Probabilistic lifetime assessment of RC structures under coupled corrosion–fatigue deterioration processes

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**Abstract**

Structural deterioration is becoming a major problem when considering long-term performance of infrastructures. The actions of corrosive environment, cyclic loading and concrete cracking lead to structural degradation. The interaction between these conditions can only be taken into account when modeling the coupled phenomena. In this paper is proposed a new model to assess the lifetime of RC structures subject to corrosion–fatigue deterioration processes. Separately, corrosion leads to cross-section reduction while fatigue induces the nucleation and the propagation of cracks in steel bars. When considered together, pitting corrosion nucleates the crack while environmental factors affect the kinematics of crack propagation. The model is applied to the reliability analysis of bridge girders located in various chloride-contaminated environments. Overall results show that the coupled effect of corrosion–fatigue on RC structures strongly affects its performance, leading to large reduction in the expected lifetime.

**1. Introduction**

Long-term performance of infrastructures is governed by structural deterioration, which is defined as the loss of capacity due to physical, chemical, mechanical or biological actions. Since corrosive environments and cyclic loading are among the main causes of reinforced concrete (RC) deterioration, a significant amount of research has been devoted to these two specific damage mechanisms [1–3]. Corrosion is the most common form of steel deterioration and consists in material disintegration as a result of chemical or electrochemical actions. Most metals corrode on contact with water (or moisture in the air), acids, bases, salts, and other solid and liquid chemicals. Metals will also corrode when exposed to gaseous materials like acid vapors, formaldehyde gas, ammonia gas, and sulfur containing gases [3]. Depending on the case, corrosion can be concentrated locally to form a pit, or it can extend across a wide area to produce general wastage. On the other hand, fatigue is the damage of a material resulting from repeated stress applications (e.g., cyclic loading). Fatigue is conditioned by many factors such as high temperature, i.e., creep–fatigue, and presence of aggressive environments, i.e., corrosion–fatigue [1,2].

The damage to RC structures resulting from the corrosion of reinforcement is exhibited in the form of steel cross-section reduction, loss of bond between concrete and steel, cracking, and spalling of concrete cover [4,5]. The corrosion of steel reinforcement has been usually associated with chloride ingress and carbonation [4]; however, recent studies have shown that other deterioration processes like biodeterioration might contribute significantly to this process [6]. In RC structures, the coupled effect of corrosion and fatigue has not been studied in as much detail as their separated effects. Coupled corrosion–fatigue deterioration results from the combined action of cycling stresses in corrosive environments. Localized corrosion leading to pitting may provide sites for fatigue crack initiation. Several experimental studies have shown that pitting corrosion has been responsible for the nucleation of fatigue cracks in a wide range of steels and aluminum alloys [7–9]. In such studies, pits are usually found at the origin of the fracture surface. Corrosive agents (e.g., seawater) increase the fatigue crack growth rate [10], whereas the morphology of metals/alloys at micro-level governs the pit nucleation sites [11]. Under these conditions, the formation and growth of pits is influenced by both a corrosive environment and cyclic loads and become a coupled damage mechanism.

Examples of structures that experience this type of damage are offshore platforms, bridges, chimneys and towers situated close to the sea or exposed to the application of de-icing salts. The effects of gradually accumulated corrosion on the low cycle fatigue of reinforcing steel have been recorded experimentally by Apostolopoulos et al. [12] showing that corrosion implies an appreciable reduction in the ductility, the strength and the number of cycles to failure.
Large research efforts have been made to predict the corrosion–fatigue life of structural members constituted by aluminum, titanium and steel alloys. Goswami and Hoeppner [13] proposed a seven-stage conceptual model in which the electrochemical effects in pit formation and the role of pitting in fatigue crack nucleation were considered. Other research studies focused on particular stages of the process. For instance, a transition model from pit to crack based on two criteria: stress intensity factor and competition between pit growth and crack growth, was proposed by Kondo [7], and further discussed by Chen et al. [14]. In order to take into consideration the entire progressive damage process and the uncertainties in each stage, Shi and Mahadevan [15] proposed a mechanics-based probabilistic model for pitting corrosion–fatigue life prediction of aluminum alloys.

The objective of this paper is to combine previous works on corrosion and fatigue to develop a probabilistic lifetime prediction model for RC structures under the coupled effect of corrosion and fatigue. The model assesses the total corrosion–fatigue life as the sum of three critical stages: (1) corrosion initiation and pit nucleation; (2) pit-to-crack transition, and (3) crack growth. The first considers the time from the end of construction until the generation of a pit. The length of this stage is estimated by considering Fick’s diffusion law and electrochemical principles. The second stage includes the pit growth until crack nucleation. In this stage the interaction between electrochemical and mechanical processes is taken into account. The latter stage covers the time of crack growth until reaching a critical crack size, which is defined as the crack size at which the RC member reaches a limit state of resistance.

The proposed model is described in Section 2. Section 3 presents a discussion about the probabilistic lifetime assessment and the reliability analysis. Finally, an application to bridge girders is given in Section 4.

2. Coupled corrosion–fatigue model

The corrosion–fatigue damage process in RC structures is conceptually depicted in Fig. 1. The process takes into account the interaction between (1) chloride ingress, (2) RC cracking and (3) cyclic loading. Chloride ingress leads to steel depassivation, and takes part in the kinematics of the corrosion process. Besides, the corrosion resulting from chloride ingress induces high localized corrosion (i.e., pitting corrosion), leading to reinforcing steel crack nucleation [7–9]. Concrete cracking generated by the accumulation of corrosion products in the steel/concrete interface plays an important role in the steel corrosion rate. Its importance depends on both the width of the crack in the concrete and the aggressiveness of the environment. On the other hand, cyclic loading governs the transition from pit to crack as well as the crack growth.

The corrosion–fatigue deterioration process is basically divided into two stages: (1) pit formation and growth and (2) fatigue crack growth. Pit formation and growth involves electrochemical processes depending predominantly on environmental factors. Crack growth is estimated in terms of linear elastic fracture mechanics (LEFM) and depends mostly upon both cyclic loads and material properties. Goswami and Hoeppner [13] proposed to separate conceptually the corrosion–fatigue life into the following stages: (1) electrochemical stage and pit nucleation, (2) pit growth, (3) competitive mechanisms between pit growth and fatigue crack nucleation, (4) chemically, “short” crack growth, (5) transition from “short crack” to “long crack”, (6) long crack growth, and (7) corrosion–fatigue crack growth until instability. However, assuming initial immunity of RC structures and the fact that some of these stages proposed by Goswami and Hoeppner are transitional stages, the total corrosion–fatigue life, $\tau_f$, will be divided into the following stages (Fig. 1):

1. corrosion initiation and pit nucleation, $\tau_{cp}$,
2. pit-to-crack transition, $\tau_{pt}$, and
3. crack growth, $\tau_c$.

2.1. Corrosion initiation and pit nucleation

This stage is divided into two sub-stages:

1. time to corrosion initiation, $\tau_{ini}$, and
2. time to pit nucleation, $\tau_{pn}$.

The first sub-stage describes the time from the end of construction until the depassivation of the corrosion protective layer of reinforcing steel, and subsequently, corrosion initiation. For RC

![Fig. 1. Scheme of corrosion–fatigue deterioration process in RC structures.](https://example.com/fig1.png)
structures, the length of this stage depends mainly upon the concrete characteristics and the thickness of the cover. It is calculated by using Fick’s second law under the assumption that concrete is a homogenous and isotropic material [16]:

\[
\frac{\partial C}{\partial t} = D_{cl} \frac{\partial^2 C}{\partial x^2}
\]

(1)

where \( C \) is the chloride ion concentration, \( D_{cl} \) is the chloride diffusion coefficient in concrete, \( t \) is the time and \( x \) is the depth in the diffusion direction. By defining the following initial conditions (1) at \( t = 0 \), the chloride concentration is zero; and (2) the chloride concentration on the surface is constant, the solution to Eq. (1) leads to the chloride ion concentration \( C(x, t) \) at the depth \( x \) for an exposure time \( t \):

\[
C(x, t) = C_s \left[ 1 - \text{erf} \left( \frac{x}{2 \sqrt{D_{cl} t}} \right) \right]
\]

(2)

where \( C_s \) is the surface chloride concentration and \( \text{erf}(\cdot) \) is the error function. The threshold concentration \( C_{th} \) is defined as the chloride concentration for which the rust passive layer of steel is destroyed and the corrosion begins. When \( C(x, t) \) is equal to \( C_{th} \) and the depth \( x \) is equal to the concrete cover \( c \), i.e., \( x = c \), the time to corrosion initiation, \( t_{ini} \), becomes

\[
t_{ini} = \frac{c^2}{4D_{cl}} \text{erf}^{-1} \left( 1 - \frac{C_{th}}{C_s} \right)^2
\]

(3)

Pit nucleation is the result of an electrochemical process induced by corrosion. Computing the time to pit nucleation, \( t_{pn} \), is a non-trivial task because it depends on several environmental, material and loading factors whose interaction is not well understood yet. The pit depth at time \( t \), \( p(t) \) (Fig. 2a), can be calculated as [3]

\[
p(t) = 0.0116 \int_{0}^{t} k_{cor}(t') \, dt'
\]

(4)

where \( p(t) \) is given in mm, \( x \) is the ratio between pitting and uniform corrosion depths, and \( k_{cor}(t') \) is the time-variant corrosion rate which is given in \( \mu A/cm^2 \). Time to pit nucleation is estimated by defining a threshold, \( p_0 \), for the pit depth, \( p(t) \). For example, according to Harlow and Wei [17], this value is \( p(t_{pn}) = p_0 = 1.98 \times 10^{-6} \) m. During a brief time period after steel depassivation, i.e., \( t_{ini} + 1 \) year \( \geq t \geq t_{ini} \), it is reasonable to assume that \( k_{cor}(t) \) remains constant (i.e., \( k_{cor}(t) = k_{ini} \)) and equal to [18]

\[
k_{ini} = \frac{37.8(1 - wc)^{1.64}}{c}
\]

(5)

where \( wc \) is the water/cement ratio and \( c \) is given in mm. Consequently, by making \( p(t_{pn}) = p_0 \) in Eq. (4), substituting Eq. (5) in Eq. (4) and by integrating, the time to pit nucleation, \( t_{pn} \), can be written as

\[
t_{pn} = \frac{2.281cp_0}{x} (1 - wc)^{1.64}
\]

(6)

Eq. (6) gives a simple relationship to estimate \( t_{pn} \) based on electrochemical principles. Other proposals to obtain \( t_{pn} \) can be found in the literature [15]; however, this discussion is beyond the scope of this paper.

2.2. Pit-to-crack transition

The time of transition from pit to crack, \( t_{pt-c} \), is defined as the time at which the maximum pit depth reaches a critical value leading to crack nucleation. Crack nucleation depends on the competition between the processes of pit and crack growth. To estimate this transition period, two criteria can be taken into account [7,14]:

1. Rate competition criterion: the transition takes part when the crack growth rate, \( da/dt \), exceeds the pit growth rate, \( dp/dt \), as illustrated in Fig. 2b.
2. Fatigue threshold criterion: the transition occurs when the stress intensity factor of the equivalent surface crack growth for the pit, \( AK_{eq} \), reaches the threshold stress intensity factor for the fatigue crack growth, \( AK_c \).

In this study, the fatigue threshold criterion was not taken into consideration because experimental observations indicate that this criterion is not appropriate to estimate fatigue crack nucleation at low loading frequencies [14]. Therefore, the rate competition criterion where pit growth rate is described by electrochemical mechanisms and fatigue crack growth rate is estimated in terms of LEFM will be discussed in the following sections. This discussion will focus on:

- pit growth rate,
- fatigue crack growth rate,
- computation of pit-to-crack transition.

2.2.1. Pit growth rate

After pit nucleation and as a result of a localized galvanic corrosion, pit growth can be estimated in terms of a change in the volumetric rate by using Faraday’s law [7,17]:

![Fig. 2. (a) Time-variant corrosion rate and (b) rate competition criterion.](image_url)
\[ \frac{dV}{dt} = \frac{M_{i_{\text{corr}}}}{nF} \]  
where \( M \) is the molecular weight of iron, \( i_{\text{corr}} \) is the corrosion rate, \( n \) is the valence of iron, \( F \) is the Faraday's constant, \( \rho \) is the density of iron, and \( T \) is the absolute temperature. It is clear, from Eq. (8), that the corrosion rate increases when temperature is raised. Nevertheless, given the complexity of the corrosion process in RC, the relationship between corrosion rate and temperature is taken into account by using the Arrhenius equation:

\[ i_{\text{corr}} = i_{\text{p}} \exp \left( \frac{-\Delta H}{RT} \right) \]  

where \( i_{\text{p}} \) is the pitting current coefficient, \( \Delta H \) is the activation enthalpy, \( R \) is the universal gas constant \( (R = 8.314 \text{ mol} \cdot \text{K} \cdot \text{mol}^{-1}) \), and \( T \) is the absolute temperature. It is known that there is a strong correlation between corrosion rate and temperature.

1. During the period corresponding to the initial corrosion age, the initial corrosion rate function, \( i_{\text{ini}}(t) \), is estimated as [18]:

\[ i_{\text{ini}}(t) = \begin{cases} \frac{378(1-1.04^{t-16})}{32.12(1-1.04^{t-44})} & \text{for } t < t_{\text{ini}} + 1 \text{ year} \\ \frac{378(1-1.04^{t-16})}{32.12(1-1.04^{t-44})} & \text{for } t > t_{\text{ini}} + 1 \text{ year} \end{cases} \]  

2. During the period corresponding to the active corrosion age, \( i_{\text{cor}} \), leads to a threshold corrosion rate, \( i_{\text{th}} \). The value of \( t_{\text{ini}} \) depends mainly on environmental aggressiveness. Namely, \( t_{\text{ini}} \) is a function of the availability of water, oxygen and chloride concentration at the corrosion cell.

Environmental aggressiveness takes an active role in the corrosion rate after severe cracking of concrete. The influence of environmental aggressiveness on corrosion rate is estimated on the basis of empirical results. For instance, Schiessl and Raupach have measured a significant increase of the corrosion rate when the concrete crack reached a limit value (i.e., 0.5 mm) [21]. The time to reach this limit crack size is called time to severe cracking, \( t_{\text{ini}} \), which is divided into two stages: (1) crack initiation and (2) crack propagation until a limit value. The length of the former stage is estimated by using the model proposed by Liu and Weyers [22] and the length of the latter is computed from an empirical formula found by Vu et al. [23]. If the membership functions are \( \mu_{\text{ini}}(t) \) for the initial corrosion age and \( \mu_{\text{cor}}(t) \) for the active corrosion age, the time-variant corrosion rate considering concrete cracking and environmental aggressiveness takes the form:

\[ i_{\text{cor}}(t) = \mu_{\text{ini}}(t)i_{\text{ini}}(t) + \mu_{\text{cor}}(t)i_{\text{cor}}(t) \]  

\[ \frac{d\sigma}{dt} = 0.0116 i_{\text{cor}}(t) \]  

where \( \sigma \) is the stress range, \( N \) is the number of cycles, \( \Delta K \) is the alternating stress intensity factor, and \( C_p \) and \( m \) are material constants. It is known that there is a strong correlation between \( C_p \) and \( m \), and their values are highly depending on the environmental aggressiveness. For RC structures, Salah el Din and Lovegrove [24] reported experimental values for \( C_p \) and \( m \) corresponding to medium and long crack growth stages, leading to

\[ \frac{da}{dN} = \begin{cases} 3.83 \times 10^{-29}(\Delta K)^{1.063} & \text{if } \Delta K < 9 \text{ MPa} \sqrt{\text{m}} \\ 3.16 \times 10^{-11}(\Delta K)^{1.143} & \text{otherwise} \end{cases} \]  

where \( \sigma \) is the stress range, \( N \) is the number of cycles, \( \Delta K \) is the alternating stress intensity factor, and \( C_p \) and \( m \) are material constants. It is known that there is a strong correlation between \( C_p \) and \( m \), and their values are highly depending on the environmental aggressiveness. For RC structures, Salah el Din and Lovegrove [24] reported experimental values for \( C_p \) and \( m \) corresponding to medium and long crack growth stages, leading to

\[ \Delta \sigma = \Delta \sigma Y(a/d_0)\sqrt{\pi a} \]  

where \( \Delta \sigma \) and \( \Delta \sigma \) are stress range and stress intensity factor, respectively, \( Y(a/d_0) \) is the dimensionless geometry notched specimen function, which can be approximated by [25]

\[ Y(a/d_0) = \frac{1.121 - 3.08(a/d_0) + 7.344(a/d_0)^2 - 10.244(a/d_0)^3 + 5.85(a/d_0)^4}{[1 - 2(a/d_0)^2]^{1/2}} \]  

The dotted line in Fig. 2b follows the shape of the fatigue crack growth rate as a function of time, which starts growing after corrosion initiation and pit nucleation.

2.2.3. Computation of pit-crack transition

In order to find the time for pit-crack transition, \( t_{\text{pit}} \), an equivalent stress intensity factor for the pit, \( \Delta K_{\text{pit}} \), must be estimated. \( \Delta K_{\text{pit}} \) is found by substituting \( a \) in Eq. (14) by the pit depth \( p(t) \) (Eq. (4)):
Then, the equivalent crack growth rate becomes

$$\frac{da}{dt} = C_p[\Delta K_{pit}(t)]^{n}f$$

(17)

where $f$ is the frequency of the cyclic load. Therefore, the time of pit-to-crack transition is obtained by equating the pit growth rate (Eq. (11)) with the equivalent crack growth rate (Eq. (17)), and solving for $\tau_{pt}$

$$0.0116 \alpha \sigma_{\text{corr}}(\tau_{pt}) = C_p[\Delta K_{pit}(\tau_{pt})]^{n}f$$

(18)

In order to calculate $\tau_{pt}$, Eq. (18) must be solved numerically. Fig. 2b illustrates the graphical solution for $\tau_{pt}$, where pit-to-crack transition is found at the intersection between the continuous and dotted lines.

2.3. Crack growth

Crack growth modeling is concerned with the time from the crack initiation until the crack size has reached a value that defines the failure of the cross-section. The size of the initial crack on the steel reinforcement, $a_0$, is estimated as the pit depth when the transition from pit to crack is reached, i.e., $a_0 = p(\tau_{pt})$ (Eq. (4)). The size of the critical crack, $a_c$, is defined as the crack at which the RC member reaches a limit state of resistance (e.g., bending capacity). This time is obtained by integrating Eq. (13):

$$\tau_{cg} = \begin{cases} \int_{a_0}^{a_1} \frac{da}{K_{pit}(a)} + \int_{a_1}^{a_c} \frac{da}{f(\Delta K(a))} & \text{for } a_0 < a_1 \\ \int_{a_0}^{a_c} \frac{da}{f(\Delta K(a))} & \text{otherwise} \end{cases}$$

(19)

where $a_1$ is the crack size at which the crack growth rate changes from medium to long crack growth. The transition between medium and long crack growth occurs when the crack size reaches a threshold stress intensity factor estimated by $\Delta K(a_1) = 9 \text{ MPa}\sqrt{\text{m}}$ [24].

The flowchart summarizing the entire procedure is depicted in Fig. 3. The corrosion initiation process depends on both environmental features and structural configuration. After corrosion initiation, the accumulation of corrosion products in the concrete/steel interface affects the corrosion rate as a result of excessive concrete cracking. The change in the corrosion rate depends upon environmental aggressiveness. By considering the interaction between pit growth and cyclic load, the rate competition criterion is evaluated, and consequently, the time of pit-to-crack transition is assessed. Finally, once the crack growth has become the predominant process, the crack growth induces the failure of the cross-section.

This model allows to evaluate the total structural lifetime including corrosion initiation, pit-to-crack transition and crack propagation. Its robustness lies in the implicit integration of various parameters and processes affecting the RC lifetime, which is very convenient for reliability analysis.

3. Probabilistic lifetime assessment and time-dependent reliability analysis

An efficient probabilistic lifetime assessment depends on the integration of the mechanical model presented in Section 2 into a suitable probabilistic framework. Thus, the cumulative distribution function (CDF) of the total corrosion-fatigue life, $F_{\tau_1}(\tau)$, is defined as [26]

$$F_{\tau_1}(\tau) = \text{Pr}(\tau_1 \leq \tau) = \int_{\tau_{pt} + \tau_{cg}}^{\tau} f(\mathbf{x}) \, d\mathbf{x}$$

(20)

where $\mathbf{x}$ is the vector of the random variables to be taken into account and $f(\mathbf{x})$ is the joint probability density function of $\mathbf{x}$. If structural failure is achieved when the crack or pit size reaches a critical value inducing the cross-section failure, the limit state function becomes

$$g(\mathbf{x}, \tau) = a_1(\mathbf{x}, \tau) - a_c(\mathbf{x}, \tau)$$

(21)

where $a_1(\mathbf{x}, \tau)$ is the crack or pit size at time $\tau$ and $a_c(\mathbf{x})$ is the critical crack or pit size corresponding to structural failure. For the above limit state function, there are two mechanisms leading to failure. In the former, the transition process from pit to crack described in

![Flowchart of the proposed model](image-url)
Section 2.2 takes place, and consequently, failure is generated by crack propagation. In the later, the transition does not occur because the pit growth rate is higher than the crack growth rate and corrosion is the failure mechanism. This case occurs when both the frequency and intensity of the load are low and/or when the structure is located in a highly aggressive environment. On the other hand, \( a_c(x) \) can be defined as (1) a deterministic value given by a fraction of the bar diameter \([24,26]\); or (2) a probabilistic value estimated in terms of ultimate limit states. In this work, \( a_c(x) \) is a random variable resulting from the evaluation of the limit state of bending, \( g_1(A(a), x) = M_d(A(a), x) - M_s(x) \) (22), where \( A(a) \) is the net steel area, \( x \) is the vector of random variables (i.e., applied load, concrete compressive strength, yield stress, etc.), \( M_d(A(a), x) \) is the bending moment capacity and \( M_s(x) \) is the applied moment. From Eq. (22), it is worthy to notice that the net steel area is a function of the crack or pit size \( a \). Thus, by defining \( A_0 \) as the initial cross-sectional area and \( \Delta A(a) \) as the cross-section loss by corrosion–fatigue action, \( A_0(a) \) is given by (see Fig. 4a)

\[
A_0(a) = A_0 - \Delta A(a)
\] (23)

Fig. 4a depicts the relationship between the bending moment and the crack or pit size. For the initial cross-sectional area of steel, the crack or pit size is equal to zero and the bending moment capacity is higher than the applied moment. When the crack or pit size grows until a critical size, \( a = a_c \), the bending moment capacity is equal to the applied moment, and consequently, failure is reached:

\[
M_d(A_0(a), x) = M_s(x)
\] (24)

where \( A_0(a) \) represents the critical cross-section. The bending moment capacity can be computed as

\[
M_d(A_0(a), x) = A_0(f'c)\left(d - 0.5\frac{A_0(f'c)}{f_yb}\right)
\] (25)

where \( f_y \) is the steel yield stress, \( d \) is the effective depth of the beam, \( f'c \) is the concrete compressive strength and \( b \) is the beam width. Thus, \( A_0(a) \) is estimated by substituting Eq. (25) in Eq. (24):

\[
A_0(a) = \frac{1}{f_y}\left(\frac{d}{b} - \frac{\sqrt{(d/f'c)^2 - 2f'yM_d(x)}}{bM_d(x)}\right)
\] (26)

Depending on a supposed shape for the crack or the pit, \( a_c \) is obtained from Eq. (26). However, given the complexity of the relationships to estimate \( A_0(a) \) [27], \( a_c \) is computed from an iterative procedure. On the other hand, it is possible to observe that for the limit state of bending, \( a_c(x) \) is not dependent on both the corrosion process and the frequency of the cyclic load. Nonetheless, the time to reach \( a_c(x) \), \( t_c \), depends on both environmental aggressiveness and the frequency of the cyclic load. By taking into account Eq. (21), the failure probability, \( p_f \), can be estimated as

\[
p_f = \int_{a_c}^{\infty} f(x) \, dx
\] (27)

Corrosion–fatigue deterioration process is a time-dependent problem where the interaction between diffusion, electrochemical, and LEFM principles is taken into account. Given the complexity of the problem, closed-form solutions for both the CDF of the total corrosion–fatigue lifetime and the failure probability are very difficult to obtain. Therefore, an appropriate tool to deal with this kind of problem is to use Monte Carlo simulations.

4. Application to bridge girder

4.1. RC girder and basic considerations

This section presents an example describing the coupled effect of corrosion–fatigue of a simply supported RC bridge girder subject to cyclic loading. The span of the girder is 10 m with the geometrical characteristics of the cross-section and the steel reinforcement given in Fig. 4b. The girder has been designed according to the EUROCODE 2 [28]. In addition to the dead load, a truck wheel load is applied on the girder. The design load, \( P_k \), corresponds to a wheel load located in the middle of the span. Table 1 presents the load and the material properties used in design and analysis.

<table>
<thead>
<tr>
<th>Variable</th>
<th>Value</th>
<th>Description</th>
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<tbody>
<tr>
<td>( P_k )</td>
<td>150 kN</td>
<td>Characteristic punctual design load</td>
</tr>
<tr>
<td>( E_{so} )</td>
<td>200,000 MPa</td>
<td>Elastic modulus of steel</td>
</tr>
<tr>
<td>( f'c )</td>
<td>30 MPa</td>
<td>Characteristic concrete compression strength</td>
</tr>
<tr>
<td>( f'c )</td>
<td>500 MPa</td>
<td>Characteristic steel strength</td>
</tr>
<tr>
<td>( c_{cr} )</td>
<td>22 kN/m^3</td>
<td>Specific weight of concrete</td>
</tr>
<tr>
<td>( \rho_{sl} )</td>
<td>18 kN/m^3</td>
<td>Specific weight of pavement</td>
</tr>
<tr>
<td>( \rho_{ct} )</td>
<td>0.2</td>
<td>Concrete Poisson ratio</td>
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<tr>
<td>( \rho_{sc} )</td>
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</tr>
<tr>
<td>ac</td>
<td>5.14</td>
<td>Aggregate-to-cement ratio</td>
</tr>
</tbody>
</table>

Table 1 Design load and material constants

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**Fig. 4.** (a) Schematic description of the computation of \( a_c \) and (b) configuration of the bridge girder.
The effect of environmental aggressiveness on structural reliability is considered by accounting for four levels of aggressiveness (Table 2). Each level is characterized by (1) a chloride surface concentration, \( C_s \); (2) an expected threshold corrosion rate, \( \lambda_{th} \); and (3) a concrete cover, \( c \). The chloride surface concentration is treated as a random variable for which the mean largely depends on the proximity to sea and is computed based on the work of McGee [29]. The expected threshold values of the corrosion rate are based on the EN206 [30], and the cover is defined according to EUROCODE 2 [28].

The basic considerations and assumptions in this study are:

- Cross-section confinement is taken into account by using the Kent and Park model [31] and stress–strain relationship for steel follows an elasto-plastic model.
- The elasticity modulus and the tensile strength of concrete are given in terms of the compressive strength according to EUROCODE 2 [28].
- A limit crack width of 0.5 mm is used as a threshold for severe concrete cracking, \( \tau_{cr} \).
- The water/cement ratio is calculated by using the Bolomey's formula [18].
- A threshold pit depth, \( p_{th} \), of \( 1.98 \times 10^{-6} \) m is utilized to estimate the time to pit nucleation.
- The deterioration process is continuous; i.e., there is no maintenance.
- The limit state of bending capacity is considered to compute \( \sigma_c \).
- The reliability analysis was carried out using Monte Carlo simulations.

The probabilistic models for the variables are given in Table 3. It is worthy to notice the high variability of surface chloride concentration. For \( C_s \), data reported by McGee [29] were obtained from a field-based study of 1158 bridges in the Australian state of Tasmania. This work appears to be the most comprehensive study for bridges in different environmental conditions. The chloride diffusion coefficient is influenced by many factors such as mix proportions (i.e., water/cement ratio, cement type), curing, compaction and environment (i.e., relative humidity and temperature), among others. Nonetheless, experimental studies report an important correlation between \( D_{sl} \) and \( w/c \) [32,33]. Papadakis et al. [32] propose the following equation:

\[
D_{sl}(w/c) = 0.15D_{sl,0} \frac{1 + \rho_c w/c}{1 + \rho_c w/c + \rho_a c/a/c} \left( \frac{\rho_c w/c - 0.85}{1 + \rho_c w/c} \right)^3
\]

where \( D_{sl,0} \) is the chloride diffusion coefficient in an infinite solution, (i.e., \( D_{sl,0} = 50491.08 \) mm²/year for NaCl), \( \rho_c \) and \( \rho_a \) are the densities of cement and aggregates, respectively. In order to compute the probabilistic model of \( D_{sl} \) from Eq. (28), Monte Carlo simulations and a Kolmogorov–Smirnov test (KS-test) with a level of significance of 5% are carried out. In this analysis it is also considered (1) the values of \( \sigma_c \), \( \rho_c \), and \( \rho_a \) presented in Table 1; (2) the correlation between \( w/c \) and the compressive strength of concrete (i.e., Bolomey’s formula) and (3) the probabilistic model of \( \sigma_c \) given in Table 3. The results of the KS-test indicate that \( D_{sl} \) follows a lognormal distribution with mean equal to 41.96 mm²/year and a COV of 67%. Vu and Stewart [18] also reported other experimental data with similar values compared to the simulation results. On the contrary, Thoif-Christensen [33] considered that \( D_{sl} \) depends principally on the water/cement ratio and the temperature, \( T \):

\[
D_{sl}(w/c,T) = 11.146 - 31.025w/c - 1.941T + 38.212w/c^2 + 4.48w/cT + 0.0247^2
\]

In this case, by assuming that \( T \) is normally distributed \( \mathcal{N}(15, 3^2) \), the KS-test indicates that \( D_{sl} \) follows a lognormal distribution with mean equal to 16.25 mm²/year and a COV of 35%. The results obtained by considering this approach are significantly smaller than in the first case. As a matter of fact, Eq. (29) results from the fitting of experimental data, and does not reflect the real scatter of the diffusion process for real structures. Therefore, by considering that the data obtained with the Papadakis’s formula are consistent with other empirical results, Eq. (28) is adopted to take into consideration the correlation between \( D_{sl} \) and \( w/c \).

The fatigue analysis is performed under the cyclic loading simulated by a random wheel load, \( P \), with a daily traffic frequency, \( f \), applied at the center of the span. It is assumed that \( P \) follows a lognormal distribution (Table 3). To simplify the analysis, the traffic frequency and the statistical parameters of \( P \) are considered as time-invariant and the effect of higher traffic frequencies on the applied load is not taken into account. To study the implications of \( f \), the values of 50, 500, 1000 and 2000 cycles/day are considered. It is important to stress that these frequencies are in the range defined by the EUROCODE 1 for heavy trucks [28].

The stress range induced by the cyclic load, \( \Delta \sigma \), is computed by evaluating the equilibrium conditions between internal and external forces. Tension at bottom reinforcement is defined as critical since this reinforcement is subject to both pitting corrosion and cyclic tension forces. The tension force due to cyclic loading varies between \( T_{min} \) corresponding to dead load, and \( T_{max} \) resulting from

---

Table 2: Description of the studied environments

<table>
<thead>
<tr>
<th>Level of aggressiveness</th>
<th>Description</th>
<th>( C_s ) [29], kg/m³</th>
<th>( \lambda_{th} ) [30], µA/cm²</th>
<th>( c ) [28], mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low</td>
<td>Structures placed at 2.84 km or more from the coast; sea spray carried by the wind is the main source of chlorides</td>
<td>0.35</td>
<td>0.5</td>
<td>40</td>
</tr>
<tr>
<td>Moderate</td>
<td>Structures located between 0.1 and 2.84 km from the coast without direct contact with seawater</td>
<td>1.15</td>
<td>2</td>
<td>45</td>
</tr>
<tr>
<td>High</td>
<td>Structures situated to 0.1 km or less from the coast, but without direct contact to seawater. RC structures subject to de-icing salts can also be classified in this level</td>
<td>2.95</td>
<td>5</td>
<td>50</td>
</tr>
<tr>
<td>Extreme</td>
<td>Structures subject to wetting and drying cycles; the processes of surface chloride accumulation are wetting with seawater, evaporation and salt crystallization</td>
<td>7.35</td>
<td>10</td>
<td>55</td>
</tr>
</tbody>
</table>

---

Table 3: Probabilistic models of the variables used in the example

<table>
<thead>
<tr>
<th>Variable</th>
<th>Distribution</th>
<th>Mean</th>
<th>COV</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>( P )</td>
<td>Lognormal</td>
<td>115 kN</td>
<td>0.20</td>
<td></td>
</tr>
<tr>
<td>( f_c' )</td>
<td>Normal</td>
<td>40 MPa</td>
<td>0.15</td>
<td>[34]</td>
</tr>
<tr>
<td>( f_y )</td>
<td>Normal</td>
<td>600 MPa</td>
<td>0.10</td>
<td>[34]</td>
</tr>
<tr>
<td>( C_n )</td>
<td>Uniform</td>
<td>0.90 kg/m³</td>
<td>0.19</td>
<td>[18]</td>
</tr>
<tr>
<td>( C_s )</td>
<td>Low</td>
<td>Lognormal</td>
<td>0.35 kg/m³</td>
<td>0.50</td>
</tr>
<tr>
<td></td>
<td>Moderate</td>
<td>Lognormal</td>
<td>1.15 kg/m³</td>
<td>0.50</td>
</tr>
<tr>
<td></td>
<td>High</td>
<td>2.95 kg/m³</td>
<td>0.50</td>
<td>[29]</td>
</tr>
<tr>
<td></td>
<td>Extreme</td>
<td>7.35 kg/m³</td>
<td>0.70</td>
<td>[29]</td>
</tr>
<tr>
<td>( \sigma_{min} )</td>
<td>Normal</td>
<td>3600 kg/m³</td>
<td>0.10</td>
<td>[35]</td>
</tr>
<tr>
<td>( \sigma_{max} )</td>
<td>Lognormal</td>
<td>12.5 × 10⁻⁶ m</td>
<td>0.20</td>
<td>[35]</td>
</tr>
<tr>
<td>( \tau_{cr} )</td>
<td>Gumbel</td>
<td>5.65</td>
<td>0.22</td>
<td>[27]</td>
</tr>
</tbody>
</table>
the combined action of dead and live loads. Based on these considerations, the stress range, $\Delta \sigma$, is computed as

$$
\Delta \sigma(P) = \frac{T_{\text{max}}(P) - T_{\text{min}}(P)}{A_i}
$$

(30)

where $A_i$ represents the cross-sectional area of the reinforcing in tension (i.e., at the bottom). On the other hand, this analysis is also carried out without considering cumulated fatigue damage.

4.2. Deterioration stages and total lifetime

The first part of this subsection is devoted to present the results concerning the time-span of each life stage. The main goal of this analysis is to understand the influence of the most important variables on the length of each stage. This analysis is carried out by fitting the results obtained from Monte Carlo simulations for each stage by an appropriate PDF. The goodness-of-fit test used as a selection criterion is the KS-test with a level of significance of 5%. In the last part of this subsection, the participation of each stage in the total corrosion-fatigue lifetime is assessed. The results presented in this subsection concern

- corrosion initiation and pit nucleation,
- pit-to-crack transition,
- crack growth,
- total corrosion-fatigue lifetime, and
- contribution of each stage to the total lifetime.

4.2.1. Corrosion initiation and pit nucleation

For all levels of aggressiveness, the goodness-of-fit test indicates that both $t_{\text{ini}}$ and $t_{\text{pt}}$ follow lognormal distributions. As expected, the mean, $\mu_{t_{\text{ini}}}$, and standard deviation, $\sigma_{t_{\text{ini}}}$, decrease when the environmental aggressiveness is increased (Fig. 5). The high standard deviations result from the high COVs of $C_s$. The impact of the high variability of $C_s$ on the PDF of $t_{\text{ini}}$ is studied by varying its COV, as shown in Fig. 6. For all the considered COVs, $\mu_{t_{\text{ini}}}$ diminishes when $\mu_{C_s}$ increases, and tends to larger values when the COV of $C_s$ becomes high. It is also observed that the relationship between $\mu_{t_{\text{ini}}}$ and $\mu_{C_s}$ is not linear and there is a limit (e.g., $\mu_{C_s} = 2 \text{ kg/m}^3$) from which an increase of $\mu_{C_s}$ does not produce significant variation of $\mu_{t_{\text{ini}}}$.

In addition, it should be noted that the COV($C_s$) has more appreciable impact on $\mu_{t_{\text{ini}}}$ for smaller values of $\mu_{C_s}$ (Fig. 6).

For the time to pit nucleation, the mean of $t_{\text{pt}}$ varies between 3.9 and 5.4 days with a COV($t_{\text{pt}}$) of 0.25 for all levels of aggressiveness. Therefore, given both the higher magnitude and variability of the time to pit nucleation, it can be concluded that the time to pit nucleation is negligible.

4.2.2. Pit-to-crack transition

For all levels of aggressiveness and traffic frequencies, the KS-test has shown that the time to transition from pit to crack follows a lognormal distribution. Fig. 7 presents the mean and 90% confidence interval of $t_{\text{pt}}$ for several traffic frequencies. The ranges of [8.6, 52.1] years and [3.1, 41.8] years are obtained for $\mu_{t_{\text{pt}}}$ and $\sigma_{t_{\text{pt}}}$, respectively. In all cases, the mean and confidence interval for $t_{\text{pt}}$ decrease for higher values of traffic frequency. This decrease is expected because the crack growth rate is a function of $f$, and therefore, the intersection between $dP/dt$ and $dP/dt_{\text{cr}}$ occurs earlier when $f$ is increased. It can also be observed that the mean and the dispersion of $t_{\text{pt}}$ are smaller when environmental aggressiveness increases.

This reduction is explained by the fact that high levels of environmental aggressiveness are related to high values of $C_s$. Thus, when $C_s$ goes to high values, the fatigue crack growth rate is increased producing early crack nucleation.

Since a crack is nucleated at the end of this stage, and the initial crack size becomes an input data to compute the length of the following stage (i.e., Eq. (19)), Fig. 8 depicts the PDFs of $a_0$ for various levels of aggressiveness with several traffic frequencies. In all cases, the goodness-of-fit test shows that $a_0$ follows a lognormal distribution with $\mu_{a_0}$ varying between 1.5 and 8.9 mm and $\sigma_{a_0}$ between 0.95 and 2.63 mm. The overall behavior indicates that both
the mean and standard deviation tend to higher values when the level of aggressiveness is increased (Fig. 8a). This increase is expected, because given the predominance of pitting corrosion on fatigue for higher levels of aggressiveness, the transition occurs for higher initial crack sizes. From the relationship between the mean and standard deviation of \(a_0\) and the traffic frequency, it is possible to notice that both \(a_0\) and \(\sigma_{a_0}\) decrease when \(f\) leads to higher values (Fig. 8b). This reduction is due to the fact that the increase of traffic frequency turns the fatigue into the predominant process, and subsequently, accelerates the pit-to-crack transition.

4.2.3. Crack growth

In all the considered cases, the KS-test confirms that \(\tau_{cg}\) follows a lognormal distribution with \(\mu_{\tau_{cg}}\) varying between 1.6 and 47.9 years and \(\sigma_{\tau_{cg}}\) between 0.3 and 11.1 years. Fig. 9 depicts the relationship between the mean, the 90% confidence interval of \(\tau_{cg}\) and the traffic frequencies. The overall behavior indicates that the mean and the dispersion of \(\tau_{cg}\) diminish when the traffic frequency increases. This behavior is explained by the fact that failure, i.e., Eq. (21), is reached in a shorter period when the frequency tends to higher values (i.e., Eq. (19)). Furthermore, it

![Fig. 7. Mean and 90% confidence interval of \(\tau_{pt}\) for several traffic frequencies.](image)

![Fig. 8. PDFs of \(a_0\) for (a) various levels of aggressiveness and \(f = 50\) cycles/day; (b) low level of aggressiveness and various traffic frequencies.](image)

![Fig. 9. Mean and 90% confidence interval of \(\tau_{cg}\) for several traffic frequencies.](image)
can be noted that the mean and the dispersion decrease for higher levels of aggressiveness. This decrement is related to the fact that the size of the initial crack leads to larger values for higher levels of environmental aggressiveness, and therefore, the time to reach failure is lower.

For all traffic frequencies and levels of environmental aggressiveness, the KS-test shows that the size of the critical crack or pit at which the RC member reaches the bending limit state, \(a_c\), follows a Weibull distribution with mean and standard deviation of 11.86 and 0.97 mm, respectively. This non-dependency on \(f\) and environmental aggressiveness is expected because the bending limit state depends mainly on the critical cross-section of steel bars leading to bending failure, \(A_s(a_c)\), as well as the load intensity. In addition, it is observed that the mean of \(a_c\) is close to the deterministic value reported by Salah el Din and Lovegrove, i.e., \(a_c = 0.5d_0 = 12.5\) mm [24].

4.2.4. Total corrosion–fatigue lifetime

The goodness-of-fit test demonstrates that \(s_T\) follows a lognormal distribution with mean varying between 35.8 and 448.8 years and standard deviation between 49 and 264.7 years. The PDFs of \(s_T\) for various levels of aggressiveness and \(f = 500\) cycles/day are depicted in Fig. 10a. As expected, \(\mu_{s_T}\) and \(\sigma_{s_T}\) decrease for higher levels of aggressiveness. Fig. 10b presents the mean of \(s_T\) for various levels of aggressiveness and traffic frequencies. In general, the critical time, \(s_T\), occurs earlier when the traffic frequency is augmented. Overall behavior of \(s_T\) agrees with the results found for each stage.

4.2.5. Contribution of each stage to the total lifetime

The percentage of participation of each stage in the total corrosion–fatigue life is estimated by the ratios of the time-spans (Fig. 11). In all cases, the participation is mainly due to \(\tau_{cp}\) followed by \(\tau_{pt}\) and \(\tau_{cg}\). Whereas for \(\tau_{cp}\), the participation decreases when the level of aggressiveness is increased, for \(\tau_{pt}\) and \(\tau_{cg}\) it grows when the level of environmental aggressiveness becomes higher. This behavior seems logical because an increment in the level of environmental aggressiveness accelerates the time to corrosion initiation and, subsequently, the participation of \(\tau_{pt}\) and \(\tau_{cg}\) takes larger values. By considering the relationship between \(\tau_{cp}\) and \(f\), the percentage of participation of \(\tau_{cp}\) increases when the traffic frequency grows. On the other hand, for \(\tau_{pt}\) and \(\tau_{cg}\), their percentages decrease when \(f\) tends to higher values. This behavior is explained by the fact that larger traffic frequencies reduce the length of the crack nucleation and propagation stages. To conclude, given the larger impact of the time to corrosion initiation and pit nucleation on the total corrosion–fatigue life (i.e., 60–93% of \(s_T\)), the maintenance efforts must be directed toward controlling this stage of the process.

Fig. 10. (a) PDFs of \(s_T\) for various levels of aggressiveness and \(f = 500\) cycles/day; (b) mean of \(s_T\) for both, various levels of aggressiveness and traffic frequencies.

Fig. 11. Participation of each phase in the total corrosion–fatigue life: (a) low aggressiveness and (b) extreme aggressiveness.
4.3. Effects of corrosion–fatigue on the total lifetime

This subsection presents the results concerning the coupled effect of corrosion–fatigue on lifetime. Firstly, the expected total lifetimes are compared for the cases: (1) deterioration induced by corrosion (i.e., without fatigue) and (2) deterioration produced by the coupled action of corrosion–fatigue. This comparison is made to estimate the additional lifetime reduction induced by considering the coupled effect. Finally, the results of the time-dependent reliability analysis including the case without fatigue, and the analysis of the influence of \( A_s \) on the failure probability are shown.

4.3.1. Additional lifetime reduction induced by corrosion–fatigue

The relationship between the mean of \( \tau_T \) and the level of aggressiveness is plotted in Fig. 12a. It should be noted that the mean of \( \tau_T \) for the case without considering fatigue damage (i.e., \( f = 0 \) cycles/day), is also included. The increase of traffic frequency induces an appreciable reduction of \( \mu_T \), in particular, in environments with low aggressiveness. By taking as a reference the case without considering fatigue damage, it is possible to estimate the additional lifetime reduction induced by the coupled action of corrosion and fatigue (Fig. 12b). The additional reduction of \( \tau_T \) is high for smaller levels of aggressiveness and bigger traffic frequencies. It can also be noticed that considering the coupled effect of corrosion and fatigue can represent an additional lifetime reduction from 3% to 39% with respect to the case without fatigue. The coupled effect of corrosion and fatigue does not produce an appreciable additional lifetime reduction for both smaller traffic frequencies and very aggressive environments. However, for traffic frequencies between 500 and 2000 cycles/day, this coupled effect induces additional lifetime reductions between 18% and 39%. These results strengthen the importance of including the coupled effect of corrosion and fatigue in the lifetime assessment, in particular, when the structure is placed in corrosive environments under cyclic loading.

4.3.2. Structural reliability of the girder

The failure probability of the girder, \( p_f \) is plotted in Fig. 13. The overall behavior indicates that \( p_f \) tends to higher values when both, the level of aggressiveness and the traffic frequency are increased. It must be noted that Fig. 13 also includes the reliability analysis of the case without fatigue. As expected, reliabilities obtained by considering the coupled action of corrosion and fatigue are higher than for the case without fatigue. It can also be observed that although the difference between the two cases is much larger for high traffic frequencies, for values between 500 and 2000 cycles/day, this difference remains almost constant. For instance, for a time-span of 100 years, a moderate level of aggressiveness, without considering fatigue damage, a failure probability of \( 1.1 \times 10^{-2} \) is obtained; for the same conditions but with different frequencies, failure probability becomes \( 9.3 \times 10^{-2} \), \( 2.2 \times 10^{-1} \), \( 2.4 \times 10^{-1} \) and \( 2.5 \times 10^{-1} \) for traffic frequencies of 50, 500, 1000 and 2000 cycles/day, respectively. That means, the failure probability is increased by 8, 20, 21.8 and 22.7 times for the traffic frequencies of 50, 500, 1000 and 2000 cycles/day, respectively. From these results, it is possible to affirm

![Fig. 12. (a) Mean of \( \tau_T \) for various traffic frequencies and (b) additional lifetime reduction induced by corrosion–fatigue.](image)

![Fig. 13. Time-dependent structural reliability of the considered bridge girder.](image)
that, although the coupled effect of corrosion and fatigue causes a significant increase of $p_f$, the increase of $f$ does not produce an appreciable change in $p_f$ above a certain threshold.

The influence of the reinforcement area, $A_t$, on failure probability for a traffic frequency of 1000 cycles/day is plotted in Fig. 14. By supposing that the design specifications must guarantee a given target failure probability, $p_t$, i.e., $p_t = 10^{-5}$, during a lifetime of 50 years, it can be observed from Fig. 14 that the reinforcement resulting from the design (i.e., 8 bars $d_0 = 25$ mm) does not assure $p_t$. For the case of low aggressiveness (Fig. 14a), increasing the number of bars of the same diameter (i.e., 14 bars $d_0 = 25$ mm) can guarantee this condition. Nevertheless, although increasing $A_t$ tends to amplify the time to reach $p_t$, for the other levels, the failure probability is lower than $p_t$. Therefore, to ensure $p_t$ during the lifetime of the structure, it is necessary to implement other countermeasures such as inspection and maintenance programs for corrosion control.

5. Conclusions

The combined action of corrosion and fatigue strongly influences the performance of RC structures and reduces substantially their lifetime. In this paper, a model that couples these two phenomena is developed and the consequences on the life-cycle of RC structures are assessed. The model takes into account the interaction between the following processes: (1) corrosion induced by chloride ingress, (2) concrete cracking resulting from the accumulation of corrosion products and (3) reinforcement fatigue produced by cyclic loading. The corrosion–fatigue deterioration process is divided into three stages: (1) corrosion initiation and pit nucleation, (2) pit-to-crack transition and (3) crack growth. Corrosion initiation is induced by chloride ingress causing the nucleation of a pit. Pit-to-crack transition is achieved when the crack growth produced by cyclic loading becomes the predominant process; a crack is nucleated at the end of this stage. Finally, crack growth ends when the crack size leads to structural failure.

In order to illustrate the model, a bridge girder located in various chloride-contaminated environments was studied. The time-dependent reliability analysis of the girder included the random nature of the material parameters, the loading scheme and the environmental conditions; the solution is found by Monte Carlo simulations. The analysis shows that the failure probability depends highly on the threshold corrosion rates, the surface chloride concentration and the traffic frequency.

The results also show that the time to corrosion initiation is highly influenced by the coefficient of variation of the chloride surface concentration. The expected time to pit nucleation has a smaller value in comparison with the expected time to corrosion initiation; therefore, it is possible to affirm that the length of this sub-stage can be neglected. It was also observed that pit-to-crack transition and crack growth occur early when both the level of aggressiveness and traffic frequency are increased. Regarding external loading, it was remarked that for traffic frequencies between 500 and 2000 cycles/day, the coupled effect of corrosion–fatigue becomes paramount by leading to additional lifetime reductions between 18% and 39%. However, there is a limit after which the increase of traffic frequency does not produce appreciable changes in the failure probability.

Finally, it was observed that if the target failure probability is set to $10^{-4}$, for a structure with operating lifetime of 50 years, the structural configuration selected does not reach this value for all the considered environmental conditions. Therefore, to guarantee the operation above a predefined target failure probability, it is necessary to implement countermeasures such as inspection and maintenance programs for corrosion control.

References