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Stochastic improvement of inspection and maintenance of corroding reinforced concrete structures placed in unsaturated environments

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Abstract

Multiple constraints imposed by economic, social and environmental considerations lead maintenance planning optimization to a major challenge for designers, owners and users of infrastructure. This study focuses on the management of reinforced concrete (RC) structures placed in chloride-contaminated atmospheres. Under these exposure conditions, chloride penetration generates corrosion of the reinforcing bars reducing the RC durability. Therefore, modeling the deterioration process as well as the maintenance actions carried out during the operational life becomes paramount for the formulation of a comprehensive maintenance strategy. This paper presents a new methodology for improving the performance of maintenance strategies for corroding RC structures. The proposed approach is based on preventive maintenance and combines: a numerical model of chloride penetration, Markov processes and decision theory. The uncertainty related to material properties, model and environmental actions as well as the effect of imperfect inspections are also integrated in the model. The aforementioned points are illustrated with a numerical example. In general, the overall results indicate that the proposed methodology can be useful to improve the costs of a given maintenance strategy by ensuring appropriate levels of serviceability and safety.

Keywords:

Chloride ingress, corrosion, reinforced concrete, reliability, inspection, maintenance, unsaturated environments

1. Introduction

Nowadays, design and management of infrastructure must consider economic, social and environmental constraints to reduce environmental impact, to optimize resource management and to decrease waste generation. This new trend in design and management must also consider all the phenomena that affect the structural performance. For reinforced concrete (RC) structures, corrosion induced by chloride ingress generates important damage after 10 or 20 years of service [1, 2]. The management of corroding RC structures should therefore integrate the stages of inspection and maintenance to ensure an optimal level of safety during its operational life. However, the complexity and the uncertainties related to the deterioration process and the repair techniques transform management into a challenge for owners/operators where decisions are

frequently taken without being aware of the consequences of the actions. Many deteriorated structures are being currently evaluated for possible repair and continued service because they are situated where their replacement would be economically unfeasible. For instance, Bhide [3] reports that about 173,000 bridges in the United States are structurally deficient or functionally obsolete due in part to corrosion. Thus, developing robust models for prediction and strategies for periodic inspection and maintenance plays a significant role in enabling target reliabilities to be met over a period of continued service [4].

Management of deteriorating structures should also include the variability and uncertainties present in both deterioration process and maintenance actions. The main sources of uncertainty are related to properties of construction and repair materials, environmental actions (climate conditions, chloride concentrations), loading and effectiveness of inspection and repair actions. The following approaches can be used to model deterioration and maintenance actions by taking into consideration the aforementioned aspects [5]: (i) failure rate functions, (ii) Markov models, (iii) stochastic processes, and (iv) time-dependent reliability index. Maintenance based on failure rate function is useful in electrical or mechanical engineering where the states of "functioning" and "failure" are well-defined. However, the concept of structural failure in civil engineering can be related to different structural conditions (i.e., corrosion initiation, loss of serviceability, etc.). Markov-based management systems are widely used in management of civil engineering systems [6, 7, 8]. However, since the most part of these systems are based on visual inspection, the major limitation of this approach is that reliability is not directly incorporated in structural management [9]. Time-dependent deterioration processes and maintenance actions can also be modeled by stochastic processes. In particular, the gamma process has been widely used to represent several deterioration processes such as: permanent erosion of dunes, long-shore rock transport near berm breakwaters, loss of steel thickness due to corrosion, etc. [5]. However, this model only focuses on one component, one failure mode and one uncertainty [10] and the identification of its parameters for common deterioration processes is still a challenge [11]. Reliability-based management systems minimize costs by ensuring an optimal level of safety. One major advantage of this approach lies in the direct incorporation of the uncertainties and in the consideration of various limit states and failure modes. However, the main disadvantage of this approach is that the effects of maintenance are difficult to estimate [5].

Each management approach has its advantages and shortcomings. The selection of a given approach should take into account other aspects such as: context (civil, mechanical or electrical engineering problems), size of studied problem (network or single structures), and other socio-economic aspects (priorities of the agency, country). The main purpose of this paper is to develop a methodology to improve the management of RC structures subjected to chloride penetration. Improved management should minimize the costs of inspection, repair, corrosion initiation and failure ensuring appropriate levels of serviceability and safety. This study uses a Markovian approach for modeling deterioration and maintenance actions. Contrary to the Markov-based management systems available in the literature, inspections are based on a measurable pa-

rameter (chloride concentration) and consider the uncertainty of the measurements (imperfect inspections). Consequently, inspection results affect the assessment of the probability of corrosion initiation or failure used in the management system. Within this context, the specific objectives of this paper are:

1. to define a comprehensive strategy that could be used to maintain corroding RC structures;
2. to integrate a probabilistic model of chloride penetration for modeling this phenomenon as a Markov chain;
3. to develop a model of maintenance that includes various practical problems such as errors in the inspection measurements and uncertainties; and
4. to study the effect of the model parameters on the total costs and to illustrate its use for decision-making.

The description of the maintenance strategy is presented in section 2. Section 3 details the assessment of the transition matrices used for modeling chloride penetration. The maintenance and cost models are described in sections 4 and 5. Finally, section 6 presents a numerical example illustrating the proposed methodology.

2. Description of the strategy of maintenance

Maintenance strategies should ensure optimal levels of serviceability and safety during the structural lifetime minimizing both costs and environmental impact. Therefore, when structural performance is strongly affected by the kinematics of the deterioration process, repair and/or replacement of structural components are the only way to maintain appropriate serviceability and safety levels. This study supposes that repair actions are based on inspection results. Then, the strategy of maintenance is divided into two stages: *inspection* and *repair*. Inspection results alert the owner/operator when a given deterioration threshold is reached. Repair actions should ensure an appropriate structural condition.

Inspection can be carried out by employing nondestructive and/or destructive methods. Visual inspection is a nondestructive technique usually used for evaluating the condition of deteriorating RC structures based on the appearance of the structure [8]. However, the assessment of the structural condition remains largely uncertain for this technique. Nowadays, there have been significant advances in the development of inspection methods based on nondestructive monitoring. Nondestructive techniques aim at quantifying the instant value or the evolution in the time of a given variable (material strength, chloride concentrations, corrosion rate, etc.) from measurements of a physical parameter (frequency, capacity, etc.). Nevertheless, their results are highly influenced by environmental conditions reducing its accuracy [12, 13]. The accuracy of inspection results is largely improved by using destructive inspection techniques. However, they are expensive, time-consuming and require a larger number of tests when there is a larger variability of the inspected parameter. The selection of one or a combination of inspection/repair techniques depends on several aspects such as:

- the type of inspected phenomenon –e.g., material strength, chloride concentration, etc.;
- size of the project –e.g., structural network or particular structure;
- accessibility to the inspection/repair area –e.g., immersed, splash and tidal or atmospheric zone;
- use of the structure –e.g., nuclear, transportation, etc. This point is related to the risk allowed by the agency; and
- other socio-economic aspects such as priorities of the country, availability of resources, etc.

This study refers to RC structures subjected to chloride ingress and is focused on the maintenance of structural components of a single structure placed in a splash and tidal zone. The requirements for the selection of a maintenance strategy were defined within the framework of the project FUI (2007-2010) MAREO[1] [14] with the collaboration of agencies, construction companies and research centers. The corrosion damage is important in this zone because there is a higher availability of chlorides and oxygen in the environment and chloride ingress is accelerated by wetting/drying cycles. Moreover, this area is the most critical for the effectiveness of inspection and repair due to varying conditions with the tide level. Then, stakeholders manifest that inspection and repair in this zone become an important issue for infrastructure management.

The selection of the strategy of maintenance took into account both the characteristics of the targeted repair zone and previous experience of stakeholders with other repair techniques –i.e., cathodic protection, use of inhibitors, patch repair, etc. In the strategy of maintenance considered herein, inspection is undertaken by analyzing the concentration of chlorides at the cover depth on concrete cores (semi-destructive method). Afterwards, depending on inspection results, the repair technique consists in rebuilding the polluted concrete cover by several methods. The advantage of the proposed approach is that repair is more preventive than corrective. This characteristic ensures an optimal level of safety during the lifespan of the project.

2.1. Definition of the repair criterion

The owners/operators define the schedule of repair actions on the basis of a given criterion related to an allowable damage threshold. The selection of the repair criterion is an important topic in maintenance because it should consider several aspects that differ for particular problems –i.e., economic, environmental, practical, etc. This study establishes a repair criterion where the structure is repaired before corrosion initiation. This criterion can be considered conservative in comparison to other criteria found in the literature where repair is carried out after initial or several concrete cracking occurs [15]. Thus, the consequences of failure (corrosion initiation) remain moderate in terms of economy, safety and functionality.

[1] Maintenance and REpair of concrete coastal structures: risk-based Optimization (MAREO) Project

The criterion proposed herein has been defined after discussion with the stakeholders participating in the MAREO project taking into account the following reasons:

- *Reduction of the structural safety decay.* When the repair criterion is based on the structural condition after corrosion initiation (e.g., concrete cracking), structural safety is affected by the loss of reinforcing steel. Therefore, the assessment of the next repair time should consider this loss and/or the replacement of the corroded reinforcing bars at a given time. These considerations make the repair scheme complex because the repair times are time-dependent and the strengthening actions should be included in the analysis. Since for the selected criterion repair is carried out before corrosion initiation, it is possible to assume that after each repair the RC member is "as good as new". Under this assumption, the repair intervals are constant.

- *Reduction of the replacement of reinforcing bars.* From a practical point of view, the building companies manifest that the replacement of reinforcing bars in existing structures is complicated. Therefore, another advantage of the proposed criterion is that, since repair takes place before corrosion initiation, the replacement of corroded bars is reduced. However, the condition of reinforcement should be checked before cover rebuilding.

- *Inspection based on a quantifiable parameter.* This criterion is convenient to combine the maintenance strategy with inspections because it is based on a measurable variable (chloride concentration at the reinforcement depth). Furthermore, the owner/operator can use the inspection results to update the condition of the structure before repair calibrating the maintenance schedule.

This repair criterion is related to the probability of corrosion initiation. Then, the determination of an appropriate threshold chloride concentration for corrosion initiation, C_{th}, becomes a major challenge for the owner/operator. It is not feasible to define a representative value for this parameter because there is a wide range of threshold chloride concentrations for corrosion initiation suggested in the literature. This larger variability can be attributed to the number of parameters influencing C_{th}: type and content of cement, exposure conditions, time and type of exposure, distance to the sea, oxygen availability at the bar depth, type of steel, electrical potential of the bar surface, presence of air voids, etc. In addition, the following aspects add more variability to the problem [16, 17]:

- *Definition of corrosion initiation:* As discussed in Duracrete project [18], corrosion initiation could be defined as the depassivation of the reinforcing steel (a shift in the potential or rate of corrosion) or associated with "visible" or "acceptable" deterioration of the RC structure. The adopted scheme of inspection/maintenance considers depassivation as the definition of corrosion initiation. According to [19], depassivation occurs when, in a surface of steel of 1 cm^2, the corrosion rate is higher than 0.1 μA.

- *Technique used to determine active corrosion:* The method used to determine depassivation also influences the measurement of C_{th}. A certain decay of corrosion potential E_{corr} or an increase of corrosion rate i_{corr} can be used as indicators of corrosion initiation. However, taking into account that there is no direct relationship between E_{corr} and i_{corr} [20], the assessment of C_{th} for each technique would lead to different values. Consequently, an increase in corrosion rate is considered herein to define C_{th}.

- *Testing methods for chloride threshold determination:* Alonso and Sanchez [16] classified these testing methods into two categories:

  - Accelerated methods: these methods are carried out in laboratory and take into account any external action that could affect the electrochemical conditions of the surface of the reinforcement from its natural state accelerating chloride penetration and/or corrosion initiation.
  - Natural methods: this category considers on-site tests or tests under natural exposure conditions. Therefore, the electrochemical changes occurring at the surface of the reinforcement during depassivation are not affected by any electrical action.

  As expected, threshold chloride concentrations for accelerated methods are lower than those for natural methods. Depending on the technique for the accelerated methods (potentiostatic and migration), the mean value of the total threshold chloride concentration measured by natural methods can be reduced from 25 to 92% [16]. Accordingly, the threshold chloride concentrations measured by natural methods are considered herein because they are more representative of real exposure conditions.

Table 1 presents the values of C_{th} related to depassivation and measured taking into account an increase of the corrosion rate for natural methods [16]. To compute C_{th} in kg/m^3, it is assumed that the concrete contains 400 kg of cement per m^3 because the real cement content is unknown. This cement content corresponds to repair materials (mortar or concrete) specially used when repair is carried out in the splash and tidal zone. However, the real cement content should be used for a practical application. Table 1 shows that for a given environmental exposure, a lower value of the mean of C_{th} implies that the quantity of chlorides required to initiate the corrosion reaction is small. It is also noted that the mean and the standard deviation of C_{th} are highly dependent on concrete cracking and exposure conditions (immersed or in air). The worst condition corresponds to cracked structures exposed to the air above the splash zone. In this case, since the availability of oxygen and water at the corrosion cell is larger, the quantity of chlorides required for corrosion initiation decreases. It is also important to note that the larger variability for "in field" case is explained by the fact that this category groups measurements for several environmental conditions.

3. Modeling chloride ingress

Taking into account that there are many factors that influence the deterioration process and the effects of maintenance on RC structures, numerical models could be used to represent deterioration and maintenance, and therefore, to improve decision-making. Many management systems use Markov chains to simulate deterioration and repair of structures over the time [8, 21, 22]. This study also uses a Markovian approach that integrates the following aspects:

- a comprehensive model of chloride penetration;
- the accuracy of the inspection technique; and
- the uncertainty inherent to chloride penetration, corrosion initiation and inspection results.

A discrete-time Markov process can be used to predict the future by knowing the present state. Towards this aim, the space of the variable of interest is discretized into M states. The Markov process is thus used to determine the probability that an event belongs to a state j knowing that for a preceding time step it belonged to a state i. This probability, noted $a_{i,j} = \mathrm{P}[X_{t+1} = j|X_t = i]$, is called *transition probability.* It is considered herein that $a_{i,j}$ is independent of t (Discrete-time Markov process). The transition probabilities can be grouped in a matrix of size $M \times M$ called transition matrix $\mathbf{P}$ [23]. According to the Chapman-Kolmogorov equations, by knowing the initial state, the probabilities of belonging to the M states after t transitions, $\mathbf{q}(t)$, are [23]:

$$\mathbf{q}(t) = \mathbf{q}_{ini}\mathbf{P}^t \tag{1}$$

where the vector $\mathbf{q}_{ini}$ contains the probabilities of belonging to the states at an initial time –i.e., $t = 0$. In this study, the variable of interest is the concentration of chlorides at the cover depth which controls corrosion initiation. Therefore, the Markov processes will provide the probability that the concentration of chlorides at the cover depth belongs to a given state in time –i.e., $\mathbf{q}(t) = \mathbf{C}(t) = \mathbf{q}_{ini}\mathbf{P}^t$. If it is supposed that after construction ($t = 0$) the concentration of chlorides at the cover depth is zero, all the concentrations will belong to the first state, and the vector $\mathbf{q}_{ini}$ becomes $\mathbf{q}_{ini} = \mathbf{C}_{ini} = [1, 0, 0, ..., 0]$.

In several applications of Markov chains, the transition matrix is obtained from experimental data or expert judgment. However, for chloride penetration, the assessment of $\mathbf{P}$ from experimental measurements presents two difficulties:

- Given that the rate of chloride ingress into concrete is slow, it is difficult to study the evolution of chloride ingress before construction. Accelerated tests or the use of data obtained for similar materials could be considered to solve this problem.

- Since chloride ingress is influenced by several parameters such as concrete properties, environmental exposure, etc. [24, 25]; it is necessary to perform a large number of measurements to obtain a good representation of the phenomenon. This is difficult to pursue in practice because experimental testing is expensive and time-consuming.

Consequently, it is proposed herein to estimate the probabilities $\mathbf{C}(t)$ on the basis of a numerical model of chloride penetration. The computed values $\widehat{\mathbf{C}}(t)$ will be used afterwards to estimate the transition matrix $\mathbf{P}$. Section 3.1 outlines the model of chloride penetration considered. Section 3.2 describes the method proposed to compute the transition matrix.

3.1. Modeling chloride ingress in unsaturated environments

The chloride ingress model adopted considers the interaction between three physical phenomena: *chloride ingress*, *moisture diffusion* and *heat transfer*. Each phenomenon is represented by a partial differential equation expressed in the following general form:

$$\zeta \frac{\partial \psi}{\partial t} = \underbrace{\operatorname{div} J}_{\text{diffusion}} + \underbrace{\operatorname{div} J'}_{\text{convection}} \tag{2}$$

where ζ represents the studied parameter, t is the time and the correspondence between ψ, J, J' and the terms for the physical problem is presented in Table 2.

For chloride ingress, C_{fc} is the concentration of free chlorides, h is the relative humidity and D_c^* and D_h^* represent the apparent chloride and humidity diffusion coefficients, respectively:

$$D_c^* = D_{c,ref} \frac{f_1(T) f_2(t) f_3(h)}{1 + (1/w_e)\,(\partial C_{bc}/\partial C_{fc})} \tag{3}$$

$$D_h^* = D_{h,ref} \frac{g_1(h) g_2(T) g_3(t_e)}{1 + (1/w_e)\,(\partial C_{bc}/\partial C_{fc})} \tag{4}$$

where $D_{c,ref}$ and $D_{h,ref}$ are reference diffusion coefficients measured to standard conditions [26], w_e is the evaporable water content, and f_i and g_i are correction functions to account for the effects of temperature, relative humidity, aging and degree of hydration of concrete. These functions are detailed in [24]. The term $\partial C_{bc}/\partial C_{fc}$ represents the binding capacity of the cementitious system which relates the free and bound chlorides concentration at equilibrium. This relationship is considered herein by using the Langmuir isotherm [24].

For moisture diffusion, the humidity diffusion coefficient D_h is estimated by accounting for the influence of the parameters presented in eq. 2. The term $\partial w_e/\partial h$ (Table 2) represents the moisture capacity which relates the amount of free water, w_e, and the pore relative humidity, h. For a given temperature this relationship has been determined experimentally by adsorption isotherms. According to the Brunauer-Skalny-Bodor model, the adsorption isotherm depends on temperature, water/cement ratio, w/c, and the

degree of the hydration attained in the concrete, t_e. This work adopts such a model to represent the moisture capacity.

Finally, for heat transfer (Table 2) ρ_c is the concrete density, c_q is the concrete specific heat capacity, λ is the thermal conductivity of concrete and T is the temperature inside the concrete after time t.

The flow of chlorides into concrete is estimated by solving simultaneously the system of equations described by eq. 2 and Table 2. The numerical approach used to solve the coupled system of partial differential equations combines a finite element formulation with finite difference to estimate the spatial and temporal variation of C_{fc}, h and T [24]. Then, the time to corrosion initiation, t_{ini}, is estimated by comparing the chloride concentration at the cover depth, c_t, with a concentration threshold for corrosion initiation C_{th}.

3.2. Estimation of transition matrix from simulations

Transition matrices are estimated from Monte Carlo simulations of the model of chloride penetration presented previously. The random variables used in the simulation are presented in Table 3. Each simulation records the chloride concentration at cover depth during time. From all simulations, it is determined the frequency of belonging to a given state –i.e., histogram. The evaluation of the probability that the chloride concentration at the cover depth belongs to the state j at time t is assessed by computing:

$$\widehat{C}_j(t) = \frac{n_j(t)}{N} \tag{5}$$

where $n_j(t)$ is the number of observations in the state j measured at time t and N is the number of simulations. Figure 1 shows the variation of $\widehat{C}_j(t)$ obtained from Monte Carlo simulations and Markov chains considering: $N = 100,000$ simulations, $M = 10$ states and for the original material (Table 3). As expected, whereas the probability of belonging to the first state decreases with time, it increases for the last state.

Once the probabilities $\widehat{\mathbf{C}}(t)$ have been estimated, the transition probabilities can be determined by minimizing the difference between $\widehat{\mathbf{C}}(t)$ and the probabilities obtained from the Markov model (eq. 1). Thus, there are M functions to minimize:

$$\begin{cases} \min\limits_{\mathbf{a}} \max\limits_{\mathbf{F}} \mathbf{F}(\mathbf{a}) = (f_1(\mathbf{a}), f_2(\mathbf{a}), \ldots, f_M(\mathbf{a}))^T \\ \text{u.c. } a_{i,j} \geq 0 \text{ and } \sum\limits_{j=0}^{\infty} a_{i,j} = 1 \end{cases} \tag{6}$$

where $\mathbf{a}$ is a vector containing the transitions probabilities (optimization parameters) and $f_j(\mathbf{a})$ is the explained sum of squares for each state j:

$$f_j(\mathbf{a}) = \sum_{t=0}^{t_{ana}} (\widehat{C}_j(t) - C_j(t, \mathbf{a}))^2 \tag{7}$$

where t_{ana} represents the analysis period used to perform the adjustment. This problem of multi-objective optimization has been solved by using the "optimization toolbox" of Matlab©. The selected optimization method minimizes the maximum value of a set of multi-variable functions from an initial value.

The optimization problem is very sensitive to the kind of Markov Model chosen. In other words, the difference between Monte-Carlo and Markov simulations decreases when the Markov model considers optimal values for: (i) the number of terms $a_{i,j}$, (ii) the number of states M, and (iii) an initiation phase t_0^M. These numerical aspects are discussed in detail in [27]. Various studies consider a Markov matrix with two terms $a_{i,j}$ per state [8, 28]. In such a case, the transition probabilities can be estimated from a non-linear regression. However, complex stochastic phenomena cannot be modeled by a transition matrix with two transition probabilities per state [8]. When all the terms a_{ij} of the transition matrix are estimated, it has been observed that most terms are close to zero [27]. Consequently, after optimizing the number of terms per state $a_{i,j}$, this study considers a transition matrix with three terms per state. For instance, for $M = 5$ states, the transition matrix becomes:

$$\mathbf{P} = \begin{bmatrix} a_{1,1} & a_{1,2} & a_{1,3} & 0 & 0 \\ 0 & a_{2,2} & a_{2,3} & a_{2,4} & 0 \\ 0 & 0 & a_{3,3} & a_{3,4} & a_{3,5} \\ 0 & 0 & 0 & a_{4,4} & a_{4,5} \\ 0 & 0 & 0 & 0 & 1 \end{bmatrix} \tag{8}$$

Taking into account the data presented in Table 3 and minimizing the difference between the values predicted with Monte-Carlo and Markov models, it is adopted a transition matrix with $M = 10$ states [27]. The consideration of an initiation phase, t_0^M, improves the predictability of the Markov model. This initiation phase represents the time needed for chlorides to reach the cover depth. Therefore, for a new or repaired structure, the defects belonging to the first state are forced to remain in such a state during the initiation phase [29]. The length of this initiation phase varies in terms of the diffusivity of the repair material. For the material presented in Figure 1, an initiation phase of $t_0^M = 15$ year minimizes the explained sum of squares. Considering this value, $M = 10$ and three transition probabilities per state, Figure 1 presents a comparison between the probabilities estimated from Markov and Monte-Carlo simulations for the states 1, 5 and 10. It is observed that, under these considerations, the probabilities estimated from the Markov model have good agreement with those computed from Monte-Carlo simulation. Therefore, this study will adopt these characteristics for modeling chloride ingress using Markov chains.

4. Probabilistic modeling of inspection and repair

4.1. Consideration of imperfect inspections

The selected maintenance strategy considers that the structure is inspected periodically every Δt years. Inspections aim to establish the "real" or "theoretical" chloride concentration at the cover depth, C, to determine the corrosion initiation probability. This information could be used by the owner/operator to decide if the structure must be repaired or not. The experimental test considered in the adopted maintenance strategy to determine the chloride concentration at a given depth (chloride profile) is based on the AFREM procedure [30, 31]. This test determines free or total chloride profiles from the titration of powder extracted from concrete cores at different depths. Although this procedure is widely used to determine chloride profiles worldwide, Bonnet et al. [32] found that there are significant differences between *theoretical* and *measured* chloride content profiles (Figure 2). Theoretical profiles are computed from analytical functions knowing the porosity and the chloride concentration in water. This difference has been related to the error in the *protocol*, the error due to *material variability* and the error induced by the *operator*. To take into account the influence of error in measurement, the difference between measured $\widehat{C}$ and theoretical C values is usually modeled by a noise [33]:

$$\eta = \widehat{C} - C \tag{9}$$

where η represents an unbiased noise. As observed in Figure 2 and as reported by [32], chloride measurements underestimate theoretical chloride concentrations. To account for this aspect, a bias b_η is added to eq. 9:

$$\widehat{C} = C + \eta + b_\eta \tag{10}$$

Errors in measurement can lead, for a given inspection, to under- or overestimations of the measured parameter. If it is underestimated, the owner/operator could decide to "do nothing" when repair is required. This erroneous decision increases the probability of failure and can produce overcharges if excessive repair should be carried out in the future. On the contrary, an overestimation generates a "wrong" decision where the early repair generates overcharges. Repair is undertaken in the adopted approach when inspection indicates that the chloride content at the corrosion cell reaches a repair value C_{rep}. This value can be defined in the construction standards or based on expert judgment and is related to the required safety level itself linked to the allowable risk. From a probabilistic point of view, two measures can be defined to consider imperfect inspections:

- *probability of good assessment*, PGA: determines the probability of detecting an event (e.g. chloride concentration higher than the repair value) given that the event exists:

$$\text{PGA} = \text{P}(\widehat{C}(\mathbf{X}) \geq C_{rep} | C(\mathbf{X}) \geq C_{rep}) \tag{11}$$

- *probability of wrong assessment*, PWA: establishes the probability of detecting an event given that it does not exist:

$$\text{PWA} = \text{P}(\widehat{C}(\mathbf{X}) \geq C_{rep} | C(\mathbf{X}) < C_{rep}) \tag{12}$$

where $\mathbf{X}$ is a vector containing the considered random variables.

The probabilities of good and wrong assessment are estimated taking into account the probability density functions (PDF) of the noise and the signal [34, 33]. Therefore, the PDFs of noise and signal (chloride concentration) and the repair concentration should be determined for a particular problem. They will depend on the characteristics of both the inspection technique and the inspected parameter. It is supposed herein that the noise is independent of the real chloride concentration because there are several sources of error influencing the results of the inspection. This assumption has been validated from a series of measurements on metallic harbor structures [12]. It is also assumed that the distribution of the noise and the repair concentration are constant for each inspection.

The noise is usually modeled by a normal random variable. Nonetheless, on the basis of experimental measurements, Bonnet et al [32] report that a generalized extreme value distribution (GEV) is more appropriate to represent the noise for the adopted technique of inspection. The parameters estimated (in kg/kg of concrete) are: scale $K_\eta = 0.016$, shape $\sigma_\eta = 9.3 \times 10^{-5}$ and location $\mu_\eta = -8.4 \times 10^{-5}$. For a density of concrete of $\rho_c = 2400$ kg/m^3, the shape and location of η in kg of chlorides per m^3 are $\sigma_\eta = 0.2$ and $\mu_\eta = -0.2$, respectively. By considering that the chloride concentration cannot be negative, and by fitting some chloride concentrations obtained from the model presented in section 3.1, it is assumed herein that C is log-normally distributed with mean μ_C and standard deviation σ_C. Taking into account that, for this problem, the noise is modeled as a random variable with an important variability, it is possible to consider that the events $\langle \widehat{C}(\mathbf{X}) \geq C_{rep} \rangle$ and $\langle C(\mathbf{X}) \geq C_{rep} \rangle$ are weakly correlated. Consequently, it is assumed that both events are independent and that the probability of good assessment becomes:

$$\text{PGA} = \text{P}[g(\mathbf{X}) \leq 0)] \tag{13}$$

where $g(\mathbf{X})$ is the limit state function. In this problem, there is no analytical solution to estimate PGA from eq. 13. Therefore, PGA is numerically computed by using a first order reliability method (FORM) where the limit state function is:

$$g(\mathbf{X}) = C_{rep} - C(\mathbf{X}) - \eta(\mathbf{X}) \tag{14}$$

It is assumed in eq. 14 that, knowing the bias, the measurements can be unbiased –i.e., $b_\eta = 0$. By taking into account that the repair concentration C_{rep} is deterministic and that η follows a GEV distribution, the probability of wrong assessment can be estimated as:

$$\text{PWA} = 1 - \exp\left(- \left[1 + K_\eta \left(\frac{C_{rep} - \mu_\eta}{\sigma_\eta} \right) \right]^{-1/K_\eta} \right) \tag{15}$$

Equation 14 shows that PGA depends on the chloride content C, the repair concentration C_{rep}, and the characteristics of the noise. From the probabilistic model of chloride penetration presented in section 3.1, it has been determined that the standard deviation of C for ordinary Portland concrete is about $\sigma_C = 0.10$ kg/m^3. However, this parameter should be determined for a particular problem. Figure 3a presents the influence of σ_C on PGA when $\mu_\eta = -0.2$ kg/m^3, $\sigma_\eta = 0.2$ kg/m^3 and $C_{rep} = 1.6$ kg/m^3. Overall behavior indicates that PGA increases for higher total chloride contents. Note that small variations of σ_C from the estimated value do not produce appreciable changes in PGA. Then, this study adopts the value estimated from simulations –i.e., $\sigma_C = 0.10$. These results indicate that the precision of the inspection technique is not suitable for measuring small chloride concentrations. However, it can be used to determine concentrations higher than 0.3 kg/m^3 which are compared with the threshold chloride concentration for corrosion initiation, that varies between 1 and 2.5 kg/m^3, to determine if the structure should be repaired or not. Further research should be addressed to improve the precision of the inspection technique.

Figure 3b describes the interaction between the repair concentration C_{rep} and PGA. These values were obtained for the following conditions: $\sigma_C = 0.10$ kg/m^3, $\mu_\eta = -0.2$ kg/m^3 and $\sigma_\eta = 0.10$ kg/m^3. PGA is very sensitive to C_{rep} diminishing when the repair concentration increases. The repair concentration indicates a limit after which the structure is repaired. This value can be defined according to the construction standards or based on expert judgment. The French standard [35] establishes that the maximum allowable chloride content is 0.4% wt. cem. ($\approx$1.6 kg/m^3 for a cement content of 400 kg per m^3 of concrete and $\rho_c = 2400$ kg/m^3) whereas the Spanish standard [36] recommends a higher value (i.e., 0.6% wt. cem. $\approx$ 2.4 kg/m^3). Taking into account the higher influence of this parameter on PGA, the considerations for selecting a representative value will be discussed in section 6.

The influence of the mean and standard deviation of noise on PGA is presented in Figure 4. The PGAs presented in this figure were computed with the following parameters: $\sigma_C = 0.10$ kg/m^3 and $C_{rep} = 1.6$ kg/m^3. As expected, the PGA are larger when μ_η tends to zero and the change in PGA from 0 to 1 is more pronounced when σ_η increases. This indicates that the precision of the inspection technique is improved for smaller μ_η and σ_η. For the value reported by [32], the PGA vary from 2% to 100% for chloride contents between 1 and 2.2 kg/m^3 which are in the interval of interest.

Given that the probability of wrong assessment is only influenced by the characteristics of the noise and the repair concentration –i.e., eq. 15, Figure 5a presents the influence of the repair concentration on PWA for $\mu_\eta = -0.2$ kg/m^3 and $\sigma_\eta = 0.2$ kg/m^3. Note that although PWA is sensitive to changes in C_{rep}, their values are low ($< 10^{-3}$) for C_{rep} varying between 0.8 and 2.4 kg/m^3. The higher PWA correspond to small repair concentrations. This indicates that the area below the PDF of the noise after C_{rep} (PWA) increases when C_{rep} is reduced (Figure 5b). This behavior is expected because this inspection technique always underestimates the real chloride content. Then, the decisions are principally based on the probability of good assessment in this maintenance strategy.

4.2. Simulation of inspection, repair and corrosion initiation

After inspection, there can be two decision outcomes:

1. to repair, in which case the structure/component returns to its initial state (this assumption is adopted herein but it can be changed without lack of generality of the method) or
2. to carry out no repair.

If no repair is decided, there is a remaining probability of corrosion initiation that increases when the chloride content is larger. When corrosion initiation occurs the structure/component is repaired or rebuilt to its initial state. These events are represented by probabilities that are calculated analytically and integrated in the Markov model. Therefore, there are two Markov matrices for modeling the whole process. The former is used to model corrosion initiation and repair between inspections $\mathbf{P}^{be}$. The latter represents inspection, repair and corrosion initiation at inspection times $\mathbf{P}^{in}$.

Once both matrices have been formulated they are used to simulate chloride ingress, inspection, repair and corrosion initiation over time. Each state is assumed to have an initial population of defects. The number of defects in each state is then calculated on a yearly basis using the relevant transition matrix, and the number of defects in each state from the previous year. This section presents the methodology for determining $\mathbf{P}^{be}$ and $\mathbf{P}^{in}$.

The variable of interest (i.e., chloride concentration at the cover depth) is discretized into M states in the following formulation. Each state i is therefore characterized by a random variable $d_i(\mathbf{X})$ which is assumed log-normally distributed.

4.2.1. Corrosion initiation between inspections

Chloride penetration into concrete produces corrosion initiation when no repair is carried out. The chloride concentration leading to corrosion initiation, C_{th}, is chosen in this work to compute the probability of corrosion initiation p_{ini}. By assuming that C_{th} is log-normally distributed, the probability of corrosion initiation between the inspection intervals for a state i can be estimated as:

$$p_{ini,i} = \mathrm{P}(C_{th}(\mathbf{X}) - d_i(\mathbf{X}) \leq 0) = \Phi\left(\frac{\lambda_{d_i} - \lambda_{C_{th}}}{\sqrt{\xi_{d_i} + \xi_{C_{th}}}}\right) \tag{16}$$

where λ and ξ are the log-normal location and scale of C_{th} and d_i.

The transition probabilities $a_{i,j}$ in the complete Markov matrix for a year between inspections $\mathbf{P}^{bet}$ are computed in terms of those of the Markov growth matrix $\mathbf{P}^{gr}$ (estimated according to the methodology described in section 3) and the probability of corrosion initiation for the corresponding state as [34]:

$$a_{i,j}^{bet} = \begin{cases} a_{i,j}^{gr} + \sum_{k=2}^{M}[a_{i,k}^{gr}\ p_{ini,k}] & \text{for } j = 1 \\ a_{i,j}^{gr}(1 - p_{ini,j}) & \text{otherwise} \end{cases} \tag{17}$$

where $a_{i,j}^{bet}$ and $a_{i,j}^{gr}$ are the transition probabilities of $\mathbf{P}^{bet}$ and $\mathbf{P}^{gr}$, respectively.

4.2.2. Corrosion initiation and repair at inspection times

At an inspection year, the events of preventive repair R or corrosion initiation F can occur (i.e., $R \cup F$). Therefore, the probability of repair or corrosion initiation for a state i, $p_{R\cup F,i}$, can be computed as follows:

$$p_{R\cup F,i} = \mathrm{P}_i[\text{Repair}] + \mathrm{P}_i[\text{Corrosion initiation} \mid \text{No repair}] \tag{18}$$

where:

$$\mathrm{P}_i[\text{Repair}] = \mathrm{P}[\hat{C} \geq C_{rep}] \tag{19}$$

Taking into account that inspections are imperfect and could lead to "repair" or "no repair" after corrosion initiation, it is assumed that the events ⟨Corrosion initiation⟩ and ⟨No repair⟩ are independent. Thus:

$$\mathrm{P}_i[\text{Corrosion initiation} \mid \text{No repair}] = \mathrm{P}[\hat{C} < C_{rep}]p_{ini,i} \tag{20}$$

On the other hand, the parameter γ_i represents the probability that for the state i the actual chloride concentration is greater than the repair concentration:

$$\gamma_i = \mathrm{P}[d_i(\mathbf{X}) \geq C_{rep}] \tag{21}$$

The parameter γ_i can be determined based on expert judgment. However, as mentioned previously, by assuming that $d_i(\mathbf{X})$ is log-normally distributed, γ_i is calculated analytically as:

$$\gamma_i = \Phi\left(\frac{\lambda_{d_i} - \ln(C_{rep})}{\xi_{d_i}}\right) \tag{22}$$

The probability of repair can be written analytically in terms of γ_i, PGA_i and PWA_i:

$$\mathrm{P}_i[\text{Repair}] = \mathrm{PGA}_i\gamma_i + \mathrm{PWA}_i(1-\gamma_i) \tag{23}$$

Similarly, the probability of corrosion initiation given that no repair is carried out writes:

$$\mathrm{P}_i[\text{Corrosion initiation} \mid \text{No repair}] = [(1-\mathrm{PGA}_i)\gamma_i + (1-\mathrm{PWA}_i)(1-\gamma_i)]p_{ini,i} \tag{24}$$

These values are then used to calculate the transition probabilities in the complete Markov matrix for an inspection year $\mathbf{P}^{in}$ in terms of those of the Markov growth matrix $\mathbf{P}^{gr}$ and the probability of corrosion initiation for the corresponding state as [34]:

$$a_{i,j}^{in} = \begin{cases} a_{i,j}^{gr} + \sum_{k=2}^{M}[a_{i,k}^{gr}\ p_{R\cup F,k}] & \text{for } j=1 \\ a_{i,j}^{gr}(1-p_{R\cup F,j}) & \text{otherwise} \end{cases} \tag{25}$$

where $a_{i,j}^{in}$ and $a_{i,j}^{gr}$ are the transition probabilities of $\mathbf{P}^{in}$ and $\mathbf{P}^{gr}$, respectively.

5. Cost analysis

Depending on both the kinematics of the deterioration process and the repair strategy, the number of defects in each state stabilizes after a given number of simulations [34]. These stabilized values are used to evaluate the cost of the maintenance strategy. This methodology distinguishes between three costs:

- the expected inspection cost, $\mathrm{E}[c_I]$,
- the expected repair cost, $\mathrm{E}[c_R]$ and
- the expected corrosion initiation cost, $\mathrm{E}[c_F]$.

The expected total cost of the maintenance strategy, $\mathrm{E}[c_T]$, is then computed as:

$$\mathrm{E}[c_T] = \mathrm{E}[c_I] + \mathrm{E}[c_R] + \mathrm{E}[c_F] \tag{26}$$

Although the aforementioned costs are agency (direct) costs, user costs can also be added to the analysis. According to Thoft-Christensen [37], in some cases, the consideration of user costs could be an influencing factor in management of deteriorating infrastructure. However, taking into account that no data is available to accurately quantify the user cost, it was decided to model only the agency costs. The development of accurate cost models for agency costs is beyond the scope of this study. Nonetheless, the cost parameters presented herein were defined on the basis of expert judgment of the operators of the port of Nantes-Saint-Nazaire in France.

For comparative purposes, the cost models described herein are computed on an annual basis. In other words, these costs indicate the annual expenditures of the agency to maintain a structure. Decision-making based on expected costs is widely used in the practice. However, Schoefs et al. [38] demonstrated that decision-making based on values different than the expected one (e.g., 5 or 95% quantiles) are more convenient in some cases where a representative value is hard to find. Although this point is not explored in this work, it is an important area for further research.

5.1. Annual inspection cost

The annual inspection cost should take into account all the stages of the inspection process: preparation of the access to the inspected area, extraction of concrete cores, laboratory tests, data analysis, etc. Therefore, for a particular problem, this cost could be considered as constant. The expected inspection cost is calculated by assuming that all structures/components are inspected. Then, the expected annual number of inspections n_I is equal to the total number of components to inspect n_t divided by the length of the inspection interval Δt:

$$\mathrm{E}[n_I] = \frac{n_t}{\Delta t} \tag{27}$$

Thus, the expected annual inspection cost $\mathrm{E}[c_I]$ is evaluated as a fraction of the initial cost of construction c_0 as:

$$\mathrm{E}[c_I] = \mathrm{E}[n_I] c_0 k_I \tag{28}$$

where k_I is a coefficient used to compute the inspection cost as a fraction of c_0.

5.2. Annual repair cost

Repair costs depend on the maintenance strategy, deterioration mechanisms, labor costs, material costs, site setup costs etc. The proposed approach assumes that the structure is repaired before corrosion initiation (preventive maintenance). Therefore, the annual repair cost only involves the expenditures incurred in avoiding corrosion initiation by the owner. For example, in the studied maintenance strategy, this cost considers mainly the cost of rebuilding chloride-polluted concrete. The deterioration mechanism defines the expected number of repairs which is computed from the results of the Markov model. Then, these costs are defined as a function of the characteristics of the repair technique.

As mentioned before, repair is carried out when inspection indicates that chloride concentration is larger than the repair concentration. Therefore, the number of repairs n_R depends on the probability of repair (eq. 23) and the stabilized number of structures/components in each state $n_d(i)$ [34]:

$$\mathrm{E}[n_R(i)] = \frac{n_d(i)}{\Delta t} \mathrm{P}_i[\mathrm{Repair}] \tag{29}$$

Thus the expected annual repair cost $\mathrm{E}[c_R]$ becomes:

$$\mathrm{E}[c_R] = c_0 k_R \sum_{i=1}^{M} \mathrm{E}[n_R(i)] \tag{30}$$

where k_R is a coefficient used to compute the repair cost as a fraction of c_0. It is observed from eq. 30 that the cost of repair is constant and only depends on the initial construction cost, c_0 and the repair cost coefficient k_R. When corrective maintenance is undertaken, repair costs depend mainly on the extent of damage. However, in the proposed approach the structure is repaired at a given damage threshold where the repair actions, and therefore the expected value of preventive repair costs, are known and can be considered as constant.

5.3. Annual corrosion initiation cost

The annual corrosion initiation cost is related to the overcharges of maintenance actions that should be carried out to ensure that the structure is repaired to its initial state when corrosion has started. For instance, replacement of corroded bars, removal of more chloride-polluted concrete, etc. Therefore, in the considered maintenance strategy, the annual corrosion initiation cost accounts for the overcharges incurred by corrective repair. The assessment of this cost depends on the number of components corroded between

inspections and at inspection times. The expected annual total cost of corrosion initiation $\mathrm{E}[c_F]$ is then calculated by summing the expected cost of corrosion initiation between inspections $\mathrm{E}[c_{F1}]$ and the expected cost of corrosion initiation at inspection year $\mathrm{E}[c_{F2}]$:

$$\mathrm{E}[c_F] = \mathrm{E}[c_{F1}] + \mathrm{E}[c_{F2}] \tag{31}$$

According to eq. 16, there is a probability of corrosion initiation that depends on the actual chloride concentration for each state. Therefore, the expected number of corroded components in each state for a year between inspections, $\mathrm{E}[n_{F1}]$, is computed by multiplying the number of corroded components in each state i each time between inspections j, $n_f(i,j)$, by its corresponding probability of corrosion initiation. At each year between inspections, there will be a different number of corroded components in each state. Therefore, the expected number of corroded components in each state must be calculated and summed together (assuming that they are independent):

$$\mathrm{E}[n_{F1}(i)] = \frac{p_{ini}(i)}{\Delta t} \sum_{j=1}^{\Delta t-1} n_f(i,j) \tag{32}$$

Then, the expected annual cost of corrosion initiation between inspections writes:

$$\mathrm{E}[c_{F1}] = c_0 k_F \sum_{i=1}^{M} \mathrm{E}[n_{F1}(i)] \tag{33}$$

where k_F is a coefficient used to compute the corrosion initiation cost as a fraction of c_0.

When repairs are carried out at inspection times, the probability of corrosion initiation given no repair (eq. 24) is used to compute the expected number of corroded components, $\mathrm{E}[n_{F2}(i)]$:

$$\mathrm{E}[n_{F2}(i)] = \frac{n_d(i)}{\Delta t} \mathrm{P}_i[\text{Corrosion initiation}|\text{No repair}] \tag{34}$$

where $n_d(i)$ is the number of structures/components in each state. Then the expected annual cost of corrosion initiation at inspection year becomes:

$$\mathrm{E}[c_{F2}] = c_0 k_F \sum_{i=1}^{M} \mathrm{E}[n_{F2}(i)] \tag{35}$$

6. Illustrative example

6.1. Problem description

The objective of this example is to illustrate the proposed methodology studying the influence of the inspection interval on the expected annual maintenance costs of a corroding RC structure. It is supposed that this structure is composed by 100 structural components placed in an oceanic climate with temperature ranging from 5 to 25°C and with relative humidity between 0.6 and 0.8 for each year. The stochastic

climate model is described in [24]. The structural components are supposed to be in a splash and tidal zone characterized by a larger environmental chloride concentration and where chlorides ingress is accelerated by the variations in temperature and humidity [24]. The environmental chloride concentration for this zone is modeled as a stochastic process generated by independent numbers following a log-normal distribution (log-normal noise) with a mean of $\mu C_{env} = 6$ kg/m^3 and a coefficient of variation of 0.2 [24]. The considered random variables for chloride penetration are presented in Table 3. These random variables have been defined based on previous experimental and probabilistic studies [24, 39]. Table 3 includes chloride diffusion coefficients for four materials. The first one corresponds to the original material which is supposed to be ordinary Portland concrete. The following three materials correspond to repair materials that are being tested in the framework of the MAREO project by accelerated and normal tests. The chloride diffusion coefficients for these repair materials were defined based on preliminary results [40]. It is important to mention that although these coefficients are used to study the influence of the quality of the repair techniques, the conclusions concerning the repair techniques are not general.

Following the procedure presented in section 3, the Markov matrices of deterioration were obtained for the four materials (original and repair materials). To estimate these Markov matrices, the problem is discretized into $M = 10$ states varying from 0 to 4 kg/m^3 by steps of 0.4 kg/m^3. The transition probabilities were obtained from 100,000 simulations for a concrete cover of 5 cm. The lengths of the initiation phase for all the materials are 15, 24, 31 and 10 years for the original material, wet shotcrete, dry shotcrete and formed concrete respectively. It is noted that, excepting formed concrete, the repair materials have initiation phases larger than the original material. This behavior is expected because according to the material specifications, these materials have lower chloride diffusivity. Unfortunately, it was impossible to obtain the material composition to study in depth its influence on chloride diffusivity.

The determination of an optimal inspection interval is very sensitive to the cost models. Therefore, to obtain realistic results, the inspection, repair and corrosion initiation coefficients (i.e., k_I, k_R and k_F) were defined by taking into account the average expenditures for these items incurred by the port of Nantes-Saint-Nazaire in France. These coefficients are presented in Table 4. Note that these costs coefficients are refereed to an initial construction cost of 1000 units per structural element. The coefficient of corrosion initiation is usually equal to or greater than 1 in engineering applications based on ultimate limit states. In such cases corrosion initiation implies a loss in the serviceability capacity and/or the replacement of the affected component. However, for the adopted limit state, the coefficient given in Table 4 indicates that after corrosion initiation the overcharges are lower than the initial cost of construction.

6.2. Results

The first part of the results is devoted to study the influence of the length of the inspection interval on the total costs and its governing parameters. To simplify the study, it is assumed that the repair material

has the same characteristics as the construction material. Finally, the discussion focuses on the use of the proposed approach as a decision-making tool for the selection of a given repair technique (Section 6.2.5).

6.2.1. Total costs

The influence of the length of the inspection intervals on the expected annual cost is presented in Figure 6. This analysis considers that the repair concentration is $C_{rep} = 1.6$ kg/m^3 which corresponds to the limit established in the French standard [35]. The total cost is composed of the costs of inspection, repair and corrosion initiation. It is observed that the costs of inspection and repair decrease and the cost of corrosion initiation increases for larger inspection intervals. This behavior is explained by the fact that when the inspection interval is greater, most part of inspections detect that the components are corroded. On the contrary, when the structure is inspected regularly, repair is preventive, and therefore, the cost of corrosion initiation decreases. By varying the length of the inspection interval, it can be concluded, in this case, that total costs are optimal when the structure is inspected every 7 years.

6.2.2. Repair concentration

Figure 7a shows the influence of the repair concentration C_{rep} on the expected total costs. For comparative purposes, two values of C_{rep} defined in the French [35] and Spanish [36] standards are also included –i.e., $C_{rep} = 1.6$ and $C_{rep} = 2.4$ kg/m^3, respectively. The overall behavior indicates that total expected annual cost is sensitive to this parameter. For $C_{rep} < 1.6$ kg/m^3, the optimal inspection intervals decrease when C_{rep} is larger. It is also noted that there is no minimum value when the repair limit is $C_{rep} = 2.4$ kg/m^3. This behavior indicates that costs could be optimized when C_{rep} is defined for a particular problem.

The assessment of an optimal C_{rep} is presented in Figure 7b. The costs presented in this figure correspond to the optimal inspection interval and include the discrimination of costs in each case. The inspection costs increase when C_{rep} is greater. This indicates that for larger values of C_{rep} there are more corrosion risks, and consequently, the structure should be inspected more periodically (Figure 7a). On the contrary, repair cost decreases and corrosion initiation cost increases for larger values of C_{rep}. When the repair concentration increases there are more corrosion risks. Therefore, the components are repaired once they have failed increasing corrosion initiation costs. These results show that the optimal inspection interval should be determined for a value lower than the suggested in the standards –i.e., $C_{rep} = 1.2$ kg/m^3. In such a case, the optimal inspection interval is 10 years.

6.2.3. Corrosion initiation threshold

The effect of the mean of the corrosion initiation threshold, C_{th}, on the total expected annual costs is presented in Figure 8. These values were obtained for a repair concentration of $C_{rep} = 1.2$ kg/m^3. The compared means were defined based on the values presented in Table 1. Then, $C_{th} = 1.92$ kg/m^3 corresponds to the mean reported by [18] and $C_{th} = 2.56$ kg/m^3 and $C_{th} = 3.16$ kg/m^3 are the values found

by [16] for "in field" and "cracked immersed" cases, respectively. Taking into account that the standard deviation reported in [16] gathers information of several studies under different conditions, it is assumed in this example that for one single structure the variability is reduced. Therefore, all threshold chloride contents have the same coefficient of variation (i.e., COV[C_{th}]=0.20). As expected, the total expected annual cost decreases when the mean of C_{th} increases. This means that a higher chloride concentration is necessary to start the corrosion reaction, and then, the number of inspections, repairs and corroded components will be reduced. As mentioned in section 2.1, a comprehensive assessment of this parameter is crucial for an optimal management of structures.

It can be also noted in Figure 8 that there is a flat zone on the curve near the minimum when the mean of C_{th} increases. Since $C_{th} = 3.16$ kg/m^3 is more than twice C_{rep}, the probability of corrosion initiation is very small reducing corrosion initiation costs. In that case, there is this flat zone where it is difficult to find an optimum value because inspection and repair costs govern total costs and decrease when the inspection interval is higher (Figure 6). Thus, it would be suitable to find an appropriate C_{rep} that will minimize costs for a given C_{th}.

6.2.4. Inspection quality

Figure 9 shows the influence of the quality of the inspection technique on the expected total annual costs. To simulate inspection quality, it is assumed that an improvement of the quality decreases the values of the parameters μ_η and σ_η. Then, Figure 9a shows that a decrease of μ_η and σ_η reduces the scatter of the measurement leading to more precise results. It is noted from Figure 9b that the expected optimal total annual cost is lower when the quality of the inspection technique increases. Furthermore, it is observed that for the same problem the structure should be inspected every 11 years for high quality instead of 10 years for low quality. These results indicate that further research is required to improve the quality of the inspection techniques, and consequently, to reduce the total costs.

6.2.5. Use of the model for decision-making

This section compares the cost-efficiency of three repair techniques that can be implemented in the maintenance strategy: formed concrete, wet shotcrete and dry shotcrete. Figure 10 presents the expected total annual cost for the three repair techniques. The optimal inspection intervals for these techniques are 8, 10 and 13 years for formed concrete, wet shotcrete and dry shotcrete, respectively. Comparing the cost of the repair techniques, it is observed that the cost of dry shotcrete is about a half of the other techniques. This behavior is explained by the fact that the repair material used in this technique has the lowest chloride diffusion coefficient (Table 3). Therefore, the largest inspections intervals correspond to this technique and the probability of corrosion initiation throughout time is lower and the costs of inspection, repair and corrosion initiation are reduced. From an economic point of view, it is concluded that dry shotcrete is the

more cost-efficient technique.

7. Conclusions

This paper focused on the improvement of a preventive maintenance strategy where repair is carried out before corrosion initiation. This strategy was developed after discussion with stakeholders that are confronted to this problem. The inspection technique measures the concentration of chlorides at the cover depth. Afterwards, based on the inspection results, the structure is repaired by replacing the polluted concrete with new material to ensure optimal levels of serviceability and safety during the structural lifetime. Given that Markov chains are more appropriate for modeling all the stages of the life-cycle (i.e., deterioration process, inspection, maintenance, failure), this paper proposed a methodology for determining the transition matrix from Monte Carlo simulations of a chloride penetration model. The Markovian approach included an initiation phase and several transition probabilities per state for improving the accuracy of the model. The inspection/maintenance strategy chosen is modeled using decision theory and Markov processes. This approach is convenient to take into account the "good" or "wrong" decisions resulting from imprecise inspection measurements as well as the uncertainties involved in the process. In addition this paper included data determined from experimental measurements or expert judgment to give more representative results. The proposed method was illustrated with an example. This example discussed the effect of the main influencing parameters as well as an application of the model for decision-making when various repair techniques can be applied. The main findings of this work are summarized as follows:

- The proposed approach can be used to determine the inspection interval that minimizes the costs of inspection, repair and corrosion initiation for the studied maintenance strategy. However, this approach can be extended to other deterioration processes or materials where maintenance actions are based on imperfect inspection results.

- For corroding RC structures, it can be concluded that the repair concentration C_{rep} should be estimated for a particular problem taking mainly into account real costs, material properties and environmental actions. For instance, the value determined in the illustrative example was lower than the values suggested in the standards. However, these results could be different for other specific applications.

- There are many factors influencing the assessment of the corrosion initiation threshold C_{th}. Therefore, further research is required to determine this parameter as a function of the conditions of the surrounding environment as well as the characteristics of the concrete/repair material.

- The increase of the quality of inspection results is essential to reduce the uncertainty on the assessment of chloride content and therefore to reduce total costs Future work on this area should be addressed

to improve the quality of the existing techniques and/or to develop new inspections techniques based on non-destructive or semi-destructive principles.

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Table 1: Total threshold chloride concentrations [16].

Exposure conditions	Mean % wt. cem.	Standard dev. % wt. cem.
– *Cracked-air*: cracks > 0.4 mm and exposed to the air above the splash zone	0.15 (0.60 kg/m^3)	0.06 (0.24 kg/m^3)
– *Cracked-immersed*: cracks > 0.4 mm and immersed in seawater	0.79 (3.16 kg/m^3)	0.39 (1.56 kg/m^3)
– *In field*: it groups measurements for cracked, uncracked, immersed and non-immersed conditions	0.64 (2.56 kg/m^3)	0.61 (2.44 kg/m^3)

Table 2: Correspondence between Eq. 2 and the governing differential equations.

Physical problem	ψ	ζ	J	J'
Chloride ingress	C_{fc}	1	$D_c^* \vec{\nabla} C_{fc}$	$C_{fc} D_h^* \vec{\nabla} h$
Moisture diffusion	h	$\partial w_e / \partial h$	$D_h \vec{\nabla} h$	0
Heat transfer	T	$\rho_c c_q$	$\lambda \vec{\nabla} T$	0

Table 3: Probabilistic models of the random variables.

Variable	**Mean**	**COV**	**Distribution**
$D_{c,ref}$ (original material)	3×10^{-11} m^2/s	0.20	log-normal
$D_{c,ref}$ (formed concrete)	4.5×10^{-11} m^2/s	0.20	log-normal
$D_{c,ref}$ (wet shotcrete)	2.0×10^{-11} m^2/s	0.20	log-normal
$D_{c,ref}$ (dry shotcrete)	1.8×10^{-11} m^2/s	0.10	log-normal
c_t	50 mm	0.25	normal[a]
U_c	41.8 kJ/mol	0.10	beta on [32;44.6]
m	0.15	0.30	beta on [0;1]
$D_{h,ref}$	3×10^{-10} m^2/s	0.20	log-normal
α_0	0.05	0.20	beta on [0.025;0.1]
n	11	0.10	beta on [6;16]
λ	2.5 W/(m °C)	0.20	beta on [1.4;3.6]
ρ_c	2400 kg/m^3	0.05	normal
c_q	1000 J/(kg °C)	0.10	beta on [840;1170]

[a]truncated at 10 mm (lower bound)

Table 4: Coefficients for cost models.

Parameter	Repair technique		
	Formed concrete	Dry shotcrete	Wet shotcrete
Initial cost of construction, c_0	1000	1000	1000
Inspection coefficient, k_I	0.005	0.005	0.005
Repair coefficient, k_R	0.15	0.21	0.26
Corrosion initiation coefficient, k_F	0.30	0.42	0.52

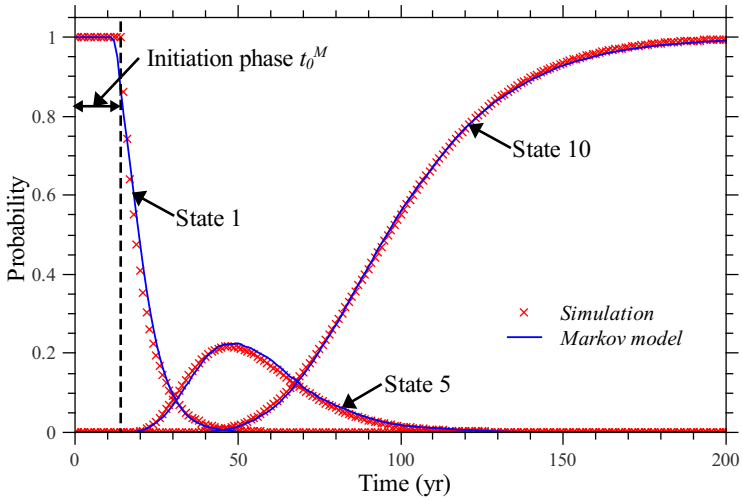

Figure 1: Comparison between the probabilities computed from Monte Carlo simulations and Markov chains.

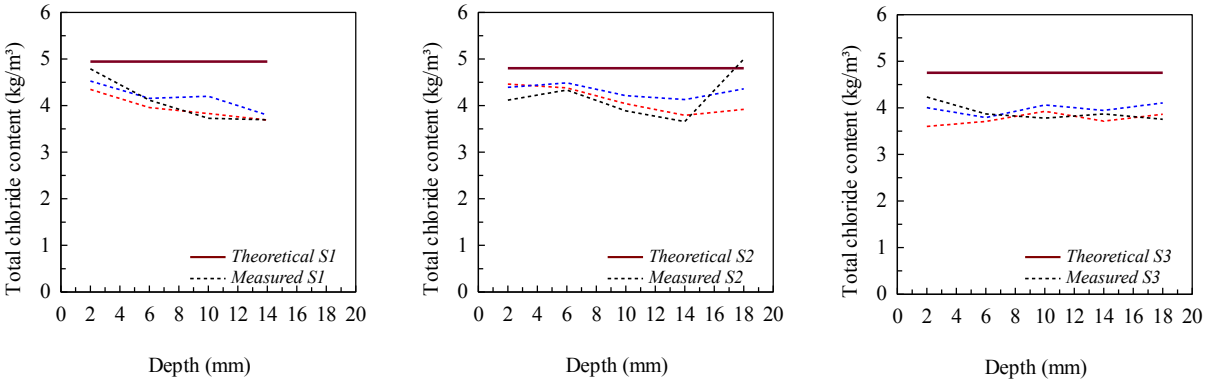

Figure 2: Differences between theoretical and measured chloride profiles for three specimens (with the values reported by [32]).

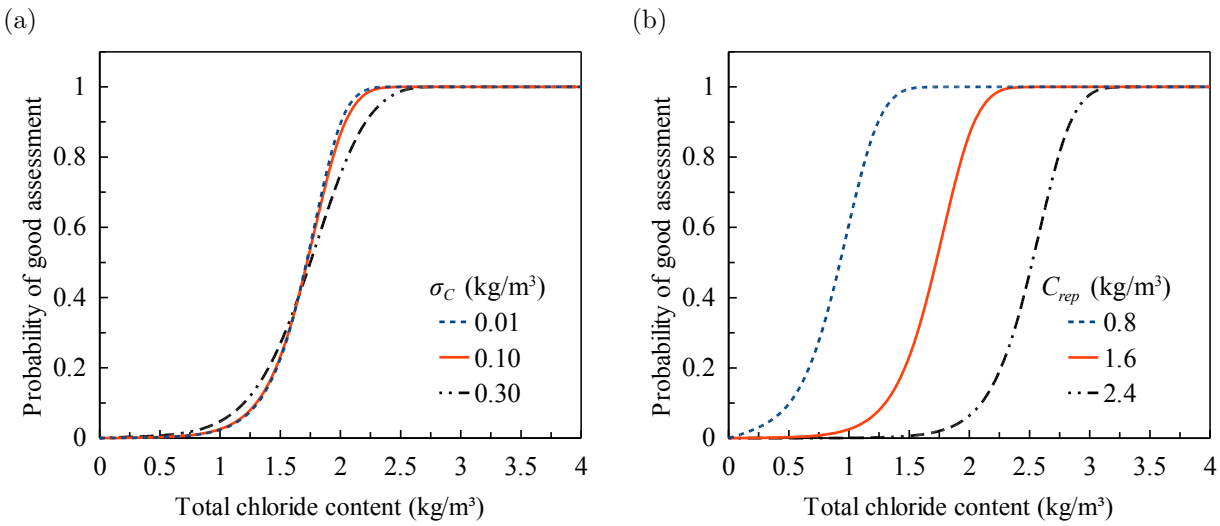

Figure 3: Factors influencing PGA: (a) σ_C and (b) C_{rep}.

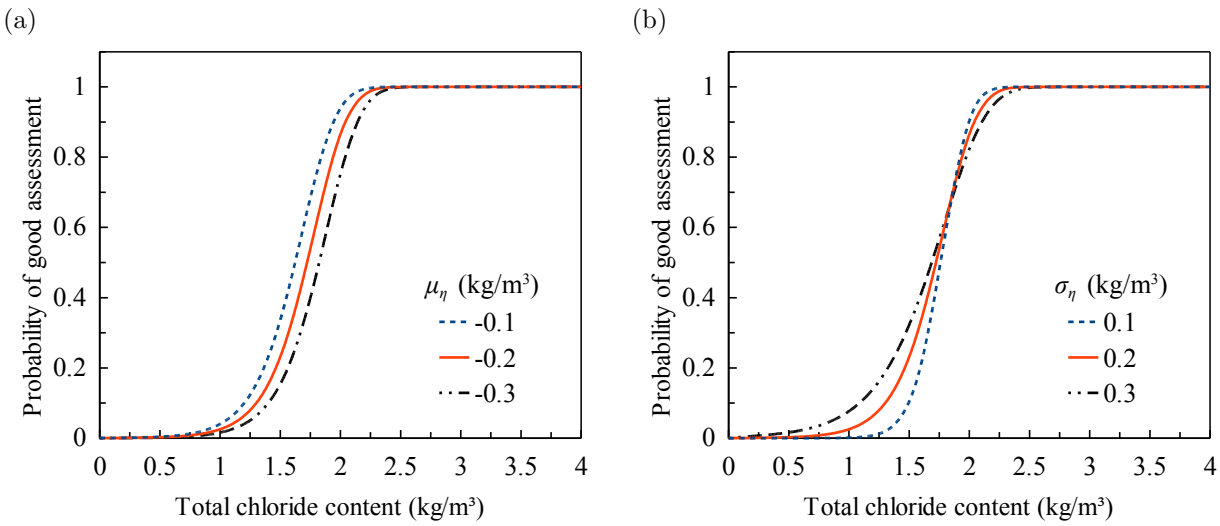

Figure 4: Factors influencing PGA: (a) μ_η and (b) σ_η.

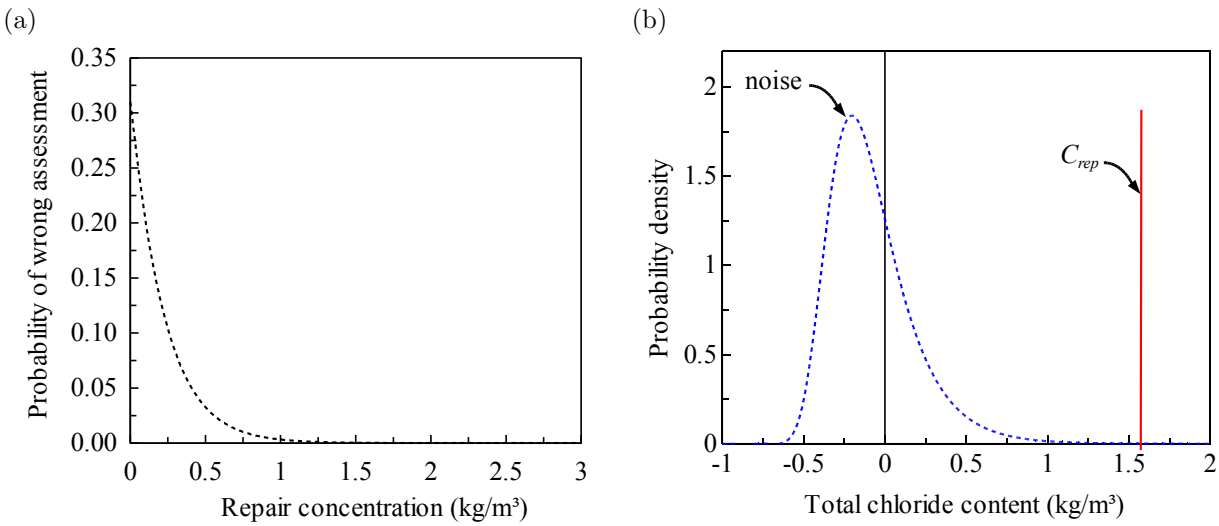

Figure 5: (a) Effect of C_{rep} on PWA. (b) PDF of noise.

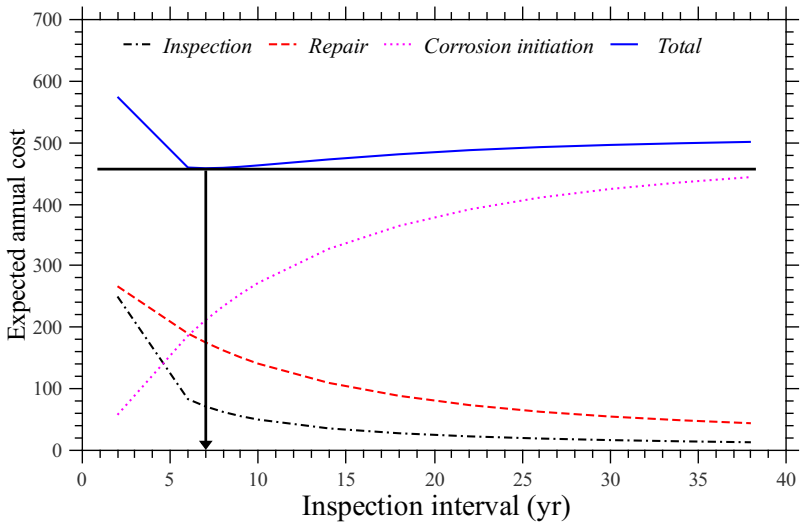

Figure 6: Effect of the inspection interval on expected annual costs.

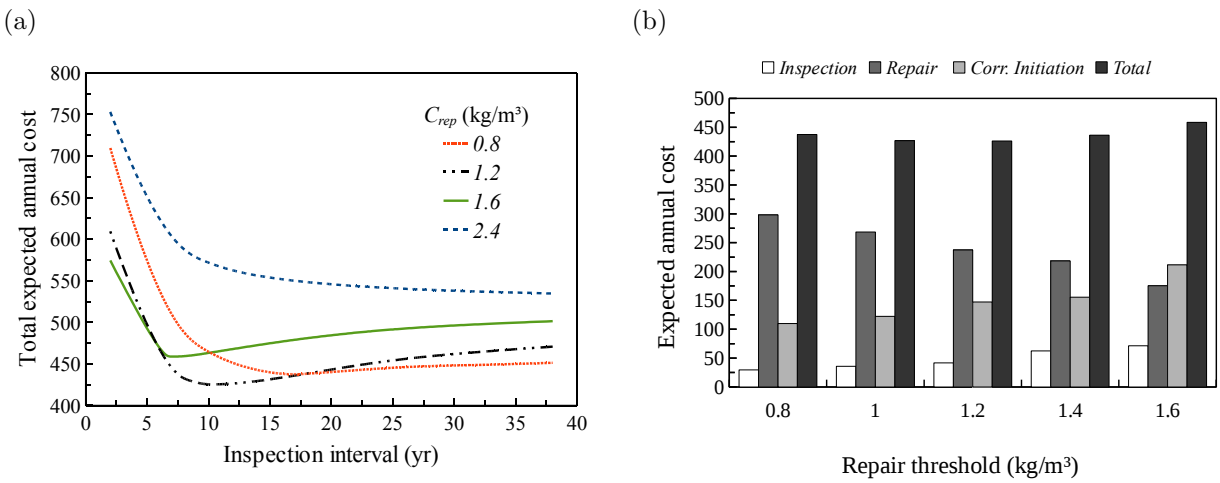

Figure 7: (a) Influence of C_{rep} on total cost. (b) Assessment of an optimal C_{rep}.

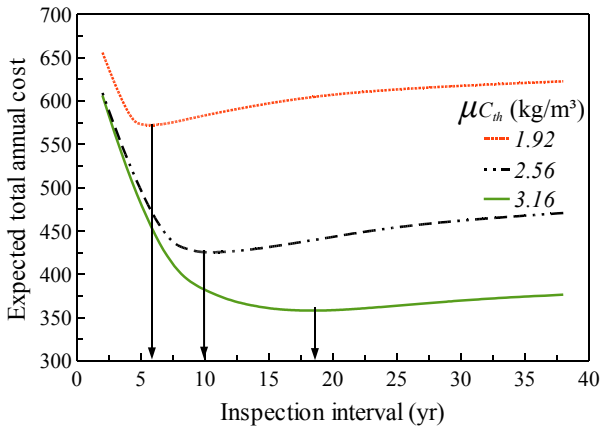

Figure 8: Impact of the mean of C_{th} on the total expected annual costs.

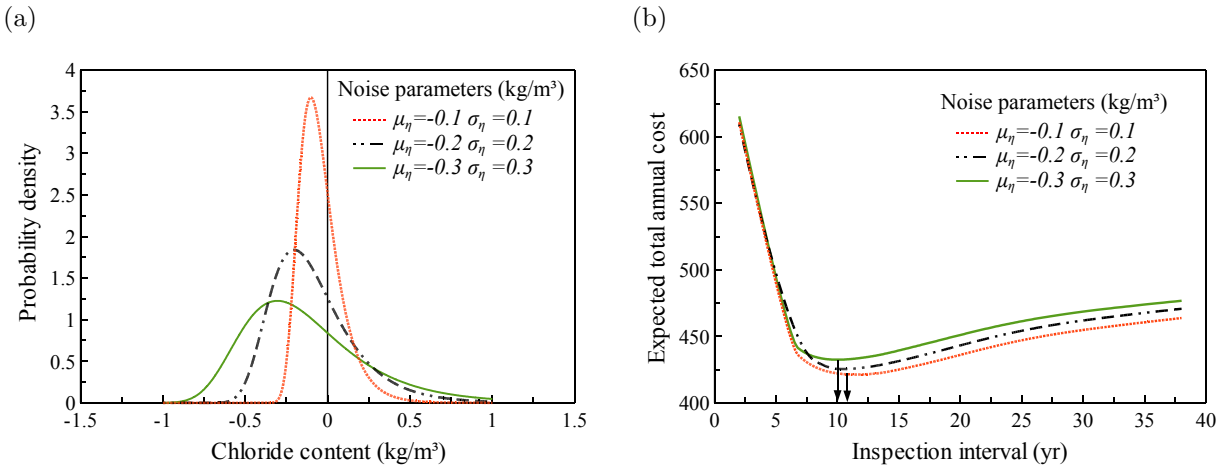

Figure 9: Influence of the quality of the inspection technique on the total expected annual costs.

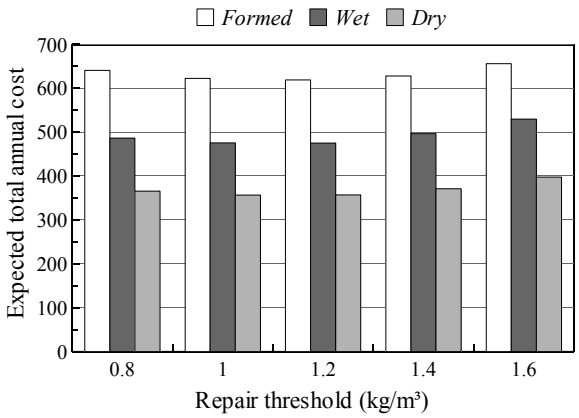

Figure 10: Expected total annual cost for various repair materials.