

JCSS PMC - MODELS FOR THE CORROSION PROPAGATION STAGE

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1. Corrosion rate model and accumulated corrosion

The corrosion penetration P_{corr} (or change in diameter or radius, $\Delta\phi_{\text{corr}}$ or Δr_{corr} ?) for a period of time t can be calculated through:

$$P_{\text{corr}} = w_t V_{\text{corr}} t$$

P_{corr} = diameter loss

V_{corr} = corrosion rate (mm/year)

t = time

w_t = the relative time of wetness. (=ToW??)

For pitting corrosion, see XX

For existing structures values for V_{corr} may be obtained via measurements. Values of V_{corr} for the design situations may be estimated from:

(1) *Directly from Table 5*

Table 5 Statistical parameter for variables related to carbonation and chloride induced corrosion according to CONTECVET Manual (6) and Duracrete (9).

EXPOSURE CLASSES	V_{corr} [$\mu\text{m}/\text{year}$]			W_v saturation [-]			α (pitting factor)		
	Mean	St.Dev.	Distr.	Mean	St.Dev.	Distr.	Mean	St.Dev.	Distr.
CARBONATION									
Sheltered	2	3	LN	0.20	0.10	N	-2??	-	-
Unsheltered	5	7	LN	0.50	0.20	N	-	-	-
CHLORIDES									
Wet	4	6	LN	1	0.25	N	9.28	4.04	N
Cyclic wet-dry	30	40	LN	0.75	0.20	N	9.28	4.04	N
Airborne seawater	30	40	LN	0.5	0.12	N	9.28	4.04	N
Submerged	4	7	LN	1	0.25	N	9.28	4.04	N
Tidal zone	50	100	LN	1	0.25	N	9.28	4.04	N

(2) Via the concrete water saturation degree

If the water saturation is known the corrosion rate can be calculated from:

$$V_{\text{corr}} \left(\frac{\text{mm}}{\text{year}} \right) = 6 \cdot W_v^2 \quad (\text{xx})$$

W_v = water saturation (see Table 5)

Note that the water saturation may be written as $W_v = S_w \cdot \varepsilon$, where S_w denotes the concrete water saturation degree and ε the porosity in volume. For example, a saturation degree of 50 % in concrete with 10% porosity in volume leads to $W_v = 0,5 \times$

0,1 = 0.05 and a corrosion rate $V_{\text{corr}} = 6 \times 0,05^2 = 0,015 \text{ mm/a} = 15 \text{ } \mu\text{m/a}$ Question: how do we get $W_v=1$ by such a calculation??

The scatter in this estimate may be taken as $\text{COV} = 15\%$.

(3) Via the concrete electric resistivity

According to (Andrade 1980):

$$V_{\text{corr}} = \frac{0.0116 \cdot 26000}{\rho_{\text{ef}}} = \frac{301.6}{\rho_{\text{ef}}} \quad (\text{xx})$$

With V_{corr} in [mm/year] and ρ the concrete resistivity in [ohm·cm]. The concrete resistivity is time-variant and depends on a number of factors

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

$$k_{R,T} = \exp[b_{R,T}(1/T_{\text{real}} - 1/T_0)]$$

$$k_{R,Cl} = 1 - (1 - a_{Cl})/2$$

ρ_0 = the specific electrical resistivity of concrete at time t_0 ,

$b_{R,T}$ = regression variable,

T_{real} = the average temperature in [K],

T_0 = 293 K,

$k_{R,RH}$ = the relative humidity factor,

a_{Cl} = chloride factor

t_0 = 28 d

t = the age of concrete),

n = the age factor for resistivity.

Table 6 shows the statistical properties.

Table 6 Statistical parameters of random variables related to corrosion rate

Variable	Mean	COV	Distribution
$K_{\text{????}}$	2.63	1.335	Shifted lognormal, min. 1.09
ρ_0 $f'_c=30 \text{ MPa (OPC)}$	116 Ωm	0.16	Normal
$f'_c=40 \text{ MPa (OPC)}$	134 Ωm	0.16	Normal
$f'_c=50 \text{ MPa (OPC)}$	155 Ωm	0.16	Normal
$b_{R,T}$	3815 K	0.15	Lognormal
$k_{R,RH}$ in splash zone	1.0	-	Deterministic
on coast	1.07	0.13	Lognormal
α_{Cl} ($C < 2\%$)	0.72 ???	0.153 ???	Normal
$k_{R,Cl}$ ($C \geq 2\%$)	0.72 ???	0.153 ???	Normal
n	0.23	0.174	Normal

(4) **Via Diffusivity**

V_{corr} can also be expressed in function of the V_{CO_2} (??) and chloride of the diffusion coefficient

carbonation	$V_{corr} \left(\frac{mm}{year} \right) = 0,0011 \cdot V_{CO_2} \left(\frac{mm}{year^{0.5}} \right) - 0,0015$	(38)
chlorides	$V_{corr} \left(\frac{mm}{year} \right) = 300000 \cdot D_{ap} \left(\frac{cm^2}{s} \right)$	(39)

A COV of 15% can be considered for either V_{corr} , V_{CO_2} and D_{ap} .

2. Limit state of crack initiation

Corrosion products occupy a larger volume than the consumed steel. This eventually leads to cracking of the concrete cover. An appropriate model is 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, $\varnothing$. Expansion of the corrosion products is represented by uniform pressure applied to the inner surface of the cylinder (see Figure 2).

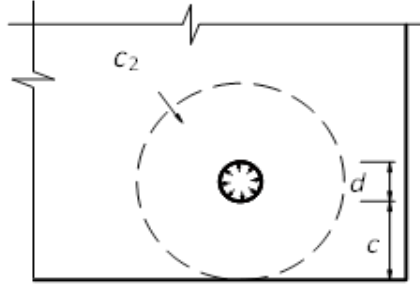


Figure 7. Thick-walled cylinder model.

A relationship between the expansion of corrosion products $\Delta\varnothing_{\text{exp}}$ (or $\Delta r_{(s)}$??) (i.e., increase of the bar diameter) and the radial pressure, p , caused by it is given by the following equation:

$$(1-\eta) \Delta d = \frac{d}{E_{c,ef}} \left[1 + \nu_c + \frac{d^2}{2c(c+d)} \right] P \quad (42)$$

$$E_{c,ef} = E_c / (1 + \varphi)$$

- d = diameter ($\varnothing$)
- c = cover
- p_r = radial pressure
- ν_c = Poisson's ratio of concrete,
- E_c = modulus of elasticity of the concrete at age of 28 days
- φ = concrete creep coefficient.
- η = the relative fraction of the corrosion products diffused into concrete

The parameter η reflects that not all corrosion products contribute to the pressure exerted on the surrounding concrete since a part of them diffuses into concrete pores and microcracks. The expansion of corrosion products can be estimated as (e.g., Chernin et al. 2010)

$$\Delta d = 2(\alpha_v - 1) V_{corr} \Delta t \quad [\mu\text{m}] \quad (44)$$

- α_v = the volumetric expansion ratio of corrosion products
- V_{corr} = the corrosion rate in $[\text{mm/a}]$
- Δt = the time since corrosion initiation in $[\text{a}] = t_{\text{prop}} = t - t_{\text{ini}}$

The concrete cover is considered as fully cracked when the average tensile stress in the cylinder wall becomes equal to the strength of concrete. So the limit state function is given by:

$$g(X, t) = f_{ct} - \sigma_{tt},$$

The average concrete tangential tensile stress can be found from:

$$\sigma_{tt} = p_r \varnothing / 2c$$

f_{ct} = tensile strength of concrete,

Statistical properties of α_v and η are given in **Table 7**.

Table 7 Statistical parameters of random variables related to cover cracking

Variable	Mean	COV	Distribution
α_v	3.0	0.30	Beta on [2.,6.4]
η	0.70	0.30	Beta on [0.,0.9]
ν	0.20	-	-
$E_{c,ef}, f_{ct}$	See Section 3.01 of PMC		
c	See Section xxx		

Note: 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 8)

$$1.5 \leq c_1/d \leq 4 \text{ or } 1.5 \leq c_2/c_1 \leq 3.5$$

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

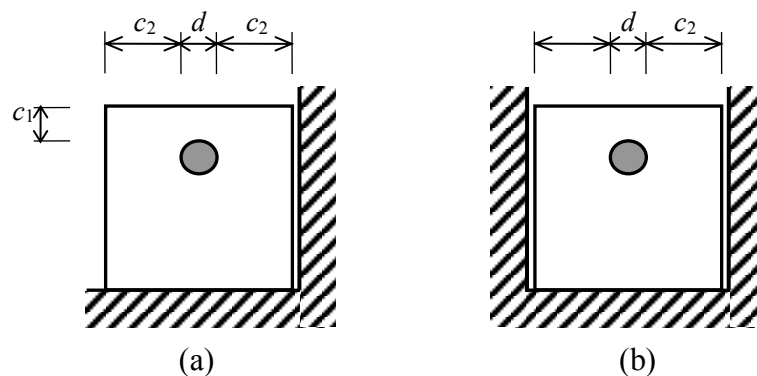


Figure 8 Fragments of reinforced concrete cross-section representing different locations of a reinforcing bar: (a) corner bar; (b) internal bar.

3. Cracking up to a limit value (SLS)

The existing models relating crack evolution and corrosion level are currently not sufficiently accurate. Next is mentioned the empirical expression included in Duracrete project and developed in Contecvet Manual X) proposed by Rodríguez (x) for crack evolution. Assuming an initial “hair” crack of width= 0.05 mm at the concrete surface already forms the further progression of the crack width is formulated (for cracks smaller than 1 mm ??):

$$w = 0.05 + \beta [P_{corr} - P_{corr,0}] \quad (xx)$$

$w(\Delta t)$ the crack width [mm]

β factor that controls the propagation depending on the bar position. (table 8) [-]

P_{corr} the attack penetration [mm] at Δt

$P_{corr,0}$ the attack penetration which produce a 0.05 mm crack width [mm]; this is the parameter that control the cracking initiation (cracks of 0.05 mm were considered as initiation of “visible” cracks in the concrete cover, during the experimental work).

Δt time after corrosion initiation (t_{prop})

Table 8 Values of β for crack width calculations

	Mean Values		Characteristic Values	
	Upper Bars	Lower Bars	Upper Bars	Lower Bars
β	0.0086	0.0104	0.01	0.0125

The limit state function for exceeding the SLS crack limit during some given period of time t can be expressed as:

$$g(X,t) = w_{limit} - w(\Delta t) \quad (xx)$$

w_{limit} = crack width; limit eg 1 mm

$w(\Delta t)$ = crack width as a result of corrosion during Δt

t = period under consideration e.g. design service life [a],

Δt = time since corrosion initiation in [a] = $t_{prop} = t - t_{ini}$

t_{ini} = initiation period [a], (see 2.2.1)

4. Reduction of load bearing capacity (ULS/collapse)

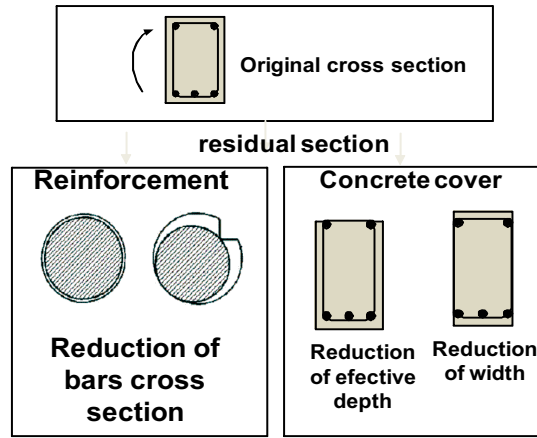


Figure 11 Reduced cross section has to be calculated for the verification of the SLS and the ULS.

Limit state function

The LSF for bending (neglecting shear) may be written as:

$$g(X,t) = M_R - M_E(G, Q)$$

M_R = Bending moment resistance, based on

M_E = Bending moment due to permanent and variable mechanical actions.

The value of M_R may be taken as its initial value as long as corrosion has not started, After corrosion initiation, the resistance of the rebars reduces depending on the corrosion rate and the type of corrosion. The failure probability can be calculated using the methods for time dependent reliability problems described in Part I of the PMC. An upperbound for the period (0,t) may be found form:

$$g(X) = M_R(t) - \max_t M_E(G, Q(t))$$

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 (CO₂)

The cross-sectional area of a reinforcing bar with the original diameter $\varnothing$ (or d) after a period Δt or t_{prop} of general corrosion will be

$$A_s(t) = \frac{\pi [d - 2 \times 10^{-3} V_{corr} t]^2}{4} \geq 0 \quad [\text{mm}^2] \quad (54)$$

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 relation α between penetration of pitting, P_{max}/P_{ave} , on the surface of a rebar is about 4 to 8 times the average penetration, P_{ave} , 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 $R=p_{max}/p$ between 4 and 10 for 5-mm and 10-mm reinforcing bars of 150 mm to 300 mm length. Thus, as said, the depth of a pit, P_{pit} (which is equivalent to the maximum penetration of pitting) after t years since corrosion initiation is evaluated as:

$$P_{pit} = P_{corr} \cdot \alpha \quad (55)$$

There is significant uncertainty associated with the geometry, number of pits and the ratio between the maximum pit depth and the average corrosion penetration. In Duracrete it was found (on the basis of 51 samples) that the best fitted distributions in the α values were the normal and Weibull distributions, with small differences between them. Then, it is possible to accept the normal as the best fitted distribution. The results are shown in table 12.

Table 12 Characterization of the pitting factor $\alpha = P_{max}/P_{ave}$

s	Mean	CoV	Distribution
Duracrete	9.28	0,45	Normal
Stewart (8 mm bar)	6	0,20	Gumbel

Other researchers have indicated the Gumbel distribution as best approximation. Stewart (2004) has found results for an 8 mm diameter bar of 125 mm length (see Table 12). For other geometrical parameters::

$$\mu = \mu_0 + \frac{1}{\alpha_0} \ln \left(\frac{A}{A_0} \right), \quad \alpha = \alpha_0 \quad (57)$$

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)

$$f_y(t) = [1 - \alpha_y Q_{corr}(t)] f_{y,0} \quad (58)$$

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

$$Q_{\text{corr}}(t) = A_p(t) / A_p(0)$$

A review of experimental studies shows that α_y can be in a range from 0.0 to 0.017, with average around 0.005. α_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 $Q_{\text{corr}(t)}$ exceeds a threshold value, Q_{limit} . This leads to two types of mechanical behaviour:

$$\begin{aligned} \text{Ductile behaviour: } Q_{\text{corr}} &\leq Q_{\text{limit}} \\ \text{Brittle behaviour: } Q_{\text{corr}} &> Q_{\text{limit}} \end{aligned} \tag{60}$$

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

Being the bond the crucial parameter in the performance of the corroding elements, next it is presented as a prior verification.

6. 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):

$$\tau = \begin{cases} \tau_{\max} \cdot \left(\frac{s}{s_1} \right)^\alpha & 0 \leq s \leq s_1 \\ \tau_{\max} & s_1 \leq s \leq s_2 \\ \tau_{\max} - \frac{\tau_{\max} - \tau_f}{s_2 - s_3} (s - s_2) & s_2 \leq s \leq s_3 \\ \tau_f & s \geq s_3 \end{cases} \quad (47)$$

Parameters of Eq. (47) are given in Table 10, except of $\tau_{\max}$.

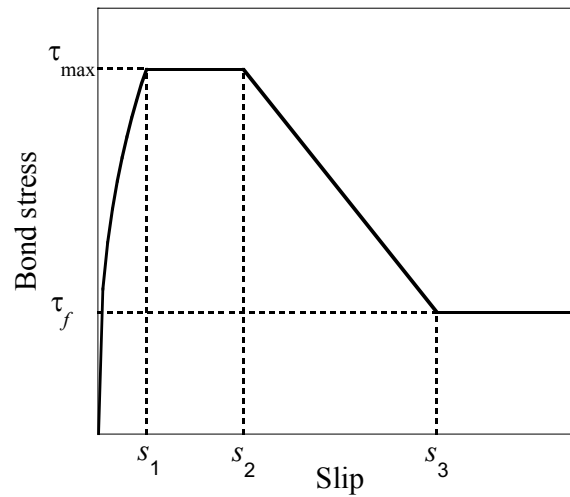


Figure 9 Bond stress-slip relationship.

Table 10 Parameters of Eq. (47).

	Unconfined concrete*		Confined concrete [#]	
	Good bond conditions	All other bond conditions	Good bond conditions	All other bond conditions
s_1	0.6 mm	0.6 mm	1.0 mm	1.0 mm
s_2	0.6 mm	0.6 mm	3.0 mm	3.0 mm
s_3	1.0 mm	2.5 mm	Clear rib spacing	Clear rib spacing
α	0.4	0.4	0.4	0.4
τ_1	$2.0\sqrt{f_c}$	$1.0\sqrt{f_c}$	$2.5\sqrt{f_c}$	$1.25\sqrt{f_c}$
τ_f	$0.15\tau_1$	$0.15\tau_1$	$0.40\tau_1$	$0.40\tau_1$

*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, ρ , does not exceed 0.25; confined concrete corresponds to $\rho > 1$. This ratio is defined as

$$\rho = \frac{A_{st}}{nA_s} \quad (48)$$

where A_{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. (47) can be estimated by interpolation of the values given in Table 10.

Uncertainty associated with Eq. (47) 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 τ obtained from Eq. (47) 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, $\tau_{\max}/\tau_{\max,0}$, representing the ratio of the bond strength with corrosion, $\tau_{\max}$, to the initial bond strength, $\tau_{\max,0}$, and the corrosion penetration (i.e., the reduction in radius of the uncorroded reinforcing steel), p , is proposed (see also Figure 10)

$$\frac{\tau_{\max}}{\tau_{\max,0}} = \begin{cases} 1 + (k_1 - 1) \frac{p}{p_{cr1}}, & p \leq p_{cr1} \\ \max[k_1 - k_2(p - p_{cr1}), 0.15], & p > p_{cr1} \end{cases} \quad (49)$$

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

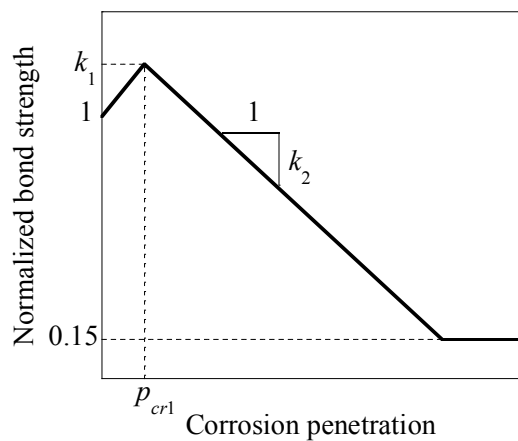


Figure 10 Normalized bond strength versus corrosion penetration.

If $\tau_{\max} < \tau_i$ then Eq. (17) becomes

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

The corrosion penetration, p , at time t and the corrosion penetration corresponding to crack initiation, p_{cr1} , are estimated as

$$p = V_{corr} t \quad [\mu\text{m}] \quad (50)$$

$$p_{cr1} = V_{corr} t_{cr1} \quad [\mu\text{m}] \quad (51)$$

where V_{corr} is calculated using that can be calculated through Table 5, or expressions, 33, 34.36, 37, 38 or 39.

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 ρ (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 ρ 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)

$$k_1 = \begin{cases} 1, & c/d \leq 1 \\ 1 + 0.085(c/d - 1), & c/d > 1 \end{cases} \quad (52)$$

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 ρ) 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 $0.005 \mu\text{m}^{-1}$. 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 ($\rho > 1$) $k_2 = 0.0025 \mu\text{m}^{-1}$, 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 μm^{-1}) and ρ can be summarized as (Val and Chernin 2009)

$$k_2 = \begin{cases} 0.005, & \rho \leq 0.25 \\ 0.005 - \frac{\rho - 0.25}{300}, & 0.25 < \rho \leq 1 \\ 0.0025, & \rho > 1 \end{cases} \quad (53)$$

Statistical properties of the random variables representing uncertainties associated with the bond stress-slip relationship, Eq.(47), are k_1 and k_2 are given in Table 11.

Table 11 Statistical parameters of random variables related to bond strength

Variable	Mean	COV	Distribution
Bond model uncertainty – Eq. (47)	1.0	0.30	Normal
k_1	Eq. (22)	0.20	Normal, truncated at 1
k_2 (μm^{-1})	Eq. (23)	0.20	Normal

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ANNEX

Bond strength of corroded bars, alternative

From the models in Duracrete project taken from Contecvet Manual, three main aspects should be considered (Rodriguez et al. 1994):

- Residual bond assessment. Table X shows empirical expressions obtained by (Rodriguez et al. 1994) based on test type developed by (Chana 1990) that allow to obtain realistic residual bond values. All of them are expressed depending on the attack penetration P_x .

Table X Relationship between bond and P_x in mm

	Bond strength (MPa)	
	With stirrups	No stirrups
Mean values	$5.25 - 2.72 P_x$	$3.00 - 4.76 P_x$
Characteristic values	$4.75 - 4.64 P_x$	$2.50 - 6.62 P_x$

For intermediate cases where the amount of stirrups is low, below the actual minimum, or the stirrups capacity can be strongly reduced by corrosion effect, expressions of table X may be applied.

Table X. Bond values for cases of intermediate amount of stirrups (expression XX)

	Mean Values	Characteristic Values
f_b	$8.25 + m(1.10 + P_x)$	$10.04 + m(1.14 + P_x)$
m	$-4.76 + 2.04(\rho/0.25)$	$-6.62 + 1.98(\rho/0.25)$
ρ	$n [(\phi_w - \alpha x)/\phi]^2$	$n [(\phi_w - \alpha x)/\phi]^2$

where ϕ = the initial longitudinal diameter in mm.; ϕ_w = the transversal diameter in mm.; n = the number of transversal reinforcements; α depends on the type of corrosion; and f_b = bond strength.

These expressions are of application with P_x values between 0,05 and 1 mm with $\rho \leq 0.25$.

- Influence of external pressures that can be present due to external supports. In this case, expressions similar to that presented in Eurocode 2 have been developed by:

$$f_b = (4.75 - 4.64 P_x) / (1 - 0.098 p)$$

Where f_b is the bond in MPa, P_x is the corrosion attack in mm y p the external pressure in the bond zone (MPa). This expression can be used for the bond evaluation of the rebar at element ends.

- Relationship between bond and crack width. Several expressions have been developed for relating the residual bond with the crack width (table X6).

Table X6 Relationship bond f_b (MPa) and crack width w (mm).

	Stirrups	No stirrups
Mean values	$f_b = 18 - 0.52 w$	$f_b = 3.19 - 1.06 w$
Characteristic values	$f_b = 4.66 - 0.95 w$	$f_b = 2.47 - 1.58 w$

The summary of the parameters of this model are given in table X. The statistical characterization was not made during Duracrete project.

The ratio of corroded transverse reinforcement to main reinforcement at anchorage length (ρ) is estimated from the following expression: $\rho = n \cdot \left[(\phi_w - \alpha \cdot x) / \phi \right]^2$ The residual bond strength f_b (MPa) is then calculated from:		
$\rho > 0.25$ (a minimum amount of stirrups greater than that specified in Eurocode 2)	$0.25 > \rho > 0$ (intermediate values)	$\rho = 0$ (no stirrups)
$f_b = 4.75 - 4.64 x$	$f_b =$ $10.04 + [-6.62 + 1.98 (\rho/0.25)]$ $[1.14 + x]$	$f_b = 2.50 - 6.62 x$
Description of parameters: ϕ is the diameter of the main bar (mm) ϕ_w is the diameter of the transverse bar (mm) n is the No. of transverse bars at anchorage length of main bar α is a constant related to the type of corrosion (see Section 3.5.2) x is the attack penetration (mm)		
Where an external pressure (p in MPa) exists at anchorage length, the expression below applies: $f_b = (4.75 - 4.64 x) / (1 - 0.08 p)$		