete cover appears 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 of the fresh 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 were occasionally removed during some critical stages of the slip forming in order to allow the slip forming work to proceed.

Compliance with the overall durability requirement for the given structure, requires proper quality control of both the specified chloride diffusivity and the concrete cover during concrete construction. For both of these durability parameters, achieved average values and standard deviations must be obtained. If cathodic prevention or preparation for such a protective measure has been specified, a regular quality control of the necessary electrical continuity within the rebar system must also be applied during construction. In the following, the procedures for measurements and control of the above durability parameters are briefly outlined and discussed.

3.2 Chloride Diffusivity

As already described above, all measurements of chloride diffusivity are based on the RCM method [44]. Although the duration of such measurements may take only a couple of days, this is not good enough for a 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 [54]:

$$D_i = \frac{R \cdot T}{Z^2 \cdot F^2} \cdot \frac{t_i}{\gamma_i \cdot c_i \cdot \rho} \tag{8}$$

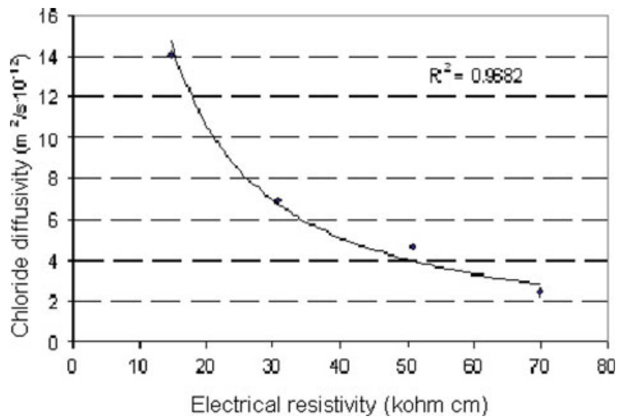

Fig. 6 A typical calibration curve for the control of chloride diffusivity based on measurements of electrical resistivity

where D_i is diffusivity for ion i, R is the gas constant, T is absolute temperature, Z is ionic valence, F is the Faraday constant, t_i is the transfer number of ion i, γ_i is the activity coefficient for ion i, c_i is the concentration of ion i in the pore water, and ρ is the electrical resistivity.

Since most of the factors in Eq. 8 are physical constants, the above relationship can be simplified for a given concrete with given temperature and moisture conditions to:

$$D = k \cdot \frac{1}{\rho} \tag{9}$$

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 very simple and rapid way, regular quality control of the electrical resistivity of the concrete provides the primary basis for indirect quality control of the chloride diffusivity during concrete construction [55]. 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 later used as a calibration curve for an indirect control of the chloride diffusivity based on regular measurements of the electrical resistivity during concrete construction (Fig. 6). Since the testing of electrical resistivity is also a non-destructive type of test, these measurements are carried out on the same concrete specimens as that being used for the regular quality control of the 28 day compressive strength during concrete construction.

3.3 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 conventional cover meters and procedures. It may also be difficult to apply conventional cover meters for control of concrete cover if the reinforcement is stainless steel, which does not respond to magnetic measurements. In such a case, cover meters based on a pulse-induction technique may be used. In both cases, however, extensive experience has shown that straight manual readings of the concrete cover on protruding bars in casting joints during concrete construction may provide a sufficiently accurate and proper basis for the regular quality control of the achieved concrete cover. However, the extent of measurements must be sufficient to give reliable statistical data both on average values and standard deviations.

4 Achieved Construction Quality

4.1 General

From the performance-based concrete quality control briefly described above, average values and standard deviation of both chloride diffusivity and concrete cover are obtained. Upon completion of the concrete construction work, therefore, these data are used as input parameters to a new durability analysis which documents compliance with the specified durability.

Because the specified chloride diffusivity is based only 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, this chloride diffusivity and the achieved concrete cover are used as input parameters for a new durability analysis and, hence, documentation of achieved durability on the construction site.

Since neither the 28 day chloride diffusivity from small laboratory 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 is also provided. Such a chloride diffusivity in combination with the achieved concrete cover provides the basis for documentation of the potential durability of the given structure.

Upon completion of the concrete structure, proper documentation of the achieved construction quality must be provided before the structure is formerly handed over from the contractor to the owner of the structure. For the owner, such documentation may have implications both for the future operation and the expected service life of the structure. In the following, some procedures for providing such documentation are briefly described and discussed.

4.2 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% has been specified. In order to show compliance with such a durability requirement, a new durability analysis must 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 other 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 scatter and variability observed. Hence, the new durability analysis provides a basis for the documentation of compliance with the specified durability.

4.3 Durability on Construction Site

Documentation of achieved chloride diffusivity on the construction site should preferably 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 too much, however, one or more un-reinforced concrete elements should be separately produced on the construction site, from which most of the concrete coring can take place during the construction period. In addition, a certain extent of coring from the real concrete structure should also be carried out, but only from locations where the coring does not weaken the concrete structure.

Separately produced concrete elements, which could be wall or slab types of elements, or both, should be produced and cured as representatively as possible for the real concrete structure or various parts of the concrete structure. From these separate dummy elements, which are produced at an early stage of concrete construction, a number of concrete cores are later on removed at various ages. Immediately upon removal these should be wrapped in plastic to avoid drying out and 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 various periods of up to at least 1 year. In addition to all the cores from the separately cast elements, some supplemental cores from the real concrete structure must also be tested during the construction period.

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 1 year. As an example, the observed development of achieved chloride diffusivity from one particular construction site is shown in Fig. 7. In this figure, the development of chloride diffusivity for the same type of concrete based on separately cast and water cured concrete specimens in the laboratory is also shown.

Based on the achieved chloride diffusivity on the construction site after approximately 1 year, a new durability analysis is carried out. In combination with the achieved data on concrete cover, this new durability

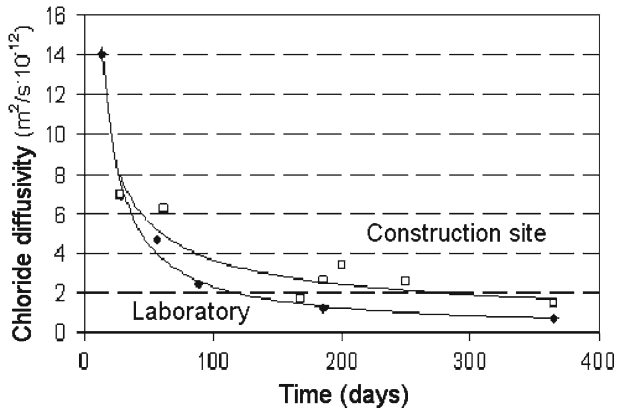

Fig. 7 Development of achieved chloride diffusivity on the construction site and in the laboratory during the construction period

analysis provides a basis for the documentation of achieved durability on the construction site during concrete construction.

4.4 Potential Durability

For establishing the necessary calibration curve, the chloride diffusivity is determined on separately cast concrete specimens after certain periods of water curing in the laboratory of up to approximately 60 days. By a continued testing of the chloride diffusivity on a few additional specimens after curing periods of up to at least 1 year, a further development of chloride diffusivity is obtained, as shown in Fig. 7. Although it may take a long time before a final and stable value of the chloride diffusivity is reached, this development curve for most types of concrete tends to level out after approximately 1 year. Hence, the observed chloride diffusivity after 1 year of water curing in the laboratory is used as an input parameter to a new durability analysis. In combination with the achieved data on the concrete cover, this analysis provides a basis for the documentation of the potential durability of the given concrete structure. Also for this new analysis, the other input parameters are the same as those used in the original durability analysis.

5 Condition Assessment and Preventive Measures

5.1 General

The typical situation during operation of most concrete structures is that maintenance and repairs are largely 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.

Recent years have seen rapid international development on general systems for life cycle management (LCM) of important infrastructure facilities [1,56–59]. 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.

However, for all important concrete structures in chloride containing environments, 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 durability design [60,61]. The following sections briefly describe some general procedures for monitoring and control of chloride penetration as a basis for the preventive maintenance.

5.2 Control of Chloride Penetration

Even if the strongest requirements both for concrete quality and concrete cover have been specified and achieved during concrete construction, extensive experience demonstrates that for concrete structures in chloride containing environments, a certain rate of chloride penetration will always take place during operation of the structure. As already discussed, 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 occasionally have already taken place during concrete construction before the concrete has gained sufficient curing and maturity [1]. For concrete structures where this is likely to occur, an early control of chloride penetration should always be required before the structure is handed over to the owner. However, an early control of chloride penetration should always be carried out in order to establish a reference level for the future control of chloride penetration, and hence provide the 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 the basis for the assessment of the future rates of chloride penetration. For the plan in question, it is also important to decide whether the future control of the chloride penetration should include any automatic readings from embedded probes or should be based only on manually observed chloride penetrations.

Since embedded probes provide information primarily on how fast the chloride front moves into the concrete, further information about the apparent chloride diffusivity (D_a) should also be provided at certain intervals during the service period (Fig. 3). In order to establish data on this apparent chloride diffusivity, manual measurements of the ongoing chloride penetration must be carried out from time to time.

From each control measurement of chloride penetration, a new apparent chloride diffusivity (D_a) is obtained, and the more new values for this parameter that become available, the better the basis for establishing a more reliable α-value for the time dependence of this apparent chloride diffusivity. As soon as two or more values for the apparent chloride diffusivity become available, an appropriate basis for establishing a reliable α-value for the given concrete structure in the given environment is obtained. Based on the obtained values for the surface chloride concentration (C_s), the apparent chloride diffusivity (D_a) and the time dependence factor (α), a new durability analysis is carried out. In combination with the previously observed data on concrete cover from the quality control during concrete construction, this analysis provides a basis for predicting the future probability of corrosion.

5.3 Protective Measures

Before the future probability of corrosion becomes too high, proper protective measures should be considered and selected [1]. Depending on the type of protective measure, the observed rate of chloride penetration can either be reduced or completely stopped. If the chlorides have not reached too deeply into the concrete cover, a proper surface treatment or coating may slow the rate of further chloride penetration. If the chlorides have already reached too deeply, however, cathodic prevention is the only protective measure that can effectively stop further chloride penetration and thus avoid development of steel corrosion.

6 Practical Applications

6.1 General

In recent years, a large number of new important concrete structures have been constructed in marine environments, and for most of them, the specified durability has typically been based on the minimum requirements according to current concrete codes. In order to obtain some further information about the quality of concrete typically applied to new concrete structures along the Norwegian coastline, samples of the concrete from some of these construction sites were collected in order to test the chloride diffusivity according to the RCM method [44]. Although all the various types of concrete generally fulfilled the specified durability requirements for a 100 year service period, with respect to both water/binder ratio ($\leq$0.40) and binder content ($\geq$360 kg/m^3), it

Table 3 Observed chloride diffusivity of the concrete applied for construction of some new concrete structures in Norwegian marine environments

Construction site	Chloride diffusivity ($\times 10^{-12}$ m^2/s) Time (days)									
	14	28	60	90	180	365	400	460	620	730
"Container Terminal 1" Oslo (2002)	13.5	6.0	4.4	3.8	3.0	–	–	–	–	–
"Gas 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	–	–
"Container Terminal 2" Oslo (2007)	14.0	6.9	4.6	2.4	1.2	0.7	–	–	–	0.7
"New City Development" Oslo (2005–)	4.7	1.6	0.4	0.4	0.3	0.2	–	–	0.16	–

can be seen from Table 3 that the chloride diffusivity of the 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 time of testing.

In order to obtain greater and more controlled durability and service life, some of these Norwegian concrete structures were also subjected to a durability design and concrete quality control as briefly outlined and described in the present paper. In the following, a brief summary of the experience gained with these practical applications is given. Most of these concrete structures were recently completed in the harbor region of Oslo City. One of the structures was a new container terminal completed in 2007, while the others were part of a new city development project which is still under construction. As a reference project, the achieved construction quality of a container terminal completed in 2002 is also briefly described, for which the specified durability was based only on then-current concrete codes and practice.

6.2 Container Terminal 1, Oslo (2002)

This concrete harbor structure was constructed with an open concrete deck on top of driven steel tubes filled with concrete. The structure was completed in 2002. At that time, the current procedures for durability design were not available. Therefore, the specified durability for the required 100 year service life was based only on then-current concrete codes and practice with the following specifications:

- Water/binder ratio: $\leq 0.40 \pm 0.03$
- Minimum cement content: $370\,\mathrm{kg/m^3}$
- Silica fume: 6–8% by wt. of cement
- Air content: $5.0 \pm 1.5\%$

As part of the durability requirements, a nominal concrete cover to the structural steel of 75 ± 15 mm was also specified.

Although no probability-based durability design was carried out, Oslo Harbor Authority as the owner of the structure wanted the best possible documentation of the 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 above was carried out.

6.3 Container Terminal 2, Oslo (2007)

During the period 2005–2007, another container terminal in Oslo Harbor was constructed. This structure, which has a waterfront of 650 m, also consists of an open concrete deck on top of driven steel tubes filled with concrete. According to the current concrete codes, the durability requirements for a 100 year service life primarily included a maximum water/binder ratio of 0.40, a minimum binder content of $330\,\mathrm{kg/m^3}$ and a minimum concrete cover of 60 mm. For ensuring proper frost resistance, a total air content of 4–6% was also required. In order to increase and better control safety against steel corrosion, however, the owner of the structure decided to carry out an additional durability design and concrete quality control as described in the present paper. The owner also decided that the new structure should have a service period of at least 100 years before 10% probability of corrosion would be reached.

Fig. 8 A new city development currently under construction in the harbor region of Oslo City

Fig. 9 A model section showing how the prefabricated concrete caissons provide large submerged parking areas in four levels

6.4 New City Development, Oslo (2005-)

In 2005, a new city development project ("Tjuvholmen") was started in the harbor region of Oslo City. This project includes a number of concrete substructures located in seawater with various depths of up to 20 m, on top of which a number of business and apartment buildings would be constructed as shown in Fig. 8. Most of these concrete substructures, which are still under construction, include large submerged parking areas. In the shallower water, these substructures include a solid concrete bottom slab on the sea bed surrounded by concrete walls partly protected by riprap 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, four large concrete caissons were constructed in a dry dock of a nearby shipyard. Upon completion, these prefabricated concrete units were moved into position and submerged in water to depths of up to 20 m. These structures provide large submerged parking areas in four levels as shown in Fig. 9.

At an early stage of planning, the developer of the project requested a best possible safety against corrosion of embedded steel for all the concrete substructures exposed to seawater. These concrete substructures should

Table 4 Specified and achieved durability based on probability of steel corrosion

	Specified durability	Compliance	Construction site	Potential
Container Terminal 1 (2002)	Current code (100 years)	–	After 100 years: Approx. 80%	After 100 years: Approx. 60%
Container Terminal 2 (2007)	After 100 years: Probability of steel corrosion $\leq 10\%$	Approx. 5%	After 100 years: Approx. 0.6%	After 100 years: Approx. 0.01%
New city development (2005−)	After 150 years: Probability of steel corrosion $\leq 10\%$	Approx. 0.1%[a] (Approx. 1–4%)[b]	After 150 years: Approx. 0.001 %[a] (Approx. 0.7%)[b]	After 150 years: $<0.001\%$[a] (Approx. 0.1 %)[b]

[a] Stainless steel
[b] Black steel

later form the basis for a large number of buildings representing huge investments. Based on the durability design as described above, the developer hoped to have a service period of 300 years.

As already discussed above, the current basis for calculating probability of corrosion for a service period of more than 150 years is considered neither valid nor relevant. As a basis for the durability design, therefore, all 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, not exceeding 10% within a service period of 150 years. In order to further ensure proper long term performance, preparation for future application of cathodic prevention was specified for the first structure. Later, however, a partial replacement of the black steel reinforcement with stainless steel in the outer layer of the most exposed parts of the structures was specified.

6.5 Results and Discussion of Results

An overall view of the specified and achieved durability for the various structures is shown in Table 4. As already discussed and emphasized above, the obtained "service periods" with a probability for corrosion of less than 10% should not be considered as real service periods for the given structures. However, the above durability analyses were carried out in order to provide a basis for an engineering judgement of the most important factors considered relevant for the durability of the structures, including the scatter and variability of all factors involved. Hence, a proper basis was obtained for comparing and selecting one of several technical solutions in order to obtain a best possible durability. During concrete construction, all the construction sites reported that the increased focus and attention on achieved construction quality resulted in very good workmanship; this was considered to be an important spin-off effect of the new procedures for durability design and concrete quality control.

For the first concrete structure (Container Terminal 1) which was based only on durability requirements according to current concrete codes and practice, a service period of approximately 30 years was obtained before 10% probability of corrosion would be reached. This was based on the use of a concrete quality with a 28 day chloride diffusivity of $6.0 \times 10^{-12}\ m^2/s$ (Table 4) in combination with a nominal concrete cover of 75 mm. Based on the achieved data from the construction site and from the observed chloride diffusivity in the laboratory after approximately half a year of curing, probabilities of about 80 and 60% for corrosion to occur after a 100 year service period were further obtained, respectively.

For the second concrete structure (Container Terminal 2), an overall durability requirement based on a 100 year service period before 10% probability of corrosion had been specified. In order to meet this specification, the concrete construction was based on a concrete quality with a 28 day chloride diffusivity of $6.9 \times 10^{-12}\ m^2/s$ (Table 4) in combination with a nominal concrete cover of 90 mm. Upon completion of this structure, a probability for corrosion of approximately 5% after a service period of 100 years was obtained, showing that the specified durability had been achieved with a proper margin. Based on data obtained from the construction site and the chloride diffusivity obtained in the laboratory after approximately 1 year of curing, probabilities of 0.6 and 0.02% for corrosion after a 100 year service period were also obtained, respectively. These results indicate that both the durability achieved on the construction site and the potential durability of the structure were very good.

For most of the concrete structures in the new city development project where a partial use of stainless steel reinforcement (W1.4301) was applied, a probability for corrosion of only 0.1% after a service period of 150 years was typically obtained, showing that the specified durability had been achieved with a very good margin. This result was obtained with concrete quality having a 28 day chloride diffusivity of 1.6×10^{-12} m^2/s (Table 4) in combination with a nominal concrete cover of 85 mm. Based on the achieved data from the construction site and the obtained chloride diffusivity in the laboratory after approximately 1 year, probabilities of less than 0.001% for corrosion to occur after a 150 year service period were further obtained, respectively. These results indicate that both the achieved durability on the construction site and the potential durability of the concrete structures were extremely good.

For the first concrete structure in this new city development project, only black steel in combination with preparation for future cathodic prevention was specified. As can be seen from Table 4, a probability of corrosion varying from 1 to 4% after a service period of 150 years was then obtained. Upon completion of this first structure, however, it was concluded that a partial use of stainless steel would have been both a simpler and more robust technical solution for ensuring the specified durability compared to that of future cathodic prevention. Even on a short-term basis, a partial use of stainless steel also proved to be economically competitive compared to the preparation for cathodic prevention. As a result, a partial use of stainless steel for all further concrete structures was specified.

7 Concluding Remarks

In recent years, an extensive amount of research has been carried out in order to better understand and control several of the most important deterioration mechanisms for concrete structures in severe environments, and never before has so much basic information and knowledge about concrete durability been available. The great challenge to the profession is, therefore, to utilize and transform more of all this existing knowledge into good and appropriate engineering practice.

Extensive experience demonstrates that the durability of concrete structures is related not only to design and material but also to the execution of the concrete construction work and the achieved construction quality. Upon completion of new concrete structures, the construction quality achieved always shows a high scatter and variability, and in severe environments, any weaknesses in the concrete structures will soon be revealed, whatever specifications and constituent materials have been applied. In order to better take all this variability into account, a probability approach to the durability design as outlined and discussed in the present paper should be applied. Since many of the durability problems also can be attributed to poor 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. Therefore, a performance-based concrete quality control during concrete construction with proper documentation of achieved construction quality as also outlined and discussed in the present paper should be applied.

When the concrete structure is completed, the owner should further be provided with a proper service manual for the future operation of the structure. Only such a service manual for condition assessment and preventive maintenance, as briefly outlined in the present paper, can provide the ultimate basis for achieving a more controlled durability and service life of the structure.

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