MODELS FOR ENVIRONMENTAL ACTIONS ON
REINFORCED CONCRETE STRUCTURES – CONSEQUENCES
FOR THE INITIATION OF CHLORIDE INDUCED
REINFORCEMENT CORROSION

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Abstract
In the current paper the influence from the exposure conditions on the initiation of chloride induced reinforcement corrosion is described, modelled and quantified. The focus is on environments where chlorides are present, i.e. marine environments and along roads where de-icing salt is spread. The environmental actions on concrete structures in general and structures in environments where chlorides are present in particular are described in the paper.

Models for environmental actions on concrete structures, subjected to chloride-induced reinforcement corrosion are presented in the paper. The models also consider the influence from some concrete properties (following from concrete composition and curing conditions). The principal with the models is to measure concrete properties in the laboratory and transfer them to the actual exposure conditions with a number of sub parameters. As a first approach these sub parameters have been assumed uncorrelated.

The consequences of the models for environmental actions are exemplified with service life predictions, where the influence from the exposure conditions and some concrete properties are described with the models. The sub parameters in the models are quantified with data from literature and field studies. The results show that the exposure conditions have large influence on the service life predictions.

1. Introduction

There is a need to more accurately predict the service life of reinforced concrete structures, where the design of the structures is optimized with respect to durability. With more accurate predictions of the service life it is also possible to better plan for future maintenance and repair. The service life of a reinforced concrete structure can be defined as the period during which the performance of the structure is fulfilled. Depending on the nature of the required performance three types of service life can be defined, namely technical, functional and
economic service life, cf. [1]. In this paper only the technical service life, i.e. the time in service until a defined unacceptable state of deterioration is reached, is considered.

The technical service life of reinforced concrete structures is to large extent dependent on the deterioration of the concrete and/or the reinforcement. The deterioration is a time dependent process, which takes place as a number of chemical, electro-chemical and physical processes, e.g. chloride ingress, carbonation or freeze/thaw actions. A common way to model the service life of reinforced concrete structures is to divide it into periods. The most common way is to divide into two periods, defined as “initiation” and “propagation” periods, which was first proposed by Tuutti [2]. In this paper only the initiation period of chloride induced reinforcement corrosion is treated.

The deterioration of concrete can be described as interactions of what takes place at both the surface and inside the concrete. The interactions are governed by the transport properties of the concrete, exposure conditions etc. Three main factors can be identified:

- **Material properties.** The material properties can be described as a combination of the properties of the constituent materials in the concrete, e.g. chemical composition and fineness of binder and aggregates etc.
- **Workmanship during construction.** The workmanship during construction includes factors related to the construction of the structure, e.g. compaction and curing of the concrete.
- **Exposure conditions.** The exposure conditions include moisture and temperature conditions and the exposure to carbon dioxide and chlorides.

Until now large research efforts have been made to clarify what influences the material properties and to some extent workmanship during construction have on the performance of reinforced concrete structures. However, there is little reliable information available about the influences from the exposure conditions. This is why the emphasis in a recently published study by Lindvall [3], has been put on how environmental actions influence the service life of reinforced concrete structures and how this influence can be modelled. In this paper a short review of the study is given, where environmental actions on concrete structures are described and modelled and the consequences for the service life are exemplified. The focus has been on the exposure conditions for reinforced concrete structures exposed to chlorides in marine environments.

### 2. Environmental Actions and Response from Concrete Structures

#### 2.1 General

The environmental actions on structures are mostly determined from the processes in the atmosphere, i.e. air temperature, air moisture, air pressure and air flow (wind). The main driving force behind these processes is the solar radiation, which heats up the atmosphere and the surface of the earth. The general approach when describing and modelling the environmental actions on a structure is to separate the actions from the response from the structure. With this approach the environmental actions can be described without being dependant of the properties of the structure. However, this is not always possible since the environmental actions sometimes are influenced by the properties of and response from the structure.
In the following first general descriptions of environmental actions on concrete structures and the response from the concrete are given. After that environments, where chlorides are present, i.e. marine and road conditions, are briefly described.

2.2 Environmental Actions

When the environmental actions are described it is convenient to classify them. The classification can be based on for example radiation balance, the soil-water balance, temperature, precipitation etc. Here a division is made into four levels, depending on geographical scale, cf. figure 1, modified from [4]:

- **Global conditions.** The environmental conditions for a larger area, e.g. a part of a country. Data on the environmental actions can be achieved from a nearby meteorological station.
- **Regional conditions.** The environmental conditions for the area surrounding a specific structure. The regional conditions can be achieved from the global conditions by also taking the influence from the surroundings of the structure into account.
- **Local conditions.** The environmental conditions for different parts of a structure. The local conditions follow from the regional conditions with the influence from the large-scale geometry and orientation of the structure taken into account. A difference is made between different parts of the structure, due to surface orientation etc.
- **Near surface and surface conditions.** The near surface and surface conditions follows from the local conditions but the shape of the structure and the texture and material properties are also taken into account.

![Figure 1: Division of the exposure conditions into different geographical scales and influencing factors.](image-url)
The most accurate way to describe the environmental actions is to express them as surface conditions. The surface conditions can be expressed in terms of an equivalent surface temperature and humidity, moisture conditions and concentrations of other substances (e.g. carbon dioxide, chlorides etc). The equivalent surface temperature and humidity include effects from radiation, evaporation/condensation, heat transfer and material and surface properties. The moisture conditions can be expressed as time of wetness (TOW), which is the time when the surface is wet due to precipitation, surface condensation, i.e. when the equivalent surface humidity exceeds 100%RH, and running water. The concentration of chlorides can be determined as equivalent surface concentrations, where also effects from position, orientation and moisture conditions are included. The concentration of carbon dioxide can be assumed constant in almost all environments, except in confined spaces, e.g. tunnels, where increased concentrations of carbon dioxide may occur.

The environmental actions on concrete structures and how they can be modelled is further described in for example [3, 5 and 6].

2.3 Response

The environmental actions acting on the surface of a concrete structure cause a response from the concrete. The response from concrete structures can be quantified in terms of temperature and moisture conditions, carbonation depths and chloride ingress etc. Since it is difficult to directly measure the environmental actions, e.g. exposure to chlorides, usually the response from the concrete is used as a measure of the environmental actions. However, since the response is not only influenced by the environmental actions but also the concrete composition and workmanship during construction these parameters will also influence the measured response. This means it can be difficult to directly compare environmental response measured in different concretes, where the result may be large uncertainties, cf. [7].

The response and in which way it influences the degradation and properties of the concrete is further described in for example [3, 5 and 6].

2.4 Marine conditions

Marine conditions are characterized by vicinity to an ocean or sea, including coastal areas. Coastal areas are influenced some kilometres from the coastline, due to wind-blown salt mist. However, on special occasions, e.g. during severe storms, the area influenced by salt from the sea may reach several 100 km from the coastline, [8].

Usually marine conditions are divided into four different exposure zones, [9], depending on position to the water surface and tidal and wave actions. The following exposure zones can be identified:

- **Submerged zone.** The submerged zone is the zone below the water level, where the surface of the concrete is constantly exposed to seawater.

- **Tidal zone.** The tidal zone is the zone between low and high tide, which means that the concrete is subjected to an alternating moist and dry environment (with an approximately 12 hour cycle). The surface of the concrete is cyclically exposed to seawater or air corresponding to the tidal cycle.
Splash zone. The splash zone is the zone above the tide level influenced by the waves, which means that the concrete is subjected to randomly alternating moist and dry environments due to wave action. The surface of the concrete is randomly exposed to splash from seawater or air.

Atmospheric zone. The atmospheric zone is the zone above the splash zone and the concrete is exposed to humid marine air with airborne chlorides that follows spray from breaking waves.

The exposure to chlorides in marine conditions depends to large extent on the position to the water surface. Below the water surface the exposure conditions are fairly trivial and depend mainly on the characteristics of the seawater (chemical composition and temperature). However, above the water surface the exposure conditions are more complicated and depend for example on extension of tidal and wave actions, exposure/shelter to rain and movements of air streams around the structure.

2.5 Road conditions

As for most outdoor environments, the road conditions are characterized by rapid changes in temperature and moisture conditions and exposure to carbon dioxide. However, what distinguishes road conditions from other outdoor environments is the exposure of chlorides originating from de-icing salt spread to prevent slippery road conditions. However, in this paper the focus has been on the exposure conditions for marine structures and therefore road conditions are not further treated. More information about the exposure conditions along thaw-salted roads is given in [3].

3. Models for environmental actions on concrete

3.1 General

The mathematical models presented in this paragraph describe the environmental actions on surfaces of reinforced concrete structures. The focus has been to describe the environmental actions on reinforced concrete structures subjected to chloride-induced reinforcement corrosion. The formulation of the models of environmental actions depends on the choice of prediction model, where in this paper the DuraCrete chloride ingress model, [10], has been chosen as an example model. This model is based on the error-function solution to Fick’s 2nd law and the principal appearance of the DuraCrete chloride ingress model is given in eq. (1a) and (1b).

\[
C_x = C_{SN} \cdot \left( \frac{x}{2 \cdot \sqrt{D_x(t) \cdot t}} \right) \quad \text{[\% by weight Cl/binder]} \quad (1a)
\]

where:

- \(C_x\): the chloride content at a certain penetration depth. [weight-% per binder]
- \(C_{SN}\): the apparent surface chloride content. [weight-% per binder]
- \(x\): the depth at which the chloride content in the concrete is determined. [m]
- \(t\): the exposure time. [s]
D_n(t): the apparent diffusion coefficient for chlorides, see eq. (1b). This parameter can be evaluated by curve fitting the error function solution of Fick’s 2nd law to measured chloride ingress profiles. [m²/s]

\[
D_n(t) = D_n(t_0) \left( \frac{t_0}{t} \right)^n = k_c \cdot k_e \cdot k_i \cdot D_0 \left( \frac{t_0}{t} \right)^n \quad [\text{m}^2/\text{s}]
\]

(1b)

where:
- \(k_c\): constant parameter that accounts for the influence of workmanship. [-]
- \(k_e\): constant parameter that accounts for the influence of environment. [-]
- \(k_i\): constant parameter that accounts for the influence of test method. [-]
- \(D_0\): potential chloride diffusion coefficient, determined at standardized conditions, with a standardized method, e.g. NT Build 492 [13]. [m²/s]
- \(t_0\): reference time, i.e. the time after casting at which \(D_0\) is measured. [s]
- \(n\): age factor. [-]

The DuraCrete chloride ingress model agrees with models proposed by for example Poulsen and Mejlbø, described in e.g. [11 and 12]. This means that the models for environmental actions presented in this chapter can be used for these prediction models as well.

3.2 Models for environmental actions

In the original DuraCrete chloride ingress model, shown in eq. (1a) and (1b), the influence from the exposure conditions is only described with three parameters, namely \(C_{SN}\), \(k_c\) and \(n\). All effects due to chloride, moisture and temperature conditions are combined in these parameters, which mean it is difficult to separate the effects of different influencing factors on a specific parameter. Therefore it is in [3] proposed to modify the modelling of \(C_{SN}\) and \(D_n(t)\) from the original DuraCrete model, according to eq. (2a) and (2b). \(C_{SN}\) is modelled with an equivalent surface chloride content, \(C_{SN,eq}\), measured in equivalent conditions, which is transferred to the actual exposure conditions with a number of parameters. \(D_n(t)\) is modelled with a potential diffusion coefficient, \(D_0\), measured in standardized conditions in the laboratory, which is transferred to the actual exposure conditions with a number of parameters. To better model the influence of exposure conditions it is proposed to further subdivide the parameters \(k_{C,e}\) and \(k_{D,e}\) with a number of sub parameters, according to eq. (2a) and (2b). All sub parameters in eq. (2a) and (2b) have, as a first approach, been assumed uncorrelated.

\[
\begin{align*}
C_{SN} &= C_{SN,eq} \cdot k_{C,conc} \cdot k_{C,e} \cdot k_{C,test} \\
k_{C,e} &= k_{C,CI} \cdot k_{C,d} \cdot k_{C,h} \cdot k_{C,o} \cdot k_{C,T}
\end{align*}
\]

(2a)

where:
- \(C_{SN}\): surface chloride content. [% by weight Cl/binder]
- \(C_{SN,eq}\): \(C_{SN}\) measured in equivalent conditions and concrete quality. \(C_{SN,eq}\) is in marine conditions determined in the submerged zone at 1.0 m depth (chloride concentration and temperature of 20g Cl/l and 20°C respectively) and in road
conditions at the road surface (vertical surface at 0.0 m distance and height for surfaces facing towards the traffic). [% by weight Cl/binder]

\( k_{C,\text{conc}} \): parameter that accounts for the influence the concrete composition has on C\(_{SN}\). [-]

\( k_{C,\text{test}} \): parameter that accounts for the influence of the test method. Here this parameter is put to \( k_{C,\text{test}}=1.0 \). [-]

\( k_{C,e} \): parameter that accounts for the influence of the exposure environment. To better model the influence of the exposure environment the parameter \( k_{C,e} \) is subdivided into the following \textit{uncorrelated} parameters, \( k_{C,\text{Cl}}, k_{C,d}, k_{C,h}, k_{C,o} \) and \( k_{C,T} \). [-]

\( k_{C,\text{Cl}} \): parameter that accounts for the influence of chloride concentration in marine submerged conditions on C\(_{SN}\) (if chloride concentration is other than 20 g/l). [-]

\( k_{C,d} \): parameter that accounts for the horizontal distance to the source of chlorides. [-]

\( k_{C,h} \): parameter that accounts for the vertical distance to the source of chlorides. [-]

\( k_{C,o} \): parameter that accounts for the orientation towards the source of chlorides. [-]

\( k_{C,T} \): parameter that accounts for the influence of temperature on C\(_{SN}\) (if temperature is other than 20\(^\circ\)C). [-]

\[
\begin{align*}
D_a(t) &= D_0 \cdot k_{D,e} \cdot k_{D,\text{curing}} \cdot k_{D,\text{test}} \cdot \left( \frac{t_0}{t} \right)^n \\
k_{D,e} &= k_{D,\text{RH}} \cdot k_{D,T}
\end{align*}
\] (2b)

where

\( D_a(t) \): apparent diffusion coefficient after a specific exposure time, \( t \). [m\(^2\)/s]

\( D_0 \): potential diffusion coefficient measured in standardised conditions in the laboratory. [m\(^2\)/s]

\( k_{D,e} \): parameter that accounts for the curing conditions. [-]

\( k_{D,\text{test}} \): parameter that accounts for the influence of test method. This parameter is equal to \( k \) in eq. (1b) but is for clarity written as \( k_{D,\text{test}} \) in eq (2b). Here \( k_{D,\text{test}} \) is put to 1.0. Examples of how \( k_{D,\text{test}} \) can be quantified are given in [10]. [-]

\( k_{D,e} \): parameter that accounts for the environmental conditions. The parameter \( k_{D,e} \) is proposed to be subdivided into two \textit{uncorrelated} parameters giving the influence of the RH, \( k_{D,\text{RH}} \) and the temperature, \( k_{D,T} \), respectively. [-]

\( t_0 \): age at which \( D_0 \) is measured (concrete cured in laboratory in standardised conditions). [s]

\( t \): exposure time. [s]

\( n \): age factor, giving the decrease in \( D_a \) with time, being partly an environmental parameter. [-]

In the following paragraphs examples are given of how some of the sub parameters can be quantified. A more thorough description of the parameters and how they can be quantified is given in [3].
3.3 Influence from concrete properties and chloride conditions

The influence of the concrete properties and chloride conditions is in eq. (2a) and (2b) modelled with $C_{SN,eq}$, $k_{C,Cl}$, $D_0$, $k_{C,con}$ and $k_{D,c}$. The equivalent surface chloride content, $C_{SN,eq}$, is influenced by a combination of the concrete properties and the exposure conditions (mainly chloride conditions), where $C_{SN,eq}$ is different for marine and road conditions. However, if the same concrete composition is exposed in both marine and road conditions it is also possible to transfer $C_{SN,eq}$ from marine to road conditions and vice versa. The potential diffusion coefficient, $D_0$, gives mainly the influence from the concrete composition. In DuraCrete it is proposed to measure $D_0$ with NT Build 492 [13].

In the following an example is given of how $C_{SN,eq}$ and $k_{C,Cl}$ in marine conditions can be quantified. Examples of how the other parameters can be quantified are given in [3].

$C_{SN,eq}$ is evaluated from chloride ingress profiles measured in concrete exposed in equivalent exposure conditions. In marine exposure conditions the equivalent conditions are at 1.0 m depth in the submerged zone (with a chloride concentration of 20 g/l and a temperature of $+20^\circ$C), since this zone provides a well defined exposure. In road conditions the equivalent conditions are at 0.0 m distance from and height above the carriageway and facing towards the traffic. An example of how $C_{SN,eq}$ and $k_{C,Cl}$ can be determined in marine conditions is given in figure 2. In this figure data on $C_{SN}$ evaluated from measured chloride ingress profiles from one concrete composition exposed in different marine submerged environments (different chloride concentrations and temperatures of the seawater), are presented. The exposure time has been approximately one year for all examined concretes.

![Figure 2: An example of how $C_{SN,eq}$ and $k_{C,Cl}$ can be determined from data evaluated chloride ingress profiles. Based on data from [3]](image-url)
From figure 2 it can be seen that $C_{SN,eq}$ is equal to 1.7 [% by wt Cl/binder] (20 g/l chloride concentration and +20°C temperature). Additionally $k_{C,Cl}$ can be quantified from the data presented in figure 6.1a, by dividing $C_{SN}$ by $C_{SN,eq}$ at a water temperature of 20°C. With this procedure $k_{C,Cl}$ has been quantified to the following values: 0.76 (5 g/l chloride concentration), 0.88 (10 g/l chloride concentration) and 1.12 (25 g/l chloride concentration).

In road conditions the quantification of $C_{SN,eq}$ is more complicated, since the equivalent conditions are defined as a concrete surface at the road surface (vertical surface at 0.0 m distance and height for surfaces facing towards the traffic). With this definition it is possible to correlate $C_{SN}$ with the amount of de-icing salt spread on the road. However, since there are no data available on $C_{SN}$ for concretes exposed at the road surface chloride ingress data from other positions need to be recalculated, with eq. (2a). An example of this is given in [3].

### 3.4 Influence from exposure conditions

#### General

The influence from the exposure conditions in modelled with $k_{C,T}$, $k_{D,RH}$, $k_{D,T}$, $n$, $k_{C,h}$, $k_{C,a}$ and $k_{C,o}$. In eq. (1b) $D_a(t)$ is modelled as a time-dependant parameter, where $D_a(t)$ decreases with increasing exposure time. This decrease is modelled with an age factor, $n$, which gives the combined influence from the concrete properties and the exposure conditions.

In the following examples are given of how $k_{C,T}$, $k_{D,T}$, $n$ in marine conditions. Examples of how the other parameters can be quantified are given in [3].

#### Effect of temperature in the concrete – $k_{C,T}$ and $k_{D,T}$

The effect from the temperature in the concrete on the apparent surface chloride content, $C_{SN}$, and chloride diffusion coefficient, $D_a(t)$ have been modelled with $k_{C,T}$ and $k_{D,T}$. An example of how $k_{C,T}$ and $k_{D,T}$ can be quantified is given in figure 3, where the temperature dependency is modelled with an Arrhenius relationship.
In figure 3 it can be seen that $k_{C,T}$ and $k_{D,T}$ are equal to 1.0 when the concrete temperature is +20°C. If the concrete temperature increases $k_{C,T}$ decreases and $k_{D,T}$ increases, i.e. the driving potential for chloride transport gets lower while the chloride transport properties in the concrete increase. The effect is opposite if the temperature decreases.

**Effect from age of concrete – n**

An important factor that influences the chloride ingress into concrete is how the chloride transport properties in concrete change over time. In eq. (1b), the change in chloride transport properties in the concrete over time has been modelled with an age factor, $n$. The age factor can be determined from $D_a$ evaluated from chloride ingress data measured from the same concrete after different exposure times. The evaluated $D_a$ after different exposure times, is plotted as a function of the exposure time in a log-log scale and the age factor is evaluated as the slope of a linear trend line for the data. This is shown in figure 4, where chloride ingress data from two concretes (concrete 1-40 and 1-50, Swedish SRPC1-concretes with w/b=0.40 and 0.50 respectively) exposed in the marine atmospheric and submerged zones have been used for the quantification. The diamonds and squares denote data from concrete 1-40 and 1-50 and the filled and unfilled symbols exposure in submerged and atmospheric conditions respectively.

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1 Swedish SRPC – Degerhamn Anläggningscement CEM I 52.5 BV/SR/LA.
Figure 4: An example of how the age factor n can be quantified. Based on data evaluated from measured chloride ingress presented in [14].

From figure 4 it can be seen that the age factors (in submerged and atmospheric exposure respectively) are equal to 0.21 and 0.54 (concrete 1-40) and 0.55 and 0.35 (concrete 1-50). However, the age factors evaluated from figure 4 have large uncertainties since they are quantified from data measured on three to five different occasions up to approximately 10 years exposure (concrete 1-40 in submerged conditions). This means that each data point has a large influence on the age factor, e.g. if the data point after 10 years exposure for concrete 1-40 were unknown the age factor would be significantly different (n=0.59 instead of n=0.21). Obviously more data are needed to be able to quantify age factors with less uncertainty. Furthermore from the results in figure 4 it is obvious that the age factor is influenced not only by the exposure conditions but also by the concrete composition. However, chloride ingress data measured at different exposure times for the same concrete are scarce and therefore chloride ingress data from different concrete compositions and exposure times are usually combined when the age factor is quantified. The result of the quantification may be a large scatter as a result of differences in concrete compositions, analysis methods etc, cf. [7].

A factor that influences the results of the quantification of the age factor is the definition of the exposure time. This is a relevant question, since it is possible to define the exposure time either as the total time of exposure or the time of exposure to chlorides. Additionally the chloride ingress in marine atmospheric and road conditions has been found to almost stop after a certain exposure time, cf. [15-17]. In road conditions the exposure to chlorides only takes place when de-icing salt is spread, i.e. approximately half the year, which means that the time of exposure to chlorides is approximately only half of the total exposure time. The exposure to chlorides may be even shorter at locations where de-icing salt is rarely spread. Thus, depending on when during the year the exposure in road conditions is started, the difference between the total time of exposure and exposure to chlorides can be up to half a year. Additionally, the maturity of
the concrete on first exposure to chlorides has been found to have a significant influence on the chloride ingress, especially for concretes with secondary binders (silica fume, fly ash etc), cf. [17].

4. Environmental actions in models of service life

4.1 General

The influence of the environmental actions on predictions of service life has been demonstrated with the DuraCrete chloride ingress model, shown in eq. (1a) and (1b). As mentioned earlier the service life is defined to be ended when reinforcement corrosion has been initiated. The DuraCrete chloride ingress model predicts the total chloride content in the concrete, which means that the driving potential for chlorides, \( C_{SN} \), and the chloride threshold level, \( C_{crit} \), is described as the total chloride content in the concrete. The service life predictions have only been made for concrete exposed in the marine submerged zone.

The chloride ingress into concrete is predicted, both with deterministic and probabilistic methods. The results from these predictions are used for service life predictions, where the concrete covers required to achieve a certain service life have been determined and vice versa. The probabilistic predictions have been made with parameters described with means values, standard deviations (SD) and statistical distribution functions. The calculations have been made with the computer program COMREL\(^2\). All calculations have been made according to FORM (First Order Reliability Methods). Only sound concrete, i.e. without any defects like cracks etc, has been considered.

4.2 Service life predictions

General

The service life predictions have been made for a Swedish SRPC-concrete with \( w/b=0.40 \), exposed in marine submerged conditions. To exemplify the influence from the exposure conditions nine different exposure locations have been studied (with different chloride concentrations and temperatures of the seawater): (1) The Mediterranean, (2) The Baltic Sea, (3) Öresund, (4) Kattegat, (5) North Sea, (6) Norwegian Fjord, (7) Atlantic Ocean around Iceland, (8) Atlantic Ocean outside Iberian Peninsula and (9) Persian Gulf. The yearly mean chloride concentrations and temperatures in the seawater at these locations vary between 3.6 g Cl/l and 23.0 g Cl/l and 7.3°C and 26.8°C respectively.

A factor that has a decisive influence on the service life of reinforced concrete structures is the chloride threshold level, \( C_{crit} \), i.e. the chloride content at the reinforcement at which reinforcement corrosion is initiated. In the literature \( C_{crit} \) has been found to vary between 0.097-3.04 [% by weight Cl/binder], according to a survey presented in [18]. There are many factors that influence \( C_{crit} \), e.g. concrete properties, carbonation, moisture and temperature conditions in the concrete, cracks, cast in chlorides, cover thickness, exposure time etc. However, the effects of these factors on \( C_{crit} \) have not been studied in this study. Instead data

\(^{2}\) Computer program for probabilistic calculations, programmed by STRUREL. More information about COMREL is available at the website of STRUREL: http://www.struruel.de
on $C_{\text{crit}}$ has been taken from [10, 19 and 20]. Based on these references the following $C_{\text{crit}}$ is used in the service life predictions: $C_{\text{crit}} = 2.00$ [% by weight Cl/binder].

The age factor $n$ has been quantified based on data from [10 and 20] to $n = 0.30$ (Beta distributed with SD=0.12). The driving potential for chloride, $C_{\text{SN}}$, has been quantified based on the data on chloride concentration and temperature of the seawater, where $C_{\text{SN}}$ varies between 1.33 and 2.79 [% by wt Cl/binder], [3]. $C_{\text{SN}}$ was found to be mainly influenced by the temperature of the seawater, where $C_{\text{SN}}$ decreases with increasing temperature and vice versa, and to less extent by the chloride concentration in the seawater. The quantification of $D_0$, $k_{D,c}$, $k_{D,\text{test}}$ and $t_0$, i.e. from the concrete properties on $D_a(t)$, have been taken from [10 and 21].

**Deterministic predications**

Based on the data presented above chloride ingress profiles have been predicted for an exposure time of 100 years. The results from the predictions with deterministic methods are shown in figure 5 together with data on $C_{\text{crit}}$. The predictions have been made for nine different locations and each location is denoted by its number according to what is earlier presented.

In figure 5 the required concrete covers to achieve a service life of 100 years ($C_{\text{crit}} = 2.00$ [% by wt Cl/binder]) are between 34.0 mm (location 7) and 0.0 mm (location 1, 3, 8 and 9). The required concrete covers of 0.0 mm are caused by the fact that $C_{\text{SN}} < C_{\text{crit}}$, i.e. there will never be any initiation of reinforcement corrosion. This effect can mainly be attributed to the fact that $C_{\text{SN}}$ decreases, while $C_{\text{crit}}$ is constant, with increasing temperature of the seawater. This will be further discussed in chapter 5 “Analysis and discussion”.

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**Figure 5:** Predicted chloride ingress profiles for 100 years’ exposure time in marine submerged conditions.
Probabilistic predictions

In the probabilistic calculations the service life is defined to be ended when the acceptable minimum reliability, i.e. the target reliability, is exceeded. The acceptable minimum reliability depends on the cost of minimising the probability that the limit state is exceeded, in this study the risk of initiation of reinforcement corrosion, in relation to the cost of repairing the structure. For the sake of clarity the reliability is expressed in terms of a so-called reliability index, $\beta$. Depending on the relation between these costs, $\beta$ in a serviceability limit state may vary between 1.0 and 2.0, which corresponds to failure probabilities of 15.866% and 2.275% respectively, [22]. Normally a reliability index of 2.0 is used to define the end of the service life for a serviceability limit state, e.g. initiation of reinforcement corrosion. This corresponds to a case where the cost of minimising the risk that the limit state is exceeded is low and the cost of repairing the structure is high. Trend lines showing $\beta=1.0$ and 2.0 are included in figure 6, where the results of the probabilistic calculations are presented.

The results of the probabilistic calculations are presented in figure 6, where the reliability, in terms of $\beta$, is expressed as a function of the exposure time. The concrete cover, $d_c$, has been set to 50 mm (Beta distributed with SD=5 mm) in the calculations. For clarity the reliability has only been determined for six different locations, where the minimum and maximum chloride ingress depths are expected (cf. figure 5). The calculations have been made with $C_{\text{crit}}=2.00$ [% by wt Cl/binder]. To illustrate the effect of $C_{\text{crit}}=0.60$, a calculation has also been made with this $C_{\text{crit}}$ for location 4.

![Figure 6: Reliability as a function of the exposure time (marine submerged conditions). Effect of chloride concentration and temperature of seawater and level of $C_{\text{crit}}$.](image)

In figure 6 the predictions of service life, with $\beta=2.0$ ($C_{\text{crit}}=2.00$ [% by wt Cl/binder] and $d_c=50$ [mm]) are between 18 years (location 7) and >100 years (location 9). With $\beta=1.0$, the
predicted service life will be considerably longer, e.g. 50 years for location 7. The large variations in calculated $\beta$ presented in figure 6 can mainly be attributed to large variations in $C_{SN}$ between the different locations, while $C_{crit}$ is constant. This can for example explain that the predicted service life for exposure in the Persian Gulf (location 9) is extremely high (1817 years with $\beta=2.0$).

5. Analysis and discussion

In this chapter the results of the predictions of chloride ingress and service life are examined. The following will be analysed and discussed: (i) effect of chloride and temperature conditions in the seawater and (ii) results from probabilistic calculations.

5.1 Effect of chloride and temperature conditions in the seawater

The effect of the temperature of the seawater can be investigated if the predictions of chloride ingress made at locations 3, 1, 5, 6, 7 and 8 are compared. At these locations the chloride concentration in the seawater is fairly similar, while the yearly mean temperature of the seawater varies. From figure 5 it can be observed that the temperature of the seawater significantly influences both the shape and level of the predicted chloride ingress profiles. The profiles get flatter, due to higher $D_a(t)$, and the level of the profiles gets lower, due to lower $C_{SN}$ with increasing temperature.

The temperature effect on $C_{SN}$, if $C_{SN}$ is defined as the total chloride content, can be explained by a temperature effect on the chloride binding, e.g. as described by Larsen [23]. Since $C_{crit}$ and $C_{SN}$ are defined as the chloride content in the concrete, it seems reasonable that there is a temperature effect on $C_{crit}$ as well. However, depending on how $C_{SN}$ and $C_{crit}$ are defined (as free or total chloride content in the concrete) the temperature effect looks different. If $C_{SN}$ is expressed as the total chloride content in the concrete it should decrease with increasing temperature, assuming that the free chloride content is constant. $C_{crit}$ (expressed as total chloride content in the concrete) should have a similar temperature dependence as $C_{SN}$, i.e. $C_{crit}$ decreases with increasing temperature and vice versa. This means that the required concrete covers would increase with increasing temperature, which also seems more realistic. However, in the literature data on the influence of temperature on $C_{crit}$ are scarce and therefore more research is needed to further clarify this important influencing factor.

The effect of the chloride concentration of the seawater can be investigated if the predictions of chloride ingress made at locations 2, 3, 4, 5, 8 and 9 are compared. At these locations the temperature of the seawater is fairly similar (locations 2, 4 and 5 + locations 8 and 9), while the chloride concentration in the seawater varies. From figure 5 it can be observed that the chloride concentration of the seawater has only a limited effect on the shape and level of the predicted chloride ingress profiles. Instead the difference between the profiles can be related to different temperatures of the seawater.

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3 Yearly mean temperature of seawater at the chosen locations (temperatures shown within brackets): 1 (16.0°C), 5 (11.0°C), 6 (7.5°C), 7 (7.3°C) and 8 (16.5°C).

4 Mean chloride concentration in the seawater at the chosen locations (chloride concentrations shown within brackets): 2 (3.6 g Cl/l), 3 (4.8 g Cl/l), 4 (10.1 g Cl/l), 5 (18.3 g Cl/l), 8 (21.0 g Cl/l) and 9 (23.0 g Cl/l)
The level of $C_{SN}$ used in the predictions, at least over longer exposure time, can be questioned. In the DuraCrete chloride ingress model $C_{SN}$ is modelled as constant over time. However in [3] measured chloride ingress data are presented which indicate that $C_{SN}$ seems to increase with increasing exposure time. Therefore it should be more realistic to model $C_{SN}$ as a parameter that increases with increasing exposure time. In this way the predictions of chloride ingress should yield more realistic results where $C_{SN} > C_{crit}$.

5.2 Results from probabilistic calculations

The required concrete covers predicted with probabilistic methods, presented in figure 6, have been compared with the covers predicted with deterministic methods, presented in figure 5. The predicted service life ($d_c=50$ mm) is 32 and 18 years with probabilistic methods ($\beta=2.0$) and 5900 and 300 years with deterministic methods for exposure in Kattegat (location 4) and in the Atlantic Ocean around Iceland (location 7) respectively. The reason for the extremely long service lives predicted with deterministic methods is that $C_{SN}$ is just slightly larger than $C_{crit}$. Obviously there is a large difference between the service lives predicted with probabilistic and deterministic methods. This indicates that the uncertainties of the parameters in the model have a large influence on the service life predictions and consequently if the uncertainties can be reduced the service life will be prolonged. There are several ways to reduce the uncertainties, e.g. by controlling the concrete composition, compaction of concrete and/or placement of the reinforcement etc.

The effect of $C_{crit}$ can be studied if the calculated reliabilities for location 4 ($C_{crit}=0.60$ and 2.00 [% by wt Cl/binder]) are compared. With $\beta=2.0$ ($d_c=50$ mm) the predicted service life varies between 3 and 32 years, with $C_{crit}=0.60$ and 2.00 [% by wt Cl/binder] respectively. Obviously $C_{crit}$, together with its scatter, has great influence on the predictions of service life.

6. Conclusions

Based on the description and modelling of the exposure conditions for reinforced concrete structures and results from the service life predictions:

- **Description of environmental actions.** It has been shown that it is not enough to describe the environmental actions with rough division into exposure classes. Instead the true exposure conditions, and their variations, should be taken into account, where preferably each structure should be treated separately. However, for economical and practical reasons this is usually not possible, and instead the methodology presented in this paper can be used to describe exposure conditions. Nevertheless, the models for environmental actions need further development and the parameters need to be further quantified to yield more reliable results.

- **Significance of exposure conditions.** The results of the predictions of chloride ingress and service life demonstrate the significance of the exposure conditions. The exposure conditions directly influence both $C_{SN}$, i.e. the driving potential for chlorides, and $D_a(t)$, i.e. the transport properties for chlorides in concrete. Furthermore the exposure conditions influence the chloride binding in concrete, which influences the level of both $C_{SN}$ and the chloride threshold level, $C_{crit}$, if these are defined as total chloride content in the concrete. Additionally $D_a(t)$ is also influenced by the temperature conditions, with $D_a(t)$ increasing with increasing temperature. However, the increase in $D_a(t)$ can also be explained by the temperature effect on the chloride binding.
The main influencing factors seem to be the temperature conditions and the severity of the exposure to chlorides. In marine submerged conditions the effect of the temperature conditions, while in the other exposure conditions the severity of the exposure to chlorides, were the most significant. The temperature conditions influence the level of both $C_{SN}$, $C_{crit}$, and $D_a(t)$, while the severity of exposure to chlorides mainly influences $C_{SN}$.

- **Drawbacks of model.** The DuraCrete chloride ingress model, chosen as an example model, has some drawbacks, due to simplifications. As an example the model assumes that the conditions at the surface and inside the concrete, e.g. moisture and temperature conditions, are constant over time and also over depth. This means that any effects due to variations in equivalent surface conditions over the structure can only be taken into account in a simplified way.

Another example is the surface chloride content, $C_{SN}$, which does not change over time. Instead it would be more realistic to model $C_{SN}$ as increasing with increasing exposure time, at least in marine submerged conditions.

Additionally the chloride ingress is only considered in one dimension, which means that chloride ingress close to sharp corners, where the transport of chlorides takes place in two or three dimensions, will be underestimated. Physical prediction models, which predict chloride ingress in more than one dimension, should be used instead. Furthermore these models consider variations in moisture and temperature conditions and the way they influence the chloride ingress. However, these models need extensive quantification and validation, which means that for the moment they cannot be used in practice, except in special cases, e.g. in submerged conditions with moisture saturated concrete etc.

- **Applicability of models for environmental actions.** By using the methodology to describe presented in this paper it is possible to optimize the design of reinforced concrete structures with respect to durability. Examples of where the design can be optimised with respect to durability are the concrete composition, where a dense concrete can be used at severely exposed spots and a less dense concrete at less severe exposed spots. Furthermore the geometry of a structure can be optimized, e.g. by avoiding placing columns etc close to sources of chlorides etc.

Finally it should be pointed out that the combination of workmanship, the maintenance and the microclimate have a decisive influence on the durability of a reinforced concrete structure. Thus, when an assessment of the condition of a concrete structure is made, it is important to identify the parts of the structure where the microclimate is most severe and where possible defects due to lack of workmanship and maintenance may adversely influence the durability.

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