ENVIRONMENTAL EFFECTS

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1. Introduction

During its life a structure is subject to various environmental influences (e.g., moisture, temperature, chemical substances, biological processes, etc.) which, over time, may cause deterioration of the structure material(s). Deterioration usually influences both resistance and appearance of a structure; thus, if its causes and effects are not controlled safely and serviceability of the structure may be seriously compromised. In order to design new durable structures (i.e., structures able to withstand environmental influences without or with limited signs of deterioration) or efficiently manage existing structures (i.e., determine maintenance and repair policies) models predicting the initiation and propagation of environmentally-induced deterioration are needed. Modelling a deterioration process involves significant uncertainties associated with environmental conditions, material properties, limitations of predictive models, and inadequacy of material testing, detection, and inspection methods. The aim of this section is to present probabilistic/stochastic models of possible deterioration processes for different types of structures (or structural materials).

2. Structure environment

Deterioration processes in a structure (or its components) depend on the interaction between the structure and its environment. The importance of different environmental parameters (factors) in this interaction depends on a particular deterioration process. Environmental parameters which affect all (or most) of the deterioration processes include:

- temperature;
- moisture/humidity
- wind, solar radiation (?)

Modelling these parameters for different environmental (climate) conditions is considered in this section.

Other environmental parameters influencing specific deterioration processes, e.g.,
- chloride content in air or sea water (affects corrosion of reinforcing steel in concrete and atmospheric corrosion of steel);
- concentration of carbon dioxide (CO₂) (affects carbonation and induced by it corrosion of reinforcing steel in concrete);
- concentration of sulfur dioxide (SO₂) (affects atmospheric corrosion of steel);
- presence of termites (may cause deterioration of timber structures); and
- etc.

are considered in sections concerning the relevant deterioration processes.
3.1.1 Corrosion of reinforcing steel

Corrosion of reinforcing steel is the main cause of deterioration of reinforced concrete (RC) structures. Under normal conditions concrete protects embedded reinforcing steel against corrosion. These protective properties of concrete are attributed to a passive oxide film which forms on the surface of steel in highly alkaline environment provided by the concrete pore solution. However, carbonation or penetration of chloride ions negates the protective properties of concrete and may lead, over time, to corrosion of reinforcing steel. After corrosion starts, its effects on a RC structure include cracking of the concrete cover, reduction and eventually loss of bond between concrete and corroding reinforcement, and reduction of cross-sectional area of reinforcing steel (i.e., corrosion affects both strength and serviceability of RC structures).

Thus, the development of corrosion with time and caused by it deterioration of RC structures can be divided into two stages – initiation and propagation, which is shown schematically in Figure 1 (Tuutti 1982). The time to corrosion initiation (or the initiation period) is denoted as \( t_i \); at this stage \( \text{CO}_2 \) and/or chloride ions are penetrating into the concrete without causing any structural damage. The time of the appearance of first corrosion-induced cracking (i.e., hairline cracks of width less than 0.05 mm) on the concrete surface is denoted as \( t_{cr1} \). The time of excessive cracking (i.e., cracking that is defined as serviceability failure) is denoted as \( t_{cr} \). As has been noted previously, corrosion affects not only serviceability but also the strength of RC structures. The time when the strength reduction is such that the RC structure does not satisfy anymore an ultimate limit state is denoted as \( t_u \). Usually, as shown in Figure 1, \( t_u \) is greater than \( t_{cr} \). However, in some cases, e.g., strongly localized corrosion or delamination occurring without surface cracking, \( t_u \) may be smaller than \( t_{cr} \), or even than \( t_{cr1} \). It should be also noted that Figure 1 provides just a schematic description of the deterioration process, and the rates may be more complex than those presented in the figure: the rate within each stage is not necessarily constant, and they may increase when going from one stage to another to a greater extent than shown in Figure 1. Modelling these two stages is considered in detail in the following sections.

![Figure 1. Schematic representation of the development of corrosion in RC structures.](image-url)
3.1.1.1 Initiation

The initiation stage is the period between the time of exposure of a RC structure to aggressive environment and the time of corrosion initiation. Depending on its cause corrosion may starts when:
- carbonation front reaches reinforcing steel (carbonation);
- chloride concentration near reinforcing steel reaches a threshold value (chloride ingress).

The initiation of corrosion due to carbonation and chloride ingress is considered in the next two sections.

3.1.1.1.1 Carbonation induced corrosion

Carbonation of concrete

Carbonation is a process involving two mechanisms – diffusion of carbon dioxide, CO₂, from the atmosphere into concrete and its reaction with the alkaline cement hydration products. The reaction leads to reduction of the alkalinity, pH, of the concrete pore solution from 12 to 13 to below 9, which destroys the thin passive oxide layer on the surface of steel reinforcement and makes the steel susceptible to corrosion.

CO₂ penetrates into concrete mainly due to gaseous diffusion through air-filled pores; its diffusion through the water that fills pores and convection with the water through the pores can be neglected. Diffusion of CO₂ into concrete is usually described by Fick's 1st law of diffusion, which for one-dimensional diffusion is expressed as

\[ J_{CO_2} = -D_{CO_2} \frac{∂C_{CO_2}}{∂x} \]  \hspace{1cm} (1)

where \( J_{CO_2} \) is the flux of CO₂, \( C_{CO_2} \) the concentration of CO₂, and \( D_{CO_2} \) the diffusion coefficient of CO₂ in concrete. Taking into account that \( J_{CO_2} = A \times \frac{dQ_{CO_2}}{dt} \), Eq. (1) can be written as

\[ dQ_{CO_2} = AD_{CO_2} \frac{ΔC_{CO_2}}{x} dt \]  \hspace{1cm} (2)

where \( Q_{CO_2} \) is the mass of diffusing CO₂, \( A \) the surface area, \( ΔC_{CO_2} \) the difference in the CO₂ concentration in the atmosphere and at the carbonation front, and \( x \) the distance from the concrete surface to the carbonation front.

CO₂ that diffuses into concrete reacts with the alkaline cement hydration products. This reaction is called carbonation and it may simply be described by the reaction between Ca(OH)₂ and CO₂ (e.g., Taylor 1997)

\[ \text{Ca(OH)}_2 + \text{CO}_2 \rightarrow \text{CaCO}_3 + \text{H}_2\text{O} \]  \hspace{1cm} (3)

It should be noted that Eq. (3) represents the final result of the reaction between Ca(OH)₂ and CO₂, which in reality involves a number of intermediate reactions not shown here for the sake of simplicity. As a result of the reaction, practically insoluble CaCO₃ and water are formed. The reaction of carbonation removes free CO₂ from the mass balance that can be described by the following equation

\[ dQ_{CO_2} = bAdx \]  \hspace{1cm} (4)

where \( b \) is the CO₂-binding capacity of concrete, which depends mainly on the composition of the concrete.
Solution

If to assume that the reaction of carbonation, Eq. (3), occurs instantaneously withdrawing CO$_2$ from the concrete pores and preventing it from diffusing further, the mass balance equation at the carbonation front can then be formulated using Eqs. (2) and (4)

$$bAdx = AD_{CO_2} \frac{\Delta C_{CO_2}}{x} \, dt$$

(5)

Solution of this equation is

$$x_c(t) = \sqrt{\frac{2D_{CO_2} \Delta C_{CO_2}}{b}} \sqrt{t}$$

(6)

where $x_c$ is the depth of the carbonation front at time $t$.

In this document the model initially proposed by Schiessl (1997) and then used in DuraCrete (2000) and in slightly modified form in LIFECON (2003) and fib (2006) is employed because of availability of statistical data on its main parameters. According to this model the depth of carbonation in [mm] at time $t$ [years] is evaluated as (fib 2006)

$$x_c(t) = \sqrt{2k_e k_c R_{NAC,0}^{-1} C_{CO_2, s} t W(t)}$$

(7)

where $k_e$ is the environmental function which accounts for the moisture conditions, $k_c$ the execution transfer parameter, $R_{NAC,0}^{-1}$ the inverse carbonation resistance of concrete determined in natural conditions in [(mm$^2$/year)/(kg/m$^3$)], $C_{CO_2, s}$ the concentration of CO$_2$ in the surrounding air in [kg/m$^3$], and $W(t)$ the weather function. Note that

$$R_{NAC,0}^{-1} = \frac{D_{CO_2}}{b}$$

(8)

Thus, it represents both the diffusivity and the binding capacity of concrete.

Probability of corrosion initiation

The probability of corrosion initiation, $P_{corr}$, is obviously time-dependent and can be expressed as

$$P_{corr}(t) = \Pr[c - x_c(t) < 0]$$

(9)

where $x_c(t)$ is the depth of carbonation and $c$ the thickness of the concrete cover.

Description of random variables (and other parameters)

(1) The inverse carbonation resistance of concrete determined in natural condition, $R_{NAC,0}^{-1}$

In order to reduce test time it is recommended to determine $R_{NAC,0}^{-1}$ using an Accelerated carbonation test (ACC-test). The ACC test is carried out using concrete specimens at the age of 28 days (7 day curing in tap water at $T=+20^\circ$C and then 21 day storage at $T=+20^\circ$C and RH=0.65). The specimens are then stored for 28 days in a carbonation chamber at $T=+20^\circ$C, RH=0.65, and the CO$_2$ concentration of 2.0 vol.%.

More detail description of the ACC test can be found in DuraCrete (1999) or fib (2006).
A relationship between $R^{-1}_{NAC,0}$ and the inverse carbonation resistance of concrete determined in the ACC-test, $R^{-1}_{ACC,0}$, is found by a linear regression analysis

$$R^{-1}_{NAC,0} = k_t R^{-1}_{ACC,0} + \varepsilon_t$$

(10)

where $k_t$ is the regression parameter and $\varepsilon_t$ the error term.

$R^{-1}_{ACC,0}$ can be described by a normal distribution with mean value estimated using the ACC test result as

$$\mu_{R^{-1}_{ACC,0}} = \left( \frac{X_c}{\tau} \right) \text{[(m}^2/\text{s})/(\text{kg/m}^3)]$$

(11)

where $X_c$ is the carbonation depth in [m] measured in the ACC test and $\tau=420$ (s/kg/m$^3$)$^{0.5}$ the "time constant". If no test data is available, data from Table 1 can be used for orientation purposes (fib 2006)

Since relative error in measuring $X_c$ decreases with an increase in $X_c$ and the mean value of $R^{-1}_{ACC,0}$ is directly proportional to $X_c$ the coefficient of variation of $R^{-1}_{ACC,0}$ depends on its mean value and can be found as

$$COV_{R^{-1}_{ACC,0}} = 0.689 \left( \frac{\mu_{R^{-1}_{ACC,0}} \times 10^{11}}{0.22} \right)$$

(12)

### Table 1. Mean values of $R^{-1}_{ACC,0}$ for different types of cement.

<table>
<thead>
<tr>
<th>Cement type</th>
<th>$R^{-1}_{ACC,0}$ [10$^{-11}$ (m$^2$/s)/(kg/m$^3$)]</th>
</tr>
</thead>
<tbody>
<tr>
<td>w/c$^{1_{equiv}}$</td>
<td>0.35 0.40 0.45 0.50 0.55 0.60</td>
</tr>
<tr>
<td>CEM I 42.5 R</td>
<td>n.d.$^2$ 3.1 5.2 6.8 9.8 13.4</td>
</tr>
<tr>
<td>CEM I 42.5 R + FA (k=0.5)</td>
<td>n.d.$^2$ 0.3 1.9 2.4 6.5 8.3</td>
</tr>
<tr>
<td>CEM I 42.5 R + SF (k=2.0)</td>
<td>3.5 5.5 n.d.$^2$ n.d.$^2$ 16.5 n.d.$^2$</td>
</tr>
<tr>
<td>CEM III/B 42.5</td>
<td>n.d.$^2$ 8.3 16.9 26.6 44.3 80.0</td>
</tr>
</tbody>
</table>

$^1$ equivalent water-cement ratio, considering fly ash (FA) or silica fume (SF) with the respective efficiency factors (k-values). The considered contents were: FA – 22% of weight of cement (22 wt% cement), SF – 5 wt% cement.

$^2$ n.d. – data on $R^{-1}_{ACC,0}$ for these cement types is not available.

The regression parameter, $k_t$, is described by a normal distribution with mean of 1.25 and COV=0.28.

The error term, $\varepsilon_t$, is described by a normal distribution with mean of 315.5 (mm$^2$/years)/(kg/m$^3$) and COV=0.152.

(2) The environmental function, $k_e$

The environmental function, $k_e$, takes into account the influence of the ambient relative humidity on the diffusion coefficient of CO$_2$ in concrete and, subsequently, on the carbonation resistance of concrete. It is described by the following function
\[ k_e = \left( \frac{1 - RH_{\text{real}}^2}{1 - RH_{\text{ref}}^2} \right)^{2.5} \]  

where \( RH_{\text{real}} \) is the relative air humidity obtained using data from the nearest weather station, \( RH_{\text{ref}} \) the reference relative humidity at test conditions, \( RH_{\text{ref}} = 0.65 \).

To describe \( RH_{\text{real}} \) data (daily mean values) from the nearest weather station are used. Since \( 0 \leq RH_{\text{real}} \leq 1 \) a distribution function with a lower limit and an upper limit should be used for its probabilistic model. A Beta-function is recommended for this purpose. Note that depending on the region the lower limit of the distribution may be much higher than zero.

(3) The execution transfer parameter, \( k_e \)

The execution transfer parameter, \( k_e \), takes into account the influence of curing. It is described by the following formula

\[ k_e = \left( \frac{t_e}{7} \right)^{b_e} \]  

where \( t_e \) is the period of curing in days, \( b_e \) the exponent of regression.

\( t_e \) is a deterministic parameter, while \( b_e \) is described by a normal distribution with mean of \(-0.567\) and COV=0.042.

(4) The ambient concentration of CO\(_2\), \( C_{CO_2,s} \)

The ambient concentration of CO\(_2\), \( C_{CO_2,s} \), is represented as the sum of the concentration of CO\(_2\) in the atmosphere, \( C_{CO_2,\text{atm}} \), and the additional CO\(_2\) concentration due to emission sources (e.g., for road tunnels), \( C_{CO_2,\text{emi}} \)

\[ C_{CO_2,s} = C_{CO_2,\text{atm}} + C_{CO_2,\text{emi}} \]  

\( C_{CO_2,\text{atm}} \) is described by a normal distribution with mean of \(8.2 \times 10^{-4} \) kg/m\(^3\) and COV=0.122.

For usual structures \( C_{CO_2,s} = C_{CO_2,\text{atm}} \).

(5) The weather function, \( W(t) \)

The weather function, \( W(t) \), takes into account the micro-climate conditions of the considered concrete surface and is described by the following formula

\[ W(t) = \left( \frac{t_0}{t} \right)^w \]  

where \( t_0 \) is the reference time, \( t \) the time of exposure, and \( w \) the weather exponent.

The weather exponent is estimated as

\[ w = \frac{(p_{sr} ToW)^{b_w}}{2} \]  

where \( ToW \) is the time of wetness, \( p_{sr} \) the parameter representing the probability of driving rain, and \( b_w \) the exponent of regression.
ToW represents the average number of rainy days per year; a day is defined as rainy if the amount of precipitation water during this day ≥2.5 mm, i.e.,

$$ToW = \frac{\text{days per year with rainfall } \geq 2.5 \text{ mm}}{365}$$

(18)

ToW is a deterministic parameter which can be estimated using data from the nearest weather station.

$p_{sr}$, a deterministic parameter representing the average distribution of the wind direction during raining:
- for vertical elements it should be evaluated from the nearest weather station data;
- for horizontal elements $p_{sr}$=1;
- for interior elements $p_{sr}$=0.

$b_w$ is described by a normal distribution with mean of 0.446 and COV=0.365.

(6) The thickness of the concrete cover, $c$

The thickness of the concrete cover $c$ can be described by a lognormal distribution with mean equal to its nominal value and standard deviation of: 8 – 10 mm - without particular execution requirements, and 6 mm – with additional execution requirements.

3.1.1.1.2 Chloride induced corrosion

Chloride ingress into concrete

The penetration of chloride ions into concrete is a complex process involving two transport mechanisms – ion diffusion and convection (e.g., Nilsson et al. 1996). It depends on a large number of factors including the properties of concrete (i.e., its composition, porosity and microstructure), the degree of concrete pore saturation, and the exposure conditions at the concrete surface sometimes referred to as "microclimate" (i.e., the surface chloride content, temperature, and humidity). A number of these factors are inter-, time-, spatial-, and temperature-dependent. Since only free chloride ions can penetrate into concrete their penetration depends also on the binding capacity of concrete and on the achieved free chloride concentration, which is obviously time-dependent as well. A number of physically based models of different level of sophistication have been proposed to describe the process (e.g., Saetta et al. 1993b, Boddy et al. 1999, Ababneh et al. 2003, Johannesson 2003, Meijers et al. 2005). The models involve a large number of parameters, which depend on numerous different factors (see above) and usually can only be determined experimentally with a large degree of uncertainty. Therefore, while such models may be useful for research purposes, their practical application is still very limited. For practical purposes, simpler semi-empirical models, which assume that the chloride ingress can be treated as a pure diffusion, are usually employed. Such a model is also used in this document.

Since the chloride ingress is treated as a pure diffusion process it can be described by Fick’s 2nd law of diffusion
\[
\frac{\partial C}{\partial t} = D \frac{\partial^2 C}{\partial x^2}
\]  

(1)

where \(C\) is the total concentration of chloride ions at distance \(x\) from the surface after the time \(t\) of exposure to chlorides and \(D\) the chloride diffusion coefficient. To complete the problem formulation boundary and initial conditions need to be defined

\[
\begin{align*}
C(\Delta x, t > 0) &= C_{s,\Delta x}(t) \\
C(x > \Delta x, t) &= C_i(x)
\end{align*}
\]

(2)

where \(\Delta x\) is the depth of the convection zone (i.e., the surface concrete layer in which chloride penetration is mainly governed by convection and Fick’s 2\(^{\text{nd}}\) law is inapplicable), \(C_{s,\Delta x}(t)\) the chloride concentration at depth \(\Delta x\), and \(C_i(x)\) the initial chloride concentration.

**Solution**

Generally, both the chloride diffusion coefficient and the surface chloride concentration may change with time. The time dependency of \(C_s\) is not taken into account, since there are indications that its built-up periods are often relatively short so for long-term predictions it can be assumed to be constant. The time-dependence of the chloride diffusion coefficient is described in the form suggested by Maage et al. (1996)

\[
D_a(t) = D_{a,\text{ref}} \left( \frac{t_{\text{ref}}}{t} \right)^m
\]

(3)

where \(D_a\) denotes the achieved (or apparent) diffusion coefficient, \(D_{a,\text{ref}}\) its value at the reference time \(t_{\text{ref}}\), and \(m\) the age factor. It is important to stress that according to its definition the achieved diffusion coefficient remains constant within a time period \(t\) and changes only when the duration of the time period changes. Therefore, \(D_a\) does not represent a physical property of concrete at a particular moment in time, but rather characterises an average diffusivity over a time period \(t\).

Assuming that the surface chloride concentration remains constant with time and the time-dependence of the diffusion is described by Eq. (3) the solution of Eq. (1) can be expressed as

\[
C(x, t) = C_i + (C_{s,\Delta x} - C_i) \text{erfc} \left( \frac{x - \Delta x}{2\sqrt{tD_a(t)}} \right)
\]

(4)

where \(D_a(t)\) the apparent diffusion coefficient, which apart from time depends also on temperature (see below) and erfc the error function complement.

It should be noted that in accordance to the adopted definition of the diffusion coefficient, Eq. (3), the solution, Eq. (4), is valid only at a single point in time, for which \(D_a\) is estimated. Strictly from a mathematical point of view this cannot be correct. However, if to assume that Eq. (3) describes an instantaneous value of the diffusion coefficient, i.e., the diffusion coefficient changes constantly within a time period \(t\), then the solution of Eq. (1) will be differ from Eq. (4) only by the multiplier \(1/(1-m)\) appearing under the square root. This means that \(D_{a,\text{ref}}\) in Eq. (4) merely replaces a reference value for the instantaneous diffusion coefficient times \(1/(1-m)\).
to keep this in mind while estimating $D_{a,\text{ref}}$, the solution given by Eq. (4) will yield exactly the same results as the mathematically correct solution with the instantaneous diffusion coefficient.

$D_{a,\text{ref}}$ can be estimated as

$$D_{a,\text{ref}} = \frac{1}{1-m} k_T D_{\text{RCM,0}}$$

(5)

where $D_{\text{RCM,0}}$ is the reference value determined as the chloride migration coefficient measured by the Rapid Chloride Migration (RCM) method (NT Build 492) and $k_T$ the temperature parameter described by the Arrhenius equation

$$k_T = \exp \left[ b_T \left( \frac{1}{T_{\text{ref}}} - \frac{1}{T} \right) \right]$$

(6)

where $b_T$ is the regression parameter, $T_{\text{ref}}$ the reference temperature (=293 K), and $T$ the ambient temperature.

Probability of corrosion initiation

The probability of corrosion initiation, $P_{\text{corr}}$, is obviously time-dependent and can be expressed as

$$P_{\text{corr}}(t) = \Pr[C_{\text{crit}} - C(x = c, t) < 0]$$

(7)

where $C_{\text{crit}}$ is the threshold chloride concentration and $c$ the thickness of the concrete cover.

Description of random variables (and other parameters)

1) The chloride migration coefficient measured by the RCM method, $D_{\text{RCM,0}}$

As mentioned above a mean value of $D_{\text{RCM,0}}$ should be determined by the RCM test method. If no test data is available, the following data (Table 2) can be used for orientation purposes (fib 2006).

Table 2. Mean values of $D_{\text{RCM,0}}$ for different types of cement.

<table>
<thead>
<tr>
<th>Cement type</th>
<th>$D_{\text{RCM,0}} \left[ 10^{-12} \text{m}^2/\text{s} \right]$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>w/c$^{-1}$ equivalent water-cement ratio, considering fly ash (FA) or silica fume (SF) with the respective efficiency factors (k-values). The considered contents were: FA – 22% of weight of cement (22 wt% cement), SF – 5 wt% cement.</td>
</tr>
<tr>
<td></td>
<td>0.35</td>
</tr>
<tr>
<td>CEM I 42.5 R</td>
<td>n.d.$^2$</td>
</tr>
<tr>
<td>CEM I 42.5 R + FA (k=0.5)</td>
<td>n.d.$^2$</td>
</tr>
<tr>
<td>CEM I 42.5 R + SF (k=2.0)</td>
<td>4.4</td>
</tr>
<tr>
<td>CEM III/B 42.5</td>
<td>n.d.$^2$</td>
</tr>
</tbody>
</table>

$^1$ equivalent water-cement ratio, considering fly ash (FA) or silica fume (SF) with the respective efficiency factors (k-values). The considered contents were: FA – 22% of weight of cement (22 wt% cement), SF – 5 wt% cement.

$^2$ n.d. – data on $D_{\text{RCM,0}}$ for these cement types are not available.

$D_{\text{RCM,0}}$ should be measured at 28 days (i.e., $t_{\text{ref}}$=28 days or 0.0767 years). It is assumed that $D_{\text{RCM,0}}$ is normally distributed with a coefficient of variation (COV) of 0.2.
2) The age factor, \(m\)

The age factor \(m\) depends on the concrete composition and exposure conditions. It can be presented by a Beta distribution on the interval \([0, 1]\). The mean and COV of \(m\) for RC structures in the splash, tidal, and submerge zones can be adopted in accordance to Table 3. Note that the values of \(m\) in Table 3 were derived by fitting Eq. (5) with \(D_{RCM,0}\) obtained from the RCM test directly to a number of values of \(D_a\) estimated at different points in time. By this reason by the multiplier \(1/(1-m)\) does not appear in Eq. (5).

<table>
<thead>
<tr>
<th>Type of cement</th>
<th>Mean</th>
<th>COV</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ordinary Portland cement (OPC)</td>
<td>0.30</td>
<td>0.40</td>
</tr>
<tr>
<td>CEM I; (0.40 \leq \text{w/c} \leq 0.60)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>OPC with fly ash (FA)</td>
<td>0.60</td>
<td>0.25</td>
</tr>
<tr>
<td>CEM I +FA; FA(\geq 20) wt% cement; (0.40 \leq \text{w/c}_{eqv} \leq 0.62)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Blast furnace slag cement (GBFS)</td>
<td>0.45</td>
<td>0.44</td>
</tr>
<tr>
<td>CEM III/B; (0.40 \leq \text{w/c} \leq 0.60)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

3) The regression parameter, \(b_T\)

The regression parameter \(b_T\) (in K) can be described by a normal distribution with mean of 4800 K and standard deviation of 700 K (COV=0.146).

4) The threshold chloride concentration, \(C_{crit}\)

The threshold chloride concentration \(C_{crit}\) is described in terms of wt\% cement by a Beta distribution on \([0.2, 2.0]\) wt\% cement with mean of 0.6 wt\% cement and COV=0.25.

5) The thickness of the concrete cover, \(c\)

The thickness of the concrete cover \(c\) can be described by a lognormal distribution with mean equal to its nominal value and standard deviation of: 8 – 10 mm - without particular execution requirements, and 6 mm – with additional execution requirements.

6) The depth of the convection zone, \(\Delta x\)

For the splash zone, \(\Delta x\) can be described by a Beta distribution on \([0.0, 50.0]\) mm with mean of 9 mm and COV=0.60.

For the submerged zone and the atmospheric zone (spray conditions, e.g., 1.5 m above the road surface), \(\Delta x\) can be treated as a deterministic parameter equal to 0.

For the tidal zone, there is currently not enough information to quantify statistical properties of \(\Delta x\).

7) The chloride concentration at depth \(\Delta x\), \(C_{s,\Delta x}\)

\(C_{s,\Delta x}\) depends on the binding capacity of the concrete, which in its turn depends on the concrete composition, the water-binder ratio and temperature, and on the ambient chloride concentration.

When the source of chlorides is de-icing salt, \(C_{s,\Delta x}\) can be described by a normal distribution with mean given by the following equation
\[ \mu_{C_{s,\Delta x}} = 0.465 - 0.051 \ln(d_h + 1) - [0.00065(d_h + 1)^{0.117}] \text{[wt.-\%/concrete]} \] (8a)

where \( d_h \) is the horizontal distance from the roadside in [cm], \( d_v \) the height (vertical distance) above the road surface, and COV = 0.75.

Eq. (8) is based on data from 5-40 year old concrete structures made of OPC (CEM I) concrete with 0.45 ≤ w/c ≤ 0.60 and located in urban and rural areas of Germany.

When the source of chlorides is sea water, the ambient chloride concentration depends on the chloride concentration of sea water, distance/height from/above the sea (e.g., splash zone, on the coast, etc.) and a number of other factors (e.g., sheltered or unsheltered surface, dominant wind direction, etc.). Thus, for realistic predictions of the probability of corrosion initiation in such cases the use of local data on \( C_{s,\Delta x} \) is highly desirable.

In the absence of such data, the mean value of \( C_{s,\Delta x} \) can be estimated as

\[ \mu_{C_{s,\Delta x}} = C_{s,0} k_{C,\text{binder}} k_{C,\text{w/c}} k_{C,\text{Cl}} k_{C,\text{ec}} \] (8b)

where \( C_{s,0} = 2.5\% \) (wt.-\%/concrete) is the average chloride concentration at depth \( \Delta x \) for OPC (CEM I) concrete in the tidal/splash zone (XS3 exposure class) exposed to sea water with typical chloride concentration (i.e., about 19.4 g/l), \( k_{C,\text{binder}} \) the parameter accounting for the type of binder, \( k_{C,\text{w/c}} \) the parameter accounting for the water-cement ratio of the concrete, \( k_{C,\text{Cl}} \) the parameter accounting for the chloride concentration of sea water and \( k_{C,\text{ec}} \) the parameter accounting for the exposure class.

The following values of the parameters are recommended:

- \( k_{C,\text{binder}} = \begin{cases} 1.0 & \text{CEM I 42.5 R} \\ 1.2 & \text{CEM I 42.5 R + FA, CEM I 42.5 R + SF} \\ 1.05 & \text{CEM III/B 42.5} \end{cases} \)
- \( k_{C,\text{w/c}} = 2.5(\text{w/c}) \)
  \[ \begin{array}{c} 0.76 & \text{sea water chloride concentration = 5 g/l} \\ 0.88 & 10 \text{ g/l} \\ 1.0 & 20 \text{ g/l} \\ 1.12 & 25 \text{ g/l} \end{array} \]
- \( k_{C,\text{Cl}} = \begin{cases} 0.4 & \text{exposure class XS1 (airborn salt, no direct contact with sea water)} \\ 2.0 & \text{exposure class XS2 (permanently submerged)} \\ 1.0 & \text{exposure class XS3 (tidal and splash zones)} \end{cases} \)

\( C_{s,\Delta x} \) can be modelled as a truncated at 0 normal random variable with COV depending on the exposure class: 0.30 for XS1, 0.15 for XS2 and 0.25 for XS3.

Comments: most of relevant data are available for OPC concrete with w/c around 0.40 in splash/tidal zone (XS3 environment) and sea water with the typical chloride concentration (i.e., 19.4 g/l). According to these data from various independent sources, the average value of the surface chloride concentration is about 2.5% wt.-\%/concrete (Bamforth et al. 1997; McCarter et al. 2008; Pack et al. 2010). Based on that the value of \( C_{s,0} \) has been assigned.
There is some evidence that the surface chloride concentration is slightly higher for concretes with FA, SF and GGBS (e.g., DuraCrete BE95-1347/R9, Pack et al. 2010). In particular, DuraCrete BE95-1347/R9 suggests a linear relationship between the surface chloride concentration, $C_{SN}$, and the water-cement ratio, w/c: $C_{SN}=A(w/c)$ and provides values of $A$ for different concretes in different environments (Table 8.5). However, these values look inconsistent and many of them are not supported by data reported in other publications. Based on data from various publications, including DuraCrete BE95-1347/R9, the above values of $k_{C,binder}$ are tentatively suggested.

$k_{C,w/c}$ and $k_{C,Cl}$ have been adopted from Lindvall (2003).

The above values of $k_{C,ec}$ are mainly based on data from DuraCrete BE95-1347/R9 and McCarter et al. (2008).

Available data on the coefficient of variation of $C_{SN}$ are very inconsistent. For example, the data presented in DuraCrete BE95-1347/R9 suggests that the COV is below 0.20 for different concretes and different environments. Statistical analysis of the data presented in Pack et al. (2010) yielded the COV=0.14 for OPC concrete and 0.05 for GGBS concrete (XS3 environment). At the same time, according to the data presented by Polder and de Rooij (2005) for OPC and GGBS concretes in XS3 environments values of the COV can be very high (for several structures between 0.3 and 0.4, and in one case even 0.71). It is expected that for permanently submerged structures environmental conditions (XS2) are more uniform so that the COV should be the lowest among the three exposure classes. The conditions are much more variable in tidal/splash zones (XS3) and even more variable in XS1 environments. Taking all these into account and with the intent not to be over conservative, the above values of the COV are suggested.

References:

3.1.1.2 Propagation

3.1.1.2.1 Corrosion rate

To predict the development of corrosion-induced deterioration with time the rate of corrosion needs to be known. The latter is usually described in terms either of the
corrosion current density, $i_{\text{corr}}$, or the corrosion penetration, $V_{\text{corr}}$. According to DuraCrete (2000) and LIFECON (2003), $V_{\text{corr}}$ is estimated as

$$V_{\text{corr}} = Vw, \quad [\mu m/\text{year}]$$

where $V$ is the corrosion rate, and $w$, the relative time of wetness. The corrosion rate is expressed via the concrete resistivity, $\rho(t)$

$$V = \frac{m_0}{\rho(t)} F_{Cl}$$

where $m_0=882 \ \mu m\cdot\Omega m/year$ and $F_{Cl}=1+k(C-C_{cri})$ the chloride corrosion rate factor ($F_{Cl} \geq 1$). The concrete resistivity is time-variant and depends on a number of factors

$$\rho(t) = \rho_0 k_{R,T} k_{R,RH} k_{R,Cl} \left( \frac{t}{t_0} \right)^n$$

where $\rho_0$ is the specific electrical resistivity of concrete at time $t_0$, $k_{R,T}=\exp[b_{R,T}(1/T_{\text{real}}-1/T_0)]$ the temperature factor, $b_{R,T}$ the regression variable, $T_0=293$ K, $k_{R,RH}$ the relative humidity factor, $k_{R,Cl}=1-(1-a_{Cl})/2$ the chloride factor, $t_0=28$ days, $t$ the age of concrete (in analysis $t \leq 1$ year), and $n$ the age factor for resistivity. In analysis $k$, $\rho_0$, $b_{R,T}$, $k_{R,RH}$, $k_{R,Cl}$ (or $a_{Cl}$), and $n$ are treated as random variables, whose statistical properties are given in Table 4.

Table 4. Statistical parameters of random variables related to corrosion rate

<table>
<thead>
<tr>
<th>Variable</th>
<th>Mean</th>
<th>COV</th>
<th>Distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td>$k$</td>
<td>2.63</td>
<td>1.335</td>
<td>Shifted lognormal, min. 1.09</td>
</tr>
<tr>
<td>$\rho_0$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$f_c' = 30 \text{ MPa}$</td>
<td>116 $\Omega m$</td>
<td>0.16</td>
<td>Normal</td>
</tr>
<tr>
<td>$f_c' = 40 \text{ MPa}$</td>
<td>134 $\Omega m$</td>
<td>0.16</td>
<td>Normal</td>
</tr>
<tr>
<td>$f_c' = 50 \text{ MPa}$</td>
<td>155 $\Omega m$</td>
<td>0.16</td>
<td>Normal</td>
</tr>
<tr>
<td>$b_{R,T}$</td>
<td>3815 K</td>
<td>0.15</td>
<td>Lognormal</td>
</tr>
<tr>
<td>$k_{R,RH}$</td>
<td>1.0</td>
<td>-</td>
<td>Deterministic</td>
</tr>
<tr>
<td>$a_{Cl}$ ($C &lt; 2%$)</td>
<td>0.72</td>
<td>0.153</td>
<td>Normal</td>
</tr>
<tr>
<td>$k_{R,Cl}$ ($C \geq 2%$)</td>
<td>0.72</td>
<td>0.153</td>
<td>Normal</td>
</tr>
<tr>
<td>$n$</td>
<td>0.23</td>
<td>0.174</td>
<td>Normal</td>
</tr>
</tbody>
</table>

3.1.1.2.2 Cracking of concrete cover

Corrosion products occupy a larger volume than the consumed steel. As they form, they exert pressure on the concrete surrounding a corroding reinforcing bar. This eventually leads to cracking of the concrete cover, which poses a serious problem for serviceability of reinforced concrete structures. There are two important parameters characterizing the performance of reinforced concrete structures in relation to corrosion-induced cracking: (i) the time of appearance of a corrosion-induced crack on the concrete surface, $t_{cr1}$; and (ii) the time of excessive cracking, $t_{cr}$ (see Figure 1). In the following probabilistic models for their estimation are described.

**Time to crack initiation, $t_{cr1}$**

The time to crack initiation equals

$$t_{cr1} = t_c + \Delta t_{cr1}$$
where \( t_i \) is the time to corrosion initiation (its probabilistic evaluation is described in Section 3.1.1.1) and \( \Delta t_{cr1} \) the time between corrosion initiation and cover cracking. The latter is estimated using the so-called thick-walled cylinder model (e.g., El Maaddawy and Soudki 2007). In this model the concrete around a corroding reinforcing bar is represented by a hollow thick-walled cylinder with the wall thickness equal to the thickness of the concrete cover, \( c \), and the internal diameter equal to the diameter of the reinforcing bar, \( d \). Expansion of the corrosion products is represented by uniform pressure applied to the inner surface of the cylinder (see Figure 2).

![Figure 2. Thick-walled cylinder model.](image)

A relationship between the expansion of corrosion products \( \Delta d \) (i.e., increase of the bar diameter) and the pressure, \( P \), caused by it is given by the following formula

\[
\Delta d = \frac{d}{E_{c,ef}} \left[ 1 + \nu_c + \frac{d^2}{2c(c+d)} \right] P
\]  

(13)

where \( \nu_c \) is the Poisson's ratio of concrete, \( E_{c,ef} = E_c/(1+\varphi) \) the effective modulus of elasticity of concrete, \( E_c \) the modulus of elasticity of the concrete at age of 28 days, and \( \varphi \) the concrete creep coefficient.

The concrete cover is fully cracked when the average tensile stress in it becomes equal to the tensile strength of concrete, \( f_{ct} \). The average tensile stress is estimated as the average tangential stress in the cylinder wall so that the internal pressure causing the concrete cover cracking, \( P_{cr} \), equals

\[
P_{cr} = \frac{2cf_{ct}}{d}
\]  

(14)

The expansion of corrosion products can be estimated as (e.g., Chernin et al. 2010)

\[
\Delta d = 2(\alpha_v - 1)\nu_{corr} \Delta t \quad [\mu m]
\]  

(15)

where \( \alpha_v \) is the volumetric expansion ratio of corrosion products, \( \nu_{corr} \) is given by Eq. (9) and \( \Delta t \) the time since corrosion initiation in years.

However, not all corrosion products contribute to the pressure exerted on the surrounding concrete since a part of them diffuses into concrete pores and microcracks. Denote the relative fraction of the corrosion products diffused into concrete as \( \eta \), then the actual expansion of the corrosion products around a corroding reinforcing bar is \( \Delta d(1-\eta) \) (Chernin et al. 2010). Substituting the last result into Eq. (13) and using Eqs. (14) and (15), the time between corrosion initiation and cover cracking is found as
\[
\Delta t_{cr} = \frac{cf \varepsilon}{(\alpha - 1)(1 - \eta)E_{c,ef}V_{corr}} \left[1 + \nu + \frac{d^2}{2(c + d)}\right] \text{ [years]}
\] (16)

The diameter of a reinforcing bar, \(d\), is deterministic; the concrete properties \(f_{c,ef}\) and \(E_{c,ef}\) are defined in Section 3.01 of PMC, \(\nu = 0.20\); \(c\) is described in Section 3.1.1.1.1; \(V_{corr}\) in Section 3.1.1.2.1; and statistical properties of \(\alpha\) and \(\eta\) are given in Table 5.

**Table 5. Statistical parameters of random variables related to cover cracking**

<table>
<thead>
<tr>
<th>Variable</th>
<th>Mean</th>
<th>COV</th>
<th>Distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td>(\alpha)</td>
<td>3.0</td>
<td>0.30</td>
<td>Beta on [2.0,6.4]</td>
</tr>
<tr>
<td>(H)</td>
<td>0.70</td>
<td>0.30</td>
<td>Beta on [0.0,0.9]</td>
</tr>
</tbody>
</table>

The model does not take into account the actual location of a reinforcing bar within a structural element and does not distinguish between cracking and delamination. As a result, it may lead to significant errors. Based on the comparison with results of more accurate nonlinear finite element analysis, it has been established that the model should not be used for predicting the time to crack initiation around internal reinforcing bars in reinforced concrete slabs when (see Figure 3)

\[
1.5 \leq \frac{c_1}{d} \leq 4 \text{ or } 1.5 \leq \frac{c_2}{c_1} \leq 3.5
\]

In such cases crack initiation can be predicted, e.g., by nonlinear finite element analysis.

![Figure 3. Fragments of reinforced concrete cross-section representing different locations of a reinforcing bar: (a) corner bar; (b) internal bar.](image)

**Time to excessive cracking, \(t_{ex}\)**

There are currently no sufficiently accurate and reliable models capable to predict crack propagation over time.

### 3.1.1.2.3 Effect of corrosion on bond strength

The bond stress-slip relationship is adopted from the CEB-FIP Model Code 1990 (MC 90, 1993) (see also Figure 4):
Parameters of Eq. (17) are given in Table 6, except of $\tau_{\text{max}}$.

\[
\tau = \begin{cases} 
\tau_{\text{max}} \left( \frac{s}{s_1} \right)^{a} & 0 \leq s \leq s_1 \\
\tau_{\text{max}} - \frac{\tau_{\text{max}} - \tau_f (s - s_1)}{s_2 - s_3} (s - s_3) & s_1 \leq s \leq s_2 \\
\tau_f & s \geq s_3 
\end{cases}
\]  

(17)

Figure 4. Bond stress-slip relationship.

Table 6. Parameters of Eq. (17).

<table>
<thead>
<tr>
<th></th>
<th>Unconfined concrete*</th>
<th>Confined concrete#</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Good bond conditions</td>
<td>All other bond</td>
</tr>
<tr>
<td></td>
<td>conditions</td>
<td>conditions</td>
</tr>
<tr>
<td>$s_1$</td>
<td>0.6 mm</td>
<td>0.6 mm</td>
</tr>
<tr>
<td>$s_2$</td>
<td>0.6 mm</td>
<td>1.0 mm</td>
</tr>
<tr>
<td>$s_3$</td>
<td>1.0 mm</td>
<td>3.0 mm</td>
</tr>
<tr>
<td>$s$</td>
<td>1.0 mm</td>
<td>2.5 mm</td>
</tr>
<tr>
<td>$\alpha$</td>
<td>1.0 mm</td>
<td>Clear rib spacing</td>
</tr>
<tr>
<td>$\alpha$</td>
<td>0.4</td>
<td>Clear rib spacing</td>
</tr>
<tr>
<td>$\tau_1$</td>
<td>2.0$\sqrt{f_c}$</td>
<td>1.0$\sqrt{f_c}$</td>
</tr>
<tr>
<td>$\tau_f$</td>
<td>2.5$\sqrt{f_c}$</td>
<td>1.25$\sqrt{f_c}$</td>
</tr>
<tr>
<td>$\tau_f$</td>
<td>0.15$\tau_1$</td>
<td>0.40$\tau_1$</td>
</tr>
</tbody>
</table>

*Failure by splitting concrete
#Failure by shearing concrete between reinforcing bar ribs

Concrete can be treated as unconfined when the ratio of transverse reinforcement to longitudinal reinforcement, $\rho$, does not exceed 0.25; confined concrete corresponds to $\rho > 1$. This ratio is defined as

\[
\rho = \frac{A_{\text{st}}}{nA_{\text{l}}}
\]

(18)

where $A_{\text{st}}$ is the cross-sectional area of stirrups (two legs) over the anchorage length (usually between $15d$ and $20d$ with $d$ denoting the diameter of longitudinal bars), $n$ the
number of longitudinal bars enclosed by stirrups, and $A_s$ the cross-sectional area of one longitudinal bar. When $0.25 < \rho \leq 1$ the parameters of Eq. (17) can be estimated by interpolation of the values given in Table 6.

Uncertainty associated with Eq. (17) can be modelled by a normal random variable with the mean value equal to unity and the coefficient of variation of 0.3 (i.e., values of $\tau$ obtained from Eq. (17) should be multiplied by this random variable). The unloading branch of the bond stress-slip relationship is linear; its slope is independent of $s$ and can be taken equal to 200 N/mm$^3$.

According to available experimental data at low levels of corrosion the bond strength initially increases and then starts to decrease as the corrosion propagates (the decrease usually occurs after the formation of corrosion-induced cracks in the concrete cover). Results of tests also indicate that there is residual bond strength (about 15% of its initial value) even at very high levels of corrosion (up to 80% of cross-sectional loss) (for more detail see Val and Chernin (2009)). Based on these observations the following relationship between the normalized bond strength, $\frac{\tau_{\text{max}}}{\tau_{\text{max},0}}$, representing the ratio of the bond strength with corrosion, $\tau_{\text{max}}$, to the initial bond strength, $\tau_{\text{max},0}$, and the corrosion penetration (i.e., the reduction in radius of the uncorroded reinforcing steel), $p$, is proposed (see also Figure 5)

$$\frac{\tau_{\text{max}}}{\tau_{\text{max},0}} = \begin{cases} 1 + (k_1 - 1) \frac{p}{p_{\text{cr1}}} , & p \leq p_{\text{cr1}} \\ \max[k_1 - k_2 (p - p_{\text{cr1}}), 0.15] , & p > p_{\text{cr1}} \end{cases} \quad (19)$$

where $p_{\text{cr1}}$ is the corrosion penetration corresponding to crack initiation in the concrete cover.

![Figure 5. Normalized bond strength versus corrosion penetration.](image)

If $\tau_{\text{max}} < \tau$, then Eq. (17) becomes

$$\tau = \begin{cases} \tau_{\text{max}} \left( \frac{s}{s_1} \right)^{\alpha} & 0 \leq s \leq s_1 \\ \tau_{\text{max}} & s > s_1 \end{cases}$$

The corrosion penetration, $p$, at time $t$ (in years) and the corrosion penetration corresponding to crack initiation, $p_{\text{cr1}}$, are estimated as
\[ p = V_{corr} t \quad [\mu m] \]  
\[ p_{cr1} = V_{corr} t_{cr1} \quad [\mu m] \]  
where \( V_{corr} \) is calculated using Eq. (9) and \( t_{cr1} \) – Eq. (16).

The value of \( k_1 \) representing the initial increase in the bond strength after corrosion initiation apparently depends on the level of confinement provided by the concrete cover and transverse reinforcement (i.e., stirrups). The former is usually characterized by the ratio of the concrete cover to the reinforcing bar diameter, \( c/d \), while the latter can be represented by the ratio \( \rho \) (see above). Thus, in order to determine \( k_1 \) the maximum values of the bond strength between corroding reinforcement and concrete for different values of \( c/d \) and \( \rho \) need to be known. However, such data are not available; published experimental results provide values of the bond strength at different levels of corrosion, most of which are at high levels of corrosion when corrosion-induced cracks have been already formed. There are just a few experimental results (mostly for specimens without transverse reinforcement, i.e., \( \rho = 0 \)), which provide values (not necessarily maximum) of the bond strength before the formation of corrosion-induced cracks. Based on these results the following bilinear relationship between \( k_1 \) and \( c/d \) is proposed (Val and Chernin 2009)

\[
k_1 = \begin{cases} 
1, & c/d \leq 1 \\
1 + 0.085(c/d - 1), & c/d > 1 
\end{cases}
\]  

(22)

It is likely that \( k_2 \), which represents the rate of decrease in the bond strength after the formation of corrosion-induced cracks, depends on the confinement provided by transverse reinforcement (i.e., on \( \rho \)) and on the remaining rib height of a reinforcing bar (that is related to the bar diameter, \( d \)), while the influence of the \( c/d \) ratio should be relatively insignificant. However, with respect to \( d \) available experimental results do not allow to make any decisive conclusion. Therefore, it is assumed that for unconfined concrete (i.e., \( \rho \leq 0.25 \)) \( k_2 \) does not depend on \( d \) (or \( c/d \)) and its mean value equals 0.005 \( \mu m^{-1} \). There are not sufficient experimental data on the effect of corrosion on the bond strength for confined concrete (i.e., \( \rho > 1 \)). It is suggested to adopt for confined concrete (\( \rho > 1 \)) \( k_2 = 0.0025 \mu m^{-1} \), which is the slope obtained by regression analysis of experimental data from Al-Sulaimani et al (1990). Values of \( k_2 \) for \( 0.25 < \rho \leq 1 \) can be found by interpolation. Thus, the relationship between \( k_2 \) (in \( \mu m^{-1} \)) and \( \rho \) can be summarized as (Val and Chernin 2009)

\[
k_2 = \begin{cases} 
0.005, & \rho \leq 0.25 \\
0.005 - \frac{\rho - 0.25}{300}, & 0.25 < \rho \leq 1 \\
0.0025, & \rho > 1 
\end{cases}
\]  

(23)

Statistical properties of the random variables representing uncertainties associated with the bond stress-slip relationship, Eq.(17), are \( k_1 \) and \( k_2 \) are given in Table 7.

<table>
<thead>
<tr>
<th>Variable</th>
<th>Mean</th>
<th>COV</th>
<th>Distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bond model uncertainty – Eq. (17)</td>
<td>1.0</td>
<td>0.30</td>
<td>Normal</td>
</tr>
<tr>
<td>( k_1 )</td>
<td>Eq. (22)</td>
<td>0.20</td>
<td>Normal, truncated at 1</td>
</tr>
</tbody>
</table>

Table 7. Statistical parameters of random variables related to bond strength
3.1.1.2.4 Effect of corrosion on reinforcing steel

Two types of corrosion – general and pitting – are possible. General corrosion affects a substantial area of reinforcement with more or less uniform metal loss over the perimeter of reinforcing bars. Pitting (or localized) corrosion, in contrast to general corrosion, concentrates over small areas of reinforcement.

General corrosion

The cross-sectional area of a reinforcing bar with the original diameter \(d\) after \(t\) years of general corrosion will be

\[
A_g(t) = \frac{\pi [d - 2 \times 10^{-3} V_{corr} t]^2}{4} \geq 0 \quad [\text{mm}^2]
\]

(24)

According to available experimental data, general corrosion does not affect such properties of reinforcing steel as strength and ductility.

Pitting corrosion

Pitting corrosion, in contrast to general corrosion, concentrates over small areas of reinforcement. According to results of Gonzales et al. (1995), the maximum penetration of pitting, \(p_{\text{max}}\), on the surface of a rebar is about 4 to 8 times the average penetration, \(p\), of general corrosion. The results were obtained for 125-mm long specimens of 8 mm diameter reinforcing bars. These results are in broad agreement with those reported by Tuutti (1982), who received the ratio \(R = p_{\text{max}}/p\) between 4 and 10 for 5-mm and 10-mm reinforcing bars of 150 mm to 300 mm length. Thus, the depth of a pit, \(p_{\text{max}}\) (which is equivalent to the maximum penetration of pitting) after \(t\) years since corrosion initiation is evaluated as

\[
p_{\text{max}}(t) = V_{corr} \times 10^{-3} t R \quad [\text{mm}]
\]

(25)

Based on a hemispherical model of a pit suggested by Val and Melchers (1997) (see Figure 6) the cross-sectional area of a pit, \(A_p\), in a reinforcing bar with a diameter \(d\) after \(t\) years of corrosion can be calculated as

\[
A_p(t) = \begin{cases} 
A_1 + A_2, & p_{\text{max}}(t) \leq \frac{d}{\sqrt{2}} \\
\frac{\pi d^2}{4} - A_1 + A_2, & \frac{d}{\sqrt{2}} < p_{\text{max}}(t) \leq d \\
\frac{\pi d^2}{4}, & p_{\text{max}}(t) > d
\end{cases}
\]

(26)

where

\[
A_1 = \frac{1}{2} \left[ \theta_1 \left( \frac{d}{2} \right)^2 - d d \frac{p_{\text{max}}(t)^2}{d} \right]
\]

(27)
Figure 6. Pit configuration.

The cross-sectional area of a reinforcing bar after $t$ years of pitting corrosion is estimated as

$$A(t) = \frac{\pi d^2}{4} - A_{p}(t) \geq 0$$

There is significant uncertainty associated with $R$, i.e., the ratio between the maximum pit depth and the average corrosion penetration. At the same time, a popular approach to modelling pit depth is based on statistical characterization of maximum pit depth using extreme value theory, in particular the Gumbel distribution. Thus, $R$ is treated as a random variable modelled by the Gumbel distribution.

$$F(R) = \exp \left\{ - \exp \left[ -\frac{(R-\mu)}{\alpha} \right] \right\}$$

where $\alpha$ and $\mu$ are the parameters of the distribution. According to Stewart (2004), for an 8 mm diameter bar of 125 mm length the parameters of the Gumbel distribution are $\mu_0=5.08$ and $\alpha_0=1.02$. For a reinforcing bar with different dimensions the parameters of the Gumbel distribution are determined as

$$\mu = \mu_0 + \frac{1}{\alpha_0} \ln \left( \frac{A}{A_0} \right), \quad \alpha = \alpha_0$$
where $A$ is the surface area of the given bar and $A_0$ the surface area of an 8 mm diameter bar of 125 mm length.

According to available experimental data, pitting corrosion may also affect strength and ductility of reinforcing steel. It has been suggested that yield strength, $f_y$, of a steel reinforcing bar reduces linearly with still loss caused by pitting corrosion (Stewart 2009)

$$f_y(t) = [1 - \alpha_y Q_{corr}(t)]f_{y,0}$$  \hspace{1cm} (34)

where $f_{y,0}$ is the yield strength of uncorroded reinforcing bar, $\alpha_y$ an empirical coefficient and $Q_{corr}(t)$ the percentage corrosion loss, which is measured in terms of reduced cross-sectional area, i.e.,

$$Q_{corr}(t) = \frac{A_p(t)}{\pi d^2/4} \times 100\%$$  \hspace{1cm} (35)

A review of experimental studies shows that $\alpha_y$ can be in a range from 0.0 to 0.017, with average around 0.005. $\alpha_y$ can be described by a Beta distribution on [0., 0.017] with mean = 0.005 and COV = 0.20.

There is strong evidence that the mechanical behaviour of reinforcing bars changes from ductile to non-ductile (brittle) as pitting corrosion loss increases (Stewart 2009). While there is a gradual transition from ductile to brittle behaviour with an increase in corrosion loss, for simplicity it can be assumed that the complete loss of ductility in corroding reinforcing bars occurs after $Q_{corr}(t)$ exceeds a threshold value, $Q_{limit}$. This leads to two types of mechanical behaviour:

- Ductile behaviour: $Q_{corr} \leq Q_{limit}$
- Brittle behaviour: $Q_{corr} > Q_{limit}$  \hspace{1cm} (36)

The literature shows that it is reasonable to set $Q_{limit}$=20%, although more research is needed to more accurately quantify this important variable.

### 3.1.6 References


NT Build 492 11.99.


