

JCSS PROBABILISTIC MODEL CODE PART 2: LOAD MODELS

2.21 ENVIRONMENTAL EXPOSURE

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To be completed

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To be completed

(2.21) 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 the appearance and the resistance of a structure, which means that safety and serviceability of the structure may be seriously compromised.

Limit state functions including deterioration will contain next to mechanical load and resistance parameters also environmental parameters like humidity chloride concentration, diffusion coefficients, etc. In most cases time is present as an explicit parameter and limit state functions may often conveniently be formulated in the time domain. For treating time dependent reliability, reference is made to Part I of this model code. Reliability targets may be formulated both on life time as on annual basis.

The limit states to be considered are still SLS (appearance, usability) and ULS (collapse). For the sake of convenience sometimes additional condition limit states like the onset of corrosion are introduced. (see ISO 2394 and ISO 16204). These limit states may help to make a split up between the deterioration process and the mechanical collapse. If done so care should be taken to be on the conservative side.

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. This section of the JCSS Model Code presents the state of the art on the models needed for the reliability analysis for a number of relevant deterioration processes.

For existing structures best estimates may be adjusted and uncertainties may be reduced by taking into account observations and monitoring/inspection results. Standard Bayesian updating procedures can be found in Part I of the JCSS probabilistic

(2.21) 2 CONCRETE

Note:

With respect to the models in this section, it has to be emphasized that those available at present have not been validated for longer than 30 years and that the validation cases are still very scarce. Then, they have to be taken with precaution and considered as a first step to develop in the future more accurate models and values for their input parameters.

In this document the corrosion initiation models have been taken from the draft Model Code 2020 and *fib* Bulletin 34., knowing that in a research environment more advanced models are under preparation. For the propagating stage

(2.21) 2.1 CORROSION OF THE REINFORCEMENT / GENERAL

(2.21) 2.1.1 The main steps in the degradation process

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, environmental actions such as carbonation or penetration of chloride ions disrupt the protective properties of concrete and may lead, over time, to corrosion of reinforcing steel. This stage of the depassivation process is called the initiation period.

After corrosion starts, the so called propagation period, its effects on a RC structure usually 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. As a consequence corrosion affects both strength and serviceability of RC structures.

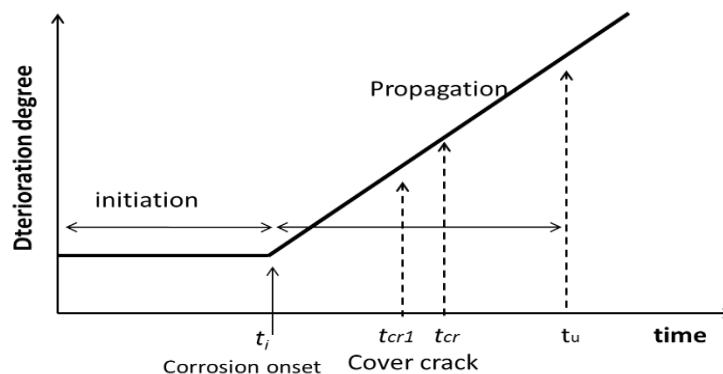


Figure 1. Schematic representation of the development of corrosion in RC structures. In the figure the time to corrosion initiation is denoted as t_i , the time of the appearance of first corrosion-induced cracking (i.e., hairline cracks of width less than 0.05 mm) on the concrete surface is denoted as t_{cr1} and the time of excessive cracking (i.e., cracking that is defined as serviceability failure) is denoted as t_{cr} . Finally, t_u indicates the loss of cross section associated to an ultimate limit state.

Two main aspects should be taken into account regarding the structural performance:

- Corrosion morphology: Whether the corrosion is homogeneous or localized (see figure 2) due to, in last case, of the stress concentration phenomenon and effect on steel ductility

- Corrosion rate. The evolution of the corrosion with time (figure 1) which leads into a decrease of diameter loss and the bar section.

With respect to the morphology of the corrosion, the attack can be homogeneous around the perimeter (see figure 2). When the corrosion is localized or asymmetric, a “pitting factor” can be introduced [1] which takes into account the deepest corrosion spot. The remaining cross section should be that having as diameter that deepest pit in order to be conservative. There may be several pits of the different depth in the same bar, but that being the deepest is which can induce a stress concentration leading in a premature failure. In the case of asymmetric corrosion, the remaining cross section should be the most conservative one due also the uncertainty on the real shape of the area lost.

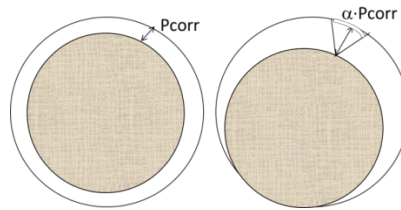


Figure 2 Homogeneous and localized corrosion [2].

Regarding to the corrosion evolution, the corrosion rate evolves with the climatic / environmental conditions and with the continuous generation of oxides which modifies the steel concrete interface. In spite of this, sometimes disperse evolution, the accumulated corrosion or corrosion penetration_r, if annually averaged, presents a continuous steady increasing trend. The cracks often produced by the expansive character of the iron oxides may induce an acceleration of the corrosion (if the structure is in continuous contact to water) or in air contact can slow down the corrosion (due to the cracked zone dries quicker than the non-cracked ones).

Once established the basic propagation model of corrosion, evolution with time, for verifying the LS it is necessary to establish the consequences of corrosion due to each accumulated level of damage. These are listed below and shown in figure 3 taken from CONTECVET Manual [2] and incorporated into Duracrete project [4]:

1. The decrease in section and bar ductility
2. The cover cracking
3. The steel/concrete bond deterioration
4. The loss in load-bearing capacity of the composite section regarding ULS

The key factor to assess structural condition which controls the rest of consequences is the loss in cross section due to the corrosion process and then, to verify the Limit States (ULS and SLS) it is necessary to consider the reduced sections of steel and concrete and their reduced properties induced by the corrosion (concrete strength and steel/concrete bond).

A particular case of corrosion is the stress corrosion cracking, SCC, mechanism which may develop in prestressed members. The process starts by the nucleation of microcracks in the surface of the steel which may take long time. One of these cracks may progress to a certain critical depth, from which the crack velocity is very high and the wire breaks in a brittle manner in relatively short time. The mechanism of nucleation and, mainly of progression of

SCC is still subjected to controversy. The structural failure may however be brittle or not depending on the reinforcing details, because the brittle failure of a tendon may not produce the brittle failure of the whole element.

(2.21) 2.1.2 Limit States to be considered

Five typical limit states associated with corrosion of reinforcement in concrete may be distinguished:

1. depassivation and onset of corrosion
2. development of a micro crack (say 0,1 mm) parallel to the bar in concrete surface.
3. crack associated with appearance, say 0,3 mm (Serviceability Limit State)
4. loss of bond
5. actual loss of (local) load bearing capacity (Ultimate Limit State)

Depassivation, as such, has no consequences for the serviceability or load bearing capacity; for that reason it is referred to as Condition Limit State (ISO 2394) or Initiation Limit State (ISO 13283). A similar note may be made in the case of loss of bond.

Notes

- (1) The target reliability required for each of these limit states in general will be more strict when going from depassivation to the ultimate limit state, roughly climbing from for instance $\beta = 0$ to $\beta = 1$ for depassivation to $\beta = 4$ for the ULS (design working life values).
- (2) If condition limit states verification is used to replace the more complex serviceability or ultimate limit state verification, the targets should be chosen accordingly.
- (3) Limit state functions may be formulated explicitly in the time domain as well as in the domain of (time dependent) physical variables like the crack width or chloride ingress.

(2.21) 2.2 CARBONATION INDUCED CORROSION, INITIATION STATE

(2.21) 2.2.1 Limit state definition

Carbonation is produced due to the ingress of the carbon dioxide gas of the air through the concrete pore network. It 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 of the concrete pore solution from pH about 12 to below 9, which destroys the thin passive oxide layer on the surface of steel reinforcement and makes the steel susceptible to corrosion.

In the case of carbonation the critical value is reached when the depth of the carbonation front is larger than the cover depth. So the limit state function $g(\cdot)$ for time t may be defined as:

$$g(\mathbf{X}, t) = c - x_c(t) \quad (1a)$$

- c is the concrete cover [mm]
- $x_c(t)$ is the carbonation depth at the time t [mm]
- t is the period of time under consideration [a]
- $\mathbf{X}$ is the set of relevant (random) parameters.

The value of x_c depends on the diffusion process as described 2.21.2.2.2

Note that given (1a) we may calculate the probability of depassivation for every point in time. Taking $t=t_d$ (where t_d is the design working life) we may find the failure probability $P\{g(t_d)<0\}$ for this limit state for the design period.

The alternative (equivalent) formulation in the time domain would be:

$$g'(\mathbf{X}, t) = t_{ini}(\mathbf{X}) - t \quad (1b)$$

where t_{ini} is the critical time for initiation at the reinforcement following from $g(\mathbf{X}, t_c) = 0$

(2.21) 2.2.2 Carbonation ingress model

The diffusion of CO₂ into concrete is usually described by Fick's 1st law of diffusion, which for one-dimensional diffusion is expressed as

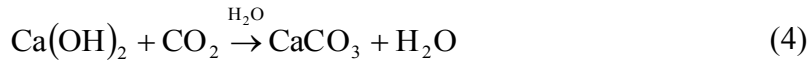
$$J_{CO_2} = -D_{CO_2} \frac{\partial C_{CO_2}}{\partial x} \quad (2)$$

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 $J_{CO_2} = A \times dQ_{CO_2}/dt$, Eq. (2) can be written as

$$dQ_{CO_2} = AD_{CO_2} \frac{\Delta C_{CO_2}}{x} dt \quad (3)$$

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 as is the responsible of the pH lowering and it may simply be described by the reaction between Ca(OH)₂ and CO₂ (e.g., Taylor 1997)



It should be noted that Eq. (4) represents the final result of the reaction between CO₂ and Ca(OH)₂, which in reality involves a number of intermediate reactions not shown here for the sake of simplicity (alkali hydroxides carbonates first and later it does the hydrated calcium silicate). 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

$$dQ_{CO_2} = bA dx \quad (5)$$

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

If to assume that the reaction of carbonation, Eq. (4), occurs instantaneously withdrawing CO₂ from the concrete pores and preventing it from diffusing further, the mass balance equation at the carbonation front can then be formulated using Eqs. (3) and (4)

$$bA dx = AD_{CO_2} \frac{\Delta C_{CO_2}}{x} dt \quad (6)$$

One solution of this equation proposed by Tutti (1981) and Schiessl (1997) [5] is:

$$x_c(t) = \sqrt{\frac{2D_{CO_2}\Delta C_{CO_2}}{b}} \sqrt{t} \quad (7)$$

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

In this document the model selected is the one used in DuraCrete (2000) [4] and in slightly modified form in LIFECON (2003) [6], *fib* (2006) [7] and MC 2010 [8]. According to this model, the depth of carbonation in [mm] at time t [years] is evaluated as (*fib* 2006):

$$x_c(t) = \sqrt{2k_e k_c R_{NAC,0}^{-1} C_{CO_2,s}} \sqrt{t} W(t) \quad (8)$$

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 [(mm²/a)/(kg/m³)], $C_{CO_2,s}$ the concentration of CO₂ in the surrounding air in [kg/m³], and $W(t)$ the weather function.

For k_e , k_c , $R_{NAC,0}^{-1}$ and $W(t)$ the weather function more detailed models exists:

$$k_e = \left(\frac{1 - RH_{real}^5}{1 - RH_{ref}^5} \right)^{2.5} \quad (9)$$

$$k_c = \left(\frac{t_c}{7} \right)^{b_c} \quad (10)$$

$$R_{NAC,0}^{-1} = \frac{D_{CO_2}}{b} \quad (11)$$

The value of $R_{NAC,0}^{-1}$ can be related to the result of accelerated tests using:

$$R_{NAC,0}^{-1} = k_t R_{ACC,0}^{-1} + \varepsilon_t \quad (12)$$

Further:

$$W(t) = \left(\frac{t_0}{t} \right)^w \quad (13)$$

where the weather exponent w which may be expressed as:

$$w = \frac{(p_{sr} ToW)^{b_w}}{2} \quad (14)$$

In the above equations is RH_{real} the relative air humidity obtained using data from the nearest weather station, RH_{ref} the reference relative humidity at test conditions, $RH_{ref} = 0.65$, t_c is the period of curing in days, b_c the exponent of regression, D_{CO_2} the diffusion coefficient of CO₂ in concrete, b is the CO₂-binding capacity of concrete, k_t , a regression parameter, ε_t an error term, ToW is the relative time of wetness, ToW represents the relative time that there is more rain than 2.5 mm per day, p_{sr} the parameter representing the probability of driving rain and b_w the exponent of regression.

(2.21) 2.2.3 Random variables for carbonation ingress

Table 1 gives an overview of the various probabilistic models. All parameters may be assumed to have a normal distribution.

Table 1 Probabilistic models for the carbonation ingress model

<i>X</i>	designation	unit	mean	scatter
c_s	Equivalent CO ₂ concentration	kg/m ³	$8 \cdot 10^{-4}$	V=0.12
R^{-1}_{NAC0}	Inverse resistance against carbonation	(m ² /s)/(kg/m ³)	Note (3)	Note (3)
RH	Equivalent RH at concrete surface	-	Note (1)	-
RH_{ref}	Reference value	-	0.65	-
b_c	Empirical constant	-	0.57	V=0.04
p_{sr}	Prob. of rain driving wind direction	-	Note (1,2)	-
k_{t_s}	Regression coefficient in resistance	-	1.25	V=0.28
ε_t	error term in resistance	(m ² /s)/(kg/m ³)-	10^{-11}	V=0.15
ToW	Relative time of wetting (days with rain >2,5 mm)	-	Note (1)	-
b_w	Coefficient weather function	-	0.446	V=0.36
t_o	Reference time weather function	(d)	28	-

Note (1): meteorological parameters

The statistics for RH , p_{sr} and ToW should be derived from local weather stations. Example values for the Netherlands are: $RH = 0.85$, $p_{sr} = 0.25$ and $ToW = 0.05$.

Note (2): rain-wind parameter

For horizontal elements $p_{sr} = 1$ and for interior elements. $p_{sr} = 0$.

Note (3): resistance against carbonation

The ACC test is carried out using concrete specimens at the age of 28 days (7 day curing in tap water at $T=20^\circ\text{C}$ and then 21 day storage at $T=20^\circ\text{C}$ and $RH=0.65$). The specimens are then stored for 28 days in a carbonation chamber at $T=20^\circ\text{C}$, $RH=0.65$, and the CO₂ concentration of 2.0 %. More detail description of the ACC test can be found in DuraCrete (1999) [4] or *fib* (2006) [7]. $R^{-1}_{ACC,0}$ can be described by a normal distribution with mean value estimated using the ACC test result as:

$$\mu(R^{-1}_{ACC,0}) = (x_c / \tau)^2 \quad [(\text{m}^2/\text{s})/(\text{kg}/\text{m}^3)] \quad (15)$$

where x_c is the carbonation depth in [m] measured in the ACC test and $\tau = 420 (\text{s}/\text{kg}/\text{m}^3)^{0.5}$ the "time constant".

If no test data is available, data from Table 2 can be used for orientation purposes (*fib* 2006) [7].

Since the relative error in determining 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:

$$V(R^{-1}_{ACC,0}) = 0.69 (\mu(R^{-1}_{ACC,0}) \times 10^{-11})^{-0.22} \quad (16)$$

Table 2. Mean values of $R_{ACC,0}^{-1}$ that can be found for different types of cement.

Cement type	$R_{ACC,0}^{-1} [10^{-11}(\text{m}^2/\text{s})(\text{kg}/\text{m}^3)]$					
	w/c_{eqv}^1					
	0.35	0.40	0.45	0.50	0.55	0.60
CEM I 42.5 R	n.d. ²	3.1	5.2	6.8	9.8	13.4
CEM I 42.5 R + FA (k=0.5)	n.d. ²	0.3	1.9	2.4	6.5	8.3
CEM I 42.5 R + SF (k=2.0)	3.5	5.5	n.d. ²	n.d. ²	16.5	n.d. ²
CEM III/B 42.5	n.d. ²	8.3	16.9	26.6	44.3	80.0

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 $R_{ACC,0}^{-1}$ for these cement types is not available.

Grey values: to be adjusted

Note (4): cover

Information on the concrete cover c can be found in Section 3.10 of the PMC; effectively it 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.

Questions:

Are the random variables in Table 2 independent?

Is there information on spatial variability (Hergenroder, Gehlen, Duracrete?)

(2.21) 2.2.4 Simplified models

The above model (11) is often written in simplified form in one of the following ways:

$$x_c(t) = V_{CO_2} \cdot \sqrt{t} \quad (17a)$$

$$x_c(t) = W(t) k \sqrt{t} \quad (17b)$$

Where V_{CO_2} = “velocity” of carbonation ingress (mm/ $\sqrt{a}$) and k is a factor reflecting all other influencing parameters. This presentation may in particular be convenient when assessing existing structures on the basis of site measurements.

What is the Scatter?

The “velocity” of carbonation ingress, V_{CO_2} , is obtained from a test in natural conditions: cylinders or cubes, after standardized 28 days wet curing, are exposed to the open atmosphere sheltering from direct rain in a station site. The representative value of velocity of carbonation is measured after 1 year of exposure.

The standard deviation in V_{CO_2} or k might also be found from the measurements, possible using Table 1 as basis for prior information.

The humidity parameter, $W(t)$, may be taken as before, but alternatively it can be used as a conversion factor from the testing conditions (sheltered from rain) to the non-sheltered conditions or to dry (interior) ones. Table 3 gives some indicative values.

Table 3 Indicative values of $W(t)$

relation of V_{CO_2} between exposure classes		
	w/c=0.4	w/c=0.6
ext shelt / ext non	2.08	1.55
ext shelt / interior	1.63	1.19

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(2.21) 2.3 CHLORIDE INDUCED CORROSION, INITIATION

(2.21) 2.3.1 Limit state formulation

The penetration of chloride ions into concrete is a Complex process involving two transport mechanisms – ion diffusion and adsorption. 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 move inwards, their penetration depends then, on the binding capacity of concrete and on the achieved free chloride concentration itself, which is obviously time-dependent as well.

The limit state function is expressed as the critical chloride concentration must be equal or higher than the chloride concentration at the rebar surface level at the design working time

$$g(\mathbf{X},t) = C_{crit} - C(c,t) \quad (1)$$

where:

- C_{crit} is the critical chloride content to achieve depassivation of the reinforcement [wt.% binder content]
- $C(c,t)$ is the chloride content at the depth c and time t [wt.% binder content]
- c is the concrete cover [mm]
- t is the period of time under consideration [a]
- $\mathbf{X}$ is the set of random variables

The formulation may also be converted into the time domain.

(2.21) 2.3.2 Chloride ingress model

Since the chloride ingress is treated as a pure diffusion process it can be described by Fick's 2nd law of diffusion:

$$\frac{\partial C}{\partial t} = D \frac{\partial^2 C}{\partial x^2} \quad (2)$$

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:

$$\begin{aligned} C(\Delta x, t > 0) &= C_{s,\Delta x}(t) \\ C(x > \Delta x, t) &= C_i(x) \end{aligned} \quad (3)$$

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

Generally, both the chloride diffusion coefficient and the surface chloride concentration may change with time. In present model the time dependency of C_s is not taken into account. The time-dependence of the chloride diffusion coefficient is described in the form suggested initially by Maage et al. (1996) and later modified to let it to be mathematically correct.

$$D_a(t) = D_{a,ref} \left(\frac{t_{ref}}{t} \right)^m \quad (4)$$

where D_a denotes the achieved (or apparent) diffusion coefficient, $D_{a,ref}$ its value at the reference time t_{ref} , and m the aging 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 (4) the solution of (2) can be expressed as:

$$C(x,t) = C_i + (C_{s,\Delta x} - C_i) \operatorname{erfc} \left(\frac{x - \Delta x}{2\sqrt{D_a(t)t}} \right) \quad (5)$$

where $D_a(t)$ the apparent diffusion coefficient, which apart from time depends also on temperature (see below) and erfc the error function complement. Finally: to find $C(c,t)$ in (1) we have to take $x = c$ in (5).

It should be noted that in accordance to the adopted definition of the diffusion coefficient, Eq. (4), the solution, Eq. (5), 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 we assume that Eq. (4) 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. (2) will differ

from Eq. (3) only by the multiplier $1/(1-m)$ appearing under the square root. This means that $D_{a,ref}$ in Eq. (5) merely replaces a reference value for the instantaneous diffusion coefficient times $1/(1-m)$. $D_{a,ref}$ can be estimated as

$$D_{a,ref} = \frac{1}{1-m} \cdot k_T \cdot D_{ref,0} \quad (6)$$

where $D_{ref,0}$ is the reference value determined in a chloride test at a reference short time (28 days for instance). k_T is the temperature parameter which can be calculated from Arrhenius equation:

$$k_T = \exp \left[b_T \left(\frac{1}{T_{ref}} - \frac{1}{T} \right) \right] \quad (7)$$

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

(2.21) 2.3.3 Random variables for chloride ingress

Table 1 and the subsequent notes give information about the various probabilistic models.

Table 1 Probabilistic models for the chloride ingress model.

X	designation	unit	mean	scatter
$D_{ref,0}$	chloride diffusion coefficient	$10^{-12}(\text{m}^2/\text{s})$	Note 1	
m	aging factor	-	Note 2	
b_T	regression parameter (in K)	K	4800	$V=0,146$
Δx	depth of absorption zone	mm	9 (see note 3)	$V = 0.6$
$C_{s,\Delta x}$	chloride concentration at depth Δx	%	Note 4	
C_{crit}	threshold chloride concentration		Note 5	

Note (1): chloride diffusion

The chloride diffusion coefficient $D_{ref,0}$ represents the rate of chloride penetration. As mentioned above a mean value of $D_{ref,0}$ should be determined by from a test. If no test data is available, the following data (Table 2) can be used for orientation purposes (*fib* 2006).

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

Cement type	$D_{ref,0} [10^{-12}(\text{m}^2/\text{s})]$					
	w/c_{eqv}^1					
	0.35	0.40	0.45	0.50	0.55	0.60
CEM I 42.5 R	n.d. ²	8.9	10.0	15.8	19.7	25.0
CEM I 42.5 R + FA (k=0.5)	n.d. ²	5.6	6.9	9.0	10.9	14.9
CEM I 42.5 R + SF (k=2.0)	4.4	4.8	n.d. ²	n.d. ²	5.3	n.d. ²
CEM III/B 42.5	n.d. ²	1.4	1.9	2.8	3.0	3.4

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 $D_{ref,0}$ for these cement types are not available.

Note (2): aging factor

The aging factor m depends on the type of cement and exposure conditions. It can be presented by a Beta distribution on the interval [0, 1]. It can be tested by extending the diffusion test to longer ages of contact with the salt solution (it can be to 6 or 12 months) and calculate the aging factor from equation (4) or (6).

Pure Portland cements may present aging factors from very low values near 0.1 to around 0.3 depending on its calcium aluminate content. The concretes with mineral additions exhibit much higher values due to the slower reaction of the addition with the Ca(OH)_2 . The range of values can be from 0.3 to 0.8.

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. (6) 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, in that case, by the multiplier $1/(1-m)$ does not appear in Eq. (6).

Table 3 Mean and COV of the aging factor m

Type of cement	Mean	COV
Ordinary Portland cement (OPC) CEM I; $0.40 \leq w/c \leq 0.60$	0.30	0.40
OPC with fly ash (FA) CEM I +FA; FA ≥ 20 wt% cement; $0.40 \leq w/c_{eqv} \leq 0.62$	0.60	0.25
Blast furnace slag cement (GBFS) CEM III/B; $0.40 \leq w/c \leq 0.60$	0.45	0.44

Note (3): absorption zone

In Duracrete it has been proposed that for the splash zone, Δx can be described by a Beta distribution on $[0, 50]$ mm with mean of 9 mm and COV=0.60 for 50 years.
For the submerged zone and the atmospheric zone (spray conditions, e.g., 1.5 m above the road surface), Δx can be treated as a deterministic parameter equal to 0.
For the tidal zone, statistical quantification is provided.

Note (4): chloride concentration at Δx

When the source of chlorides is de-icing salt, $C_{s,\Delta x}$ is described in Duracrete Project by a normal distribution with mean given by the following equation

$$\mu_{C_{s,\Delta x}} = 0.465 - 0.051 \ln(d_h + 1) - \left[0.00065(d_h + 1)^{-0.187} \right] d_v [\text{wt.-%/concrete}] \quad (8)$$

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. (30) is based on data from 5-40 year old concrete structures made of OPC (CEM I) concrete with $0.45 \leq w/c \leq 0.60$ 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

$$\mu_{C_{s,\Delta x}} = C_{s,0} k_{C,binder} k_{C,w/c} k_{C,Cl} k_{C,ec} \quad (31)$$

where $C_{s,0}=2.5\%$ (wt.-%/concrete) is the average chloride concentration at depth Δx 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), $k_{C,binder}$ the parameter accounting for the type of binder, $k_{C,w/c}$ the parameter accounting for the water-cement ratio of the concrete, $k_{C,Cl}$ the parameter accounting for the chloride concentration of sea water and $k_{C,ec}$ the parameter accounting for the exposure class.

The following values of the parameters are suggested:

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

$C_{s,\Delta x}$ 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.

Note (5): critical chloride concentration

The threshold concentration in fresh concrete in numerous standards is of 0.4% by weight of cement although this threshold is not a fixed quantity as it is influenced by several variables. In the case of chlorides, statistical distributions of the Cl_{th} are few in the literature (13-16). That of Hausmann is expressed in function of the Cl^-/OH^- ratio and then it is more difficult to be used. In figure 5 are reproduced two ones (15,16) which are expressed in function of the chloride concentration alone.

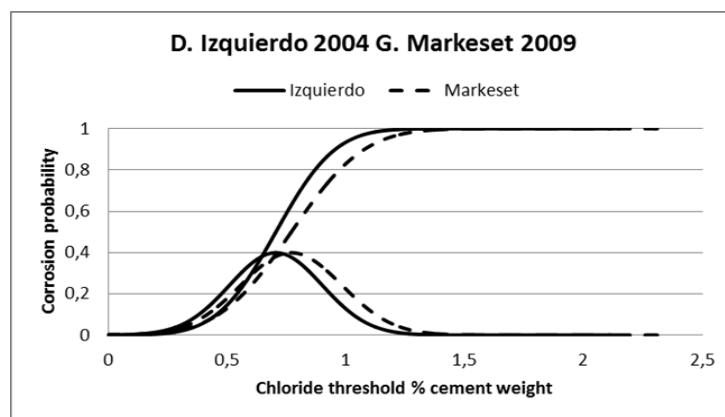


Figure 5. Statistical distributions of the chloride threshold by Izquierdo et al (plain line) made in laboratory conditions and by Markeset (discontinuous line) collected in real structures

One is from Izquierdo et al. (15), it is a normal distribution with a mean of $0,70 \pm 0,20$ and was built from laboratory results made on mortar specimens fabricated with different cement types and submitted to potentiostatic tests in which the chlorides were forced to arrive to the rebar applying a voltage difference. Another interesting distribution is that found by Markeset published in 2004 (16) (figure 5) from the results of a survey made in bridges in Norway where cores were drilled in order to study their state of chloride contamination and the values of chloride at the rebar level when the corrosion was “incipient”. These chloride values with incipient corrosion were considered the site chloride threshold. The results were also statistically treated by Markeset and her distribution is a log-normal with a mean value of 0.77 of chloride ion by cement weight and a COV of 0,25. Instead of a lognormal also a beta-distribution on $[0,2, 2.0]$ can be used

With respect to the submerged or very wet conditions a log-normal distribution with a mean value of $1.53\% \pm 0,53$ (figure 6) (15).

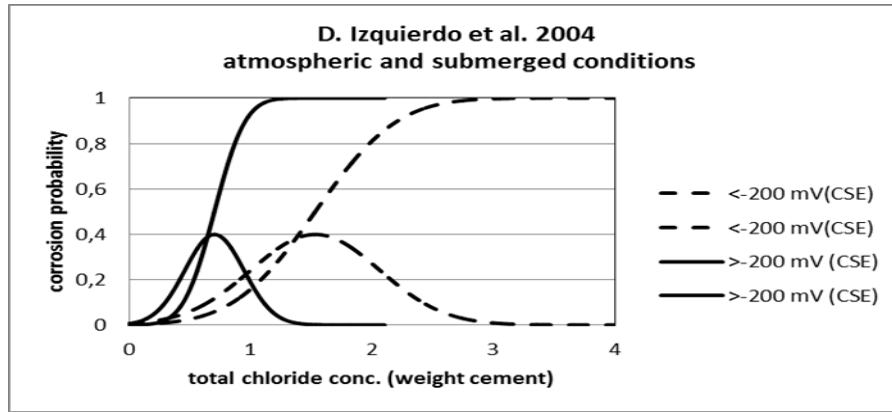


Figure 6 Statistical distributions of the chloride threshold (15) made in laboratory conditions for two ranges of potential: a) more anodic than -200 mV(SCE) typical atmospheric conditions and b) more cathodic than -200 mV(SCE) typical of submerged conditions.

Note (6): cover

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.

Missing?

Information on t_{ref} in (4) and C_i in (5)

REFERENCES

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