



HAL
open science

Probability-based maintenance modeling and planning for reinforced concrete assets subjected to chloride ingress

Quynh Truong, Charbel-Pierre El Soueidy, Yue Li, Emilio Bastidas-Arteaga

► **To cite this version:**

Quynh Truong, Charbel-Pierre El Soueidy, Yue Li, Emilio Bastidas-Arteaga. Probability-based maintenance modeling and planning for reinforced concrete assets subjected to chloride ingress. *Journal of Building Engineering*, 2022, 54, pp.104675. 10.1016/j.jobbe.2022.104675 . hal-03693118

HAL Id: hal-03693118

<https://hal.science/hal-03693118v1>

Submitted on 10 Jun 2022

HAL is a multi-disciplinary open access archive for the deposit and dissemination of scientific research documents, whether they are published or not. The documents may come from teaching and research institutions in France or abroad, or from public or private research centers.

L'archive ouverte pluridisciplinaire **HAL**, est destinée au dépôt et à la diffusion de documents scientifiques de niveau recherche, publiés ou non, émanant des établissements d'enseignement et de recherche français ou étrangers, des laboratoires publics ou privés.

1 **Probability-based Maintenance Modeling and Planning for Reinforced**
2 **Concrete Assets Subjected to Chloride Ingress**

3
4 Quynh Chau Truong^{1,2}, Charbel-Pierre El Soueidy¹, Yue Li³, Emilio Bastidas-Arteaga^{4,*},

5
6 ¹ Institute for Research in Civil and Mechanical Engineering (GeM - UMR 6183), Université de Nantes, Nantes,
7 France

8 ² Faculty of Project Management, The University of Danang – University of Science and Technology, Danang,
9 Viet Nam

10 ³ Department of Civil and Environmental Engineering, Case Western Reserve University, Cleveland, Ohio
11 44106, USA

12 ⁴ Laboratory of Engineering Sciences for the Environment (LaSIE - UMR CNRS 7356), La Rochelle University,
13 La Rochelle, France

14
15
16 *Corresponding author : Emilio Bastidas-Arteaga

17 Phone : +33 (0)5 86 56 22 32

18
19 E-mail addresses and ORCID:

20 quynh-chau.truong@etu.univ-nantes.fr, ORCID: 0000-0002-6600-7147 (Q.C Truong),

21 charbel@univ-nantes.fr (C.P. El Soueidy),

22 yxl1566@case.edu, ORCID: 0000-0002-2654-1580 (Y. Li)

23 ebastida@univ-lr.fr, ORCID: 0000-0002-7370-5218 (E Bastidas-Arteaga)

24
25
26
27
28
29
30
31
32 **Submitted to Journal of Building Engineering**

33 **(May 2022)**

35 **Abstract**

36 Chloride ingress into concrete is the primary cause of steel corrosion that leads to the deterioration of
37 reinforced concrete (RC) assets and eventual reduction of its service life. The maintenance of RC
38 buildings and structures has become a major concern, and thus the development of an effective
39 maintenance plan is required. This study focuses on deterministic and probabilistic analysis of service
40 life (or repair application time) for RC assets repaired by cover rebuilding techniques in extreme
41 marine environments exposed to chloride attack. A maintenance model is developed, based on the
42 mechanism of chloride transport in repaired concrete elements, that simulates different cover
43 rebuilding methods and strategies. Computationally, the probability-based maintenance model
44 combines the Latin Hypercube technique with finite element and finite difference methods.
45 Probabilistic data for five governing random variables is considered, including diffusion coefficient,
46 surface chloride concentration, concrete cover depth, concrete replacement depth, and critical chloride
47 value for corrosion initiation. Two maintenance strategies are introduced, which can be categorized as
48 corrective and preventive repair strategies. The deterministic study shows that a deeper repair depth
49 leads to fewer repairs and larger repair intervals. The results of the probabilistic assessment reveal that
50 a preventive repair model with regularly scheduled repair intervals efficiently decreases corrosion
51 initiation risks. It was also demonstrated how shorter repair intervals and deeper depths of concrete
52 repair can achieve an acceptable probability of corrosion initiation.

53 **Key words:** chloride transport; maintenance model; probabilistic analysis; cover rebuilding; corrective
54 repair; preventive repair.

55

56

1. Introduction

57 Corrosion of steel in concrete due to chloride ingress is the primary cause of RC deterioration and
58 eventual reduction of assets' service life [1]. The high alkalinity of the cement paste surrounding the
59 reinforcing steel creates a reactively passive state; however, chloride ingress breaks down the passive
60 film on the steel bars and initiates corrosion [2,3]. The process of corrosion can be divided into two
61 main stages: the initiation period and the propagation period. Corrosion initiation occurs when chloride
62 content diffuses through concrete until reaching a threshold value at the depth of rebar cover which
63 triggers steel corrosion. The propagation period follows corrosion initiation and is defined as the
64 period from initiation to critical steel destruction. This propagation period is relatively short compared
65 to the first stage [4,5]. Therefore, the period from a structure's original condition to the onset of
66 corrosion is critical and defines the structure's service life [2].

67 High chloride environments are known to accelerate corrosion deterioration and may reduce structure
68 service life from an expected range of 50-100 years to a range of 10-30 years. Structure maintenance
69 and, when deemed necessary, repair can extend structure serviceability and maintain its safety and,
70 therefore, its lifespan [6,7]. Indeed, maintenance and repair operations should be implemented as soon
71 as corrosion begins, mostly by removing the chloride-polluted concrete and replacing it with the
72 application of cement-based repair mortars or new concrete [8]. This repair technique is called patch
73 repair if it is applied locally; it could be also generalized to all the concrete surface when the cover is
74 redone. This "cover rebuilding" is among the most common repair measures used in practice. In this
75 study, the maintenance strategy of applying periodic cover rebuilding is considered, with the use of
76 hydro demolition solution without damaging the reinforcing bars. These maintenance activities incur
77 costs and concrete waste, thus there should be balanced to ensure the long service life of structures and
78 minimize costs and environmental impacts. Therefore, developing a robust model for predicting the
79 service life of corroding RC structures, as well as effective strategies for periodic inspection and
80 repair, plays an important role in achieving the durability goal and ensuring the continued service for
81 repaired structures [6,9].

82 Recent studies have focused on the modelling of chloride transport in concrete structure elements after
83 repair, and the methods to predict repair application times during the service life of a structure. For
84 instance, Eldho et al. [10] studied the performance of the patch repair method and showed that the
85 electrochemical consequences of the method may decrease the effectiveness of patch repairs. More
86 specifically, according to Rahimi et al. [11], the diffusion mechanism of chloride ions in repaired
87 concrete is a combination of chloride penetration from the concrete surface exposed to the
88 environment and distribution of chloride ions that remain in the original and aged concrete. It was
89 further shown that the resistivities of the patch and the original concrete, which can be used as an

90 indicator of material quality, are among the most significant factors that influence the efficiency of
91 maintenance [12]. Soleimani et al. [12] recommended to choose patches with similar properties to
92 avoid mechanical incompatibility problems at the interface between the patch and the original
93 concrete. Concrete replacement depth is also a critical factor that reduces or delays the corrosion in
94 repaired concrete structures by limiting the amount of chloride content at the rebar cover [13,14].
95 Several models have been proposed to determine the time of corrosion initiation in RC structures with
96 multiple repairs throughout its service life. Both deterministic and probabilistic analysis have been
97 performed, taking into account some main performance factors such as the properties of the patch, the
98 replacement depth, or the repair time interval [13–15]. A deterministic study was implemented by
99 Song et al. [15] to predict the service life of repaired concrete structures under chloride environment.
100 Their work considered time dependency of both surface chloride concentration and diffusion
101 coefficient. However, due to the complexity of the chloride-induced corrosion mechanism in
102 reinforced concrete after one or several repairs, research on this topic with a comprehensive
103 probabilistic approach is still limited. For instance, Petcherdchoo's study [14] presented a probability-
104 based sensitivity analysis of service life (the time to repair) of chloride-attacked concrete structures,
105 which was solved by the combination of the Crank-Nicolson-based finite difference approach and the
106 Latin Hypercube sampling technique. However, this methodology did not include some factors in
107 chloride transport, such as the weather conditions (temperature and humidity), the chloride binding
108 capacity, and time-dependency of the chloride diffusion coefficient. Moreover, the cover replacement
109 depth, which is intrinsically random due to the uncertainty in repair quality control, was considered
110 deterministic.

111 Recently, Monte-Carlo Simulation (MCS) and the first-order reliability method (FORM) have been
112 applied in probabilistic assessment of the durability of RC structures [16,17]. Notably, most of the
113 studies using reliability-based analysis such as MCS, FORM, or SORM (second-order reliability
114 method) were performed on simplified models of chloride ingress using analytical solutions based on
115 Fick's laws of diffusion [18,19]. However, it is widely shown that the FORM and SORM may be
116 inaccurate for probabilistic problems with nonlinear limit state functions, while the computational cost
117 of MCS can be prohibitively high, especially when involved with time-consuming finite element (FE)
118 analysis. Compared to the crude MCS, Latin Hypercube Sampling (LHS) adequately covers of the
119 domain of variations of the random variables using a stratified sampling strategy [5].

120 This research proposes a robust probability-based model of service life prediction for corroding RC
121 structures with cover rebuilding repair. The study focuses on the assessment of corrosion initiation risk
122 due to one-dimensional chloride ingress (e.g., concrete slab) under marine environments, such as
123 atmospheric and submerged zones. First, an efficient chloride ingress model is proposed, considering
124 the diffusion process as the main governing mechanism. The constitutive nonlinear partial differential
125 equations are solved by coupling finite element and finite difference methods. The model considers the

126 influence of general conditions of weather, chloride binding capacity, decrease of chloride diffusivity
127 with concrete age, and the evolution of chloride concentration with time. A maintenance model is then
128 developed based on the mechanism of chloride transport in repaired concrete elements that simulates
129 different strategies of cover rebuilding method. The deterministic study considers the expected number
130 of repairs, depth of replacement concrete, and repair interval (time between two repairs) to evaluate the
131 effectiveness of the maintenance plan. A numerical solution developed in FreeFem++ [20] is
132 presented. Next, a probabilistic lifetime assessment of repaired RC structures is performed, including
133 both corrective and preventive repair models. The corrective strategy is that repair is carried out when
134 the chloride content at the rebar cover reaches a specific critical chloride value inducing corrosion
135 initiation. Meanwhile, the preventive approach considers the application of pre-scheduled repair
136 intervals for structures as the condition to perform a repair. Because of its efficiency, LHS is used for
137 probabilistic analysis of the maintenance model of corrosion initiation and repair timings for corroded
138 RC structures. Model development considers and assigns probabilistic values to uncertain parameters,
139 including surface chloride concentration, diffusion coefficient with time-variant nature, material
140 properties, concrete cover, critical chloride value and cover replacement depth (for repaired
141 structures). The findings of this paper are intended to serve as an introductory or basic guidelines for
142 inspection and maintenance planning to ensure long-term structural serviceability.

143 The work starts with the description of chloride ingress modeling before and after repair (Section 2).
144 After that, a deterministic example is illustrated in Section 3. Section 4 deals with the probabilistic
145 methodology used for the maintenance model, followed by an illustrated examples and some
146 discussions (Section 5).

147 **2. Chloride ingress modeling including repair**

148 ***2.1 Chloride ingress model***

149 The modeling of chloride transport into reinforced concrete is based on whether exposure conditions
150 are saturated or unsaturated. In saturated conditions, Fick's second law was used to simulate chloride
151 diffusion in concrete and considered a sufficient model [2,21]. The analytical solution for this model is
152 based on an error function, which is valid only under the assumptions that RC structures are saturated,
153 and diffusivity and surface chloride concentration are time-invariant.

154 In comparison, under unsaturated conditions, chloride transport in concrete is a complex interaction
155 between physical and chemical processes, and thus the diffusion and convection phenomena have been
156 accounted for to study this mechanism [5,22–25]. A comprehensive model of chloride ingress was
157 proposed by Bastidas-Arteaga et al. [26], considering the interaction of three phenomena: chloride
158 transport, moisture transport and heat transfer. The model solves the necessary nonlinear and partial

159 differential equations by coupling finite element and finite difference methods. Besides the influence
 160 of weather conditions, the effect of chloride binding, concrete aging, concrete mix parameters and the
 161 time-dependency of apparent diffusion coefficient as well as the chloride content of the exposed
 162 surface were considered.

163 Therefore, the chloride transport modeling could be performed under saturated or unsaturated exposure
 164 conditions. In this paper, a time dependent chloride transport model is developed with the assumptions
 165 of constant temperature, humidity, and surface chloride concentration. With these assumptions, this
 166 model can be applied for marine structures exposed to atmospheric or submerged zones, where
 167 diffusion is the main driving force to transport chloride ions from the surrounding environment into
 168 concrete and the capillary suction is not considered [27,28]. In fact, this approach allows us to develop
 169 a maintenance model with multiple cover rebuilding repairs with reduced technical difficulty and
 170 calculation time, particularly for performing probabilistic analysis.

171 Thus, according to our objective, the choice is using a simplified one-dimensional model of the
 172 diffusion system solved by coupling finite element and finite difference methods. The main governing
 173 equation for chloride transport in concrete is presented below:

$$174 \quad \frac{\partial C_{fc}}{\partial t} = D^* \left(\frac{\partial^2 C_{fc}}{\partial x^2} \right) \quad (1)$$

175 where x (m) is the distance from the exposure surface; C_{fc} (kg/m^3 of concrete) is the chloride
 176 concentration at position x at the exposure time t (s), D^* (m^2/s) represents the apparent chloride
 177 diffusion coefficients (m^2/s). To evaluate corrosion initiation risk, the chloride concentration, C_{fc}
 178 is calculated at the concrete cover depth and compare with the critical chloride value (C_{crit}). Eq. (1) is
 179 solved by combining finite or semi-infinite boundary conditions, where the boundary conditions or the
 180 flux of chlorides crossing the concrete surface J_c^s ($\text{kg}/(\text{m}^2 \cdot \text{s})$), is expressed as:

$$181 \quad J_c^s = B_c (C_{fc}^s - C_{env}) \quad (2)$$

182 where B_c (m/s) is the surface chloride transfer coefficient, C_{fc}^s (kg/m^3) is the value of chloride content
 183 at the concrete exposed surface, and C_{env} (kg/m^3) is the chloride concentration in the surrounding
 184 environment. D^* in Eq. (1) is estimated as:

$$185 \quad D^* = \frac{D_c}{1 + \left(\frac{1}{w_e} \right) \left(\frac{\partial C_{bc}}{\partial C_{fc}} \right)} \quad (3)$$

186 where w_e is the evaporable water content. $\partial C_{bc} / \partial C_{fc}$ is the binding capacity of the concrete which is
 187 driven by the binding isotherm. The Langmuir-type relationship is considered in [29–31]:

188
$$C_{bc} = \frac{\alpha C_{fc}}{1 + \beta C_{fc}} \quad (4)$$

189 where C_{fc} and C_{bc} are the free and bound chlorides, and α and β are binding constant obtained
 190 experimentally. D_c in eq. (3) is the time-dependent chloride diffusion coefficient [23,24] defined as:

191
$$D_c = D_{ref} f_1 \cdot f_2(t) \cdot f_3 \quad (5)$$

192 where D_{ref} is a diffusion coefficient determined at standard conditions. f_1 is a factor to account for
 193 temperature effects:

194
$$f_1 = \exp\left[\frac{U_c}{R} \left(\frac{1}{T_{ref}} - \frac{1}{T}\right)\right] \quad (6)$$

195 where T is the temperature inside the concrete that depends on external environmental conditions,
 196 T_{ref} is the reference temperature at which D_{ref} is determined, U_c is the activation energy of the
 197 chloride diffusion process, and R is the gas constant. $f_2(t)$ considers concrete aging:

198
$$f_2(t) = \left(\frac{t_{ref}}{t}\right)^{m_c} \quad (7)$$

199 where m_c is the aging factor and t_{ref} is the time at which D_{ref} is measured. f_3 accounts for
 200 variations in relative humidity:

201
$$f_3 = \left[1 + \frac{(1-h)^4}{(1-h_c)^4}\right]^{-1} \quad (8)$$

202 where h is the relative humidity inside the concrete that depends on external environmental conditions,
 203 and h_c is the relative humidity at which D_c drops halfway between its maximum and minimum values.
 204 Temperature (f_1) and humidity (f_3) are considered constant in this model. Previous studies [23,24,29–
 205 35] proposed deterministic data to present those parameters, such as B_c , w_e , α , β , D_{ref} , U_c , R , T_{ref} ,
 206 m_c , t_{ref} , and h_c .

207 **2.2 Chloride ingress model considering the repair**

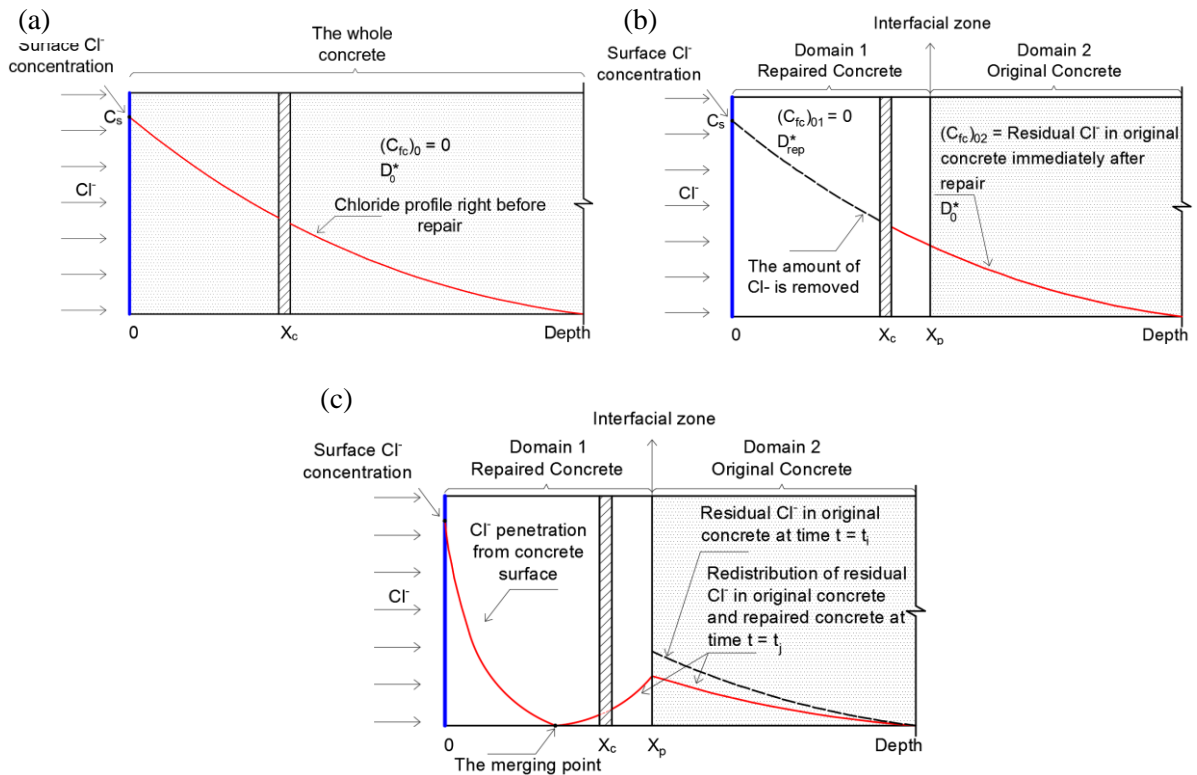
208 Different approaches for maintaining corroding RC structures have already been considered previously
 209 [6,9,36]. In this study, the focus is on cover concrete replacement. This repair involves removing the
 210 chloride-polluted concrete cover and replacing it with a new cement-based material using several
 211 techniques (with formed concrete, wet shotcrete, dry shotcrete, or manual repair).

212 Modeling chloride penetration in concrete after repair is challenging because of the interaction of the
 213 two main sources of chloride ions: chloride concentration in the surrounding environment and residual

214 chloride ions in the remaining concrete. Previous studies concluded that both macrocell corrosion
 215 (between two materials) and microcell corrosion (within a homogeneous material) may occur at the
 216 same time in repaired concrete [12,37]. This macrocell corrosion is caused by the differences in
 217 existing and new concrete causing: 1) an electrochemical potential difference, and 2) inhomogeneity of
 218 chloride concentration in the materials and then 3) the redistribution of chloride ions in the aged
 219 concrete to the new concrete after repair. These consequences were studied in existing experimental
 220 and in situ observations [37,38]. Also, both macrocell and microcell corrosions were observed in
 221 existing numerical models for the simulation of chloride transport in repaired structures [11,13,15].

222 In this paper, a detailed description of the chloride transport in repaired RC structures as well as the
 223 numerical approach considering a spatially domain-dependent and time-dependent diffusion
 224 coefficient in two material layers are illustrated in Figure 1. Figure (1a) illustrates the chloride
 225 penetration immediately before repair. At this stage, the chloride concentration abutting the concrete
 226 surface is the main source of chloride ions. The initial chloride content is assumed equal to 0, $(C_{fc})_0 =$
 227 0 for the entirety of concrete and $D^*(t) = D_0^*(t)$ (Eq. (1)) is defined as the diffusion coefficient of the
 228 original concrete.

229



232

233 **Figure 1.** (a) Chloride profile immediately before repair, (b) Chloride profile immediately after repair at $t = t_i$, (c)
 234 Chloride profile after repair at $t = t_j$ ($t_j > t_i$)

235 Figure (1b) shows the chloride profile immediately after cover rebuilding repair. At repair time t_i , the
 236 chloride-contaminated concrete cover is removed to the depth X_p . X_p is defined as repaired depth or

237 thickness of replaced concrete, which is not necessarily the same depth as concrete cover, X_c . In this
 238 case $X_p > X_c$. Consequently, the amount of chloride ions between the concrete surface and the repair
 239 depth (domain 1 in Figure (1b)) is also removed, and a chloride-free, cement-based repair material is
 240 applied as a repair layer. Only this repaired concrete (domain 1) is exposed to the surface chloride
 241 concentration in the environment and will be penetrated first before reaching the original concrete
 242 (domain 2). The main diffusion mechanism of chloride penetration in repaired material at this stage
 243 can also be modelled with Eqs. (1-8), where the boundary condition is still the flux of chlorides
 244 crossing the concrete surface, but the initial conditions in domains 1 and 2, which are the repaired
 245 concrete and the original concrete respectively, will be different. The initial chloride content is reset to
 246 zero in the repaired layer immediately after repair, $(C_{fc})_{01} = 0$ and the diffusion coefficient of the
 247 new concrete is updated as $D_{rep}^*(t)$. All parameters in Eqs. (1-8) are also updated if a new material is
 248 used to repair. The new initial chloride content in the original concrete section, $(C_{fc})_{02}$, is the residual
 249 chloride ions remaining in the aged concrete, whereas $D_0^*(t)$ and the initial model parameter are still
 250 used for modeling purposes in domain 2 (Figure (1b)).

251 After the repair, there are three principal stages of chloride distribution as shown in Figure (1c). Firstly,
 252 the chloride ions remaining in the aged concrete will be redistributed in the aged concrete layer itself.
 253 Secondly, these residual chlorides will also diffuse across the transition zone and into the new, chloride
 254 free concrete layer which provides low chloride diffusion resistance immediately following repair. Due
 255 to the difference of the diffusion coefficients in the aged concrete and the new repaired material, the
 256 problem will involve a spatially domain-dependent diffusion coefficient after repair $D_{ar}^*(x, t)$:

$$257 \quad D_{ar}^*(x, t) = \begin{cases} D_{rep}^*(t) & \text{if } 0 < x < X_p \text{ (Domain 1)} \\ D_0^*(t) & \text{if } x \geq X_p \text{ (Domain 2)} \end{cases} \quad (9)$$

258 Consequently, the Partial differential equation (PDE) used in this stage based on the Fick's diffusion
 259 law can be written as:

$$260 \quad \frac{\partial C_{fc}}{\partial x} = \frac{\partial}{\partial x} D_{ar}^*(x, t) \frac{\partial C_{fc}}{\partial x} \quad (10)$$

261 Thirdly, at time t_j (with $t_i > t_j$) the resulting diffusion of chloride ions in the original and repaired
 262 concrete is a combination of chloride ingress from both the exposed concrete surface and the chloride
 263 redistribution inside the concrete. The problem of solving the PDE occurs again and can be further
 264 complicated by multiple repairs in the same location during the RC structure service life.

265 These processes are implemented and solved for a 2-layer concrete system using the FreeFEM++
 266 software. The technique called domain decomposition for interfacial discontinuities is adopted to solve
 267 the problem and compute the residual chloride content in the remaining concrete layer immediately

268 after repair, which is an important step for calculating chloride penetration in repaired structures.
269 Specifically, the whole original concrete is split into two interconnected domains, which are the
270 repaired concrete layer and the old concrete layer. After each repair, initial conditions are set for each
271 domain, the connection between layers is established at the interface, and a new RC structure service
272 life is calculated. The efficiency of the FreeFEM++ allows developing maintenance models for several
273 different cover rebuilding method strategies. This will be presented in the deterministic example in the
274 next section.

275 **3. Illustrative example of deterministic maintenance model**

276 ***3.1 Problem description***

277 The maintenance model allows introducing several repair strategies to ensure the durability of the
278 structure. A deterministic analysis of chloride ingress in repairing concrete structures in extreme
279 marine environments by considering time-dependent variation of diffusion coefficient, chloride
280 binding, average temperature and humidity, and constant surface chloride concentration, is illustrated
281 in this section. The deterministic analysis implements the approach to modeling the stages of chloride
282 diffusion in repaired concrete described in Section 2 and provides a baseline comparison of model
283 responses to different repair strategies. First, the following assumptions are made:

- 284 • Chloride flow is a one-dimensional process (e.g., concrete slab).
- 285 • The average temperature $T = 15^{\circ}\text{C}$; the average humidity $h = 0.7$.
- 286 • The structure is exposed to an extreme marine environment. The chloride surface
287 concentration is time-invariant at the value of $C_s = 6 \text{ kg/m}^3$.
- 288 • The concrete contains 400 kg/m^3 of Ordinary Portland Cement (OPC) with 8% C3A and a
289 water/cement ratio of 0.5.
- 290 • The depth of concrete cover X_c is 50 mm.
- 291 • The service life of the structure is 70 years. Time of corrosion initiation is determined when
292 chloride content reaches the critical chloride value $C_{crit} = 2 \text{ kg/m}^3$ (0.5% of cement mass) at
293 the concrete cover depth.
- 294 • The quality and properties of repair material are the same as original concrete. Values selected
295 for the model parameters in the example are presented in Table 1.

296
297
298
299
300

301
302
303
304
305

Table 1. Values selected for the model parameters of the chloride penetration model.

Parameter	Value and reference
Binding constants in Langmuir isotherm	$\alpha = 0.1185$ [31], $\beta = 0.09$ [31]
Evaporable water content	$w_e = 0.1316$ [34]
Reference chloride diffusion coefficient	$D_{ref} = 3 \times 10^{-11}$ m ² /s [23]
Activation energy of the chloride diffusion process	$U_c = 41.8$ kJ/mol [32]
Gas constant	$R = 8.314$ J/ (mol °K)
Age reduction factor	$m_c = 0.15$ [35]
Relative humidity at which D_c drops halfway	$h_c = 0.75$ [33]
Reference temperature at which D_{ref} is determined	$T_{ref} = 296$ °K [23]
Exposure time at which D_{ref} is measured	$t_{ref} = 28$ days [23]
Surface chloride transfer coefficient	$B_c = 1$ m/s [23]

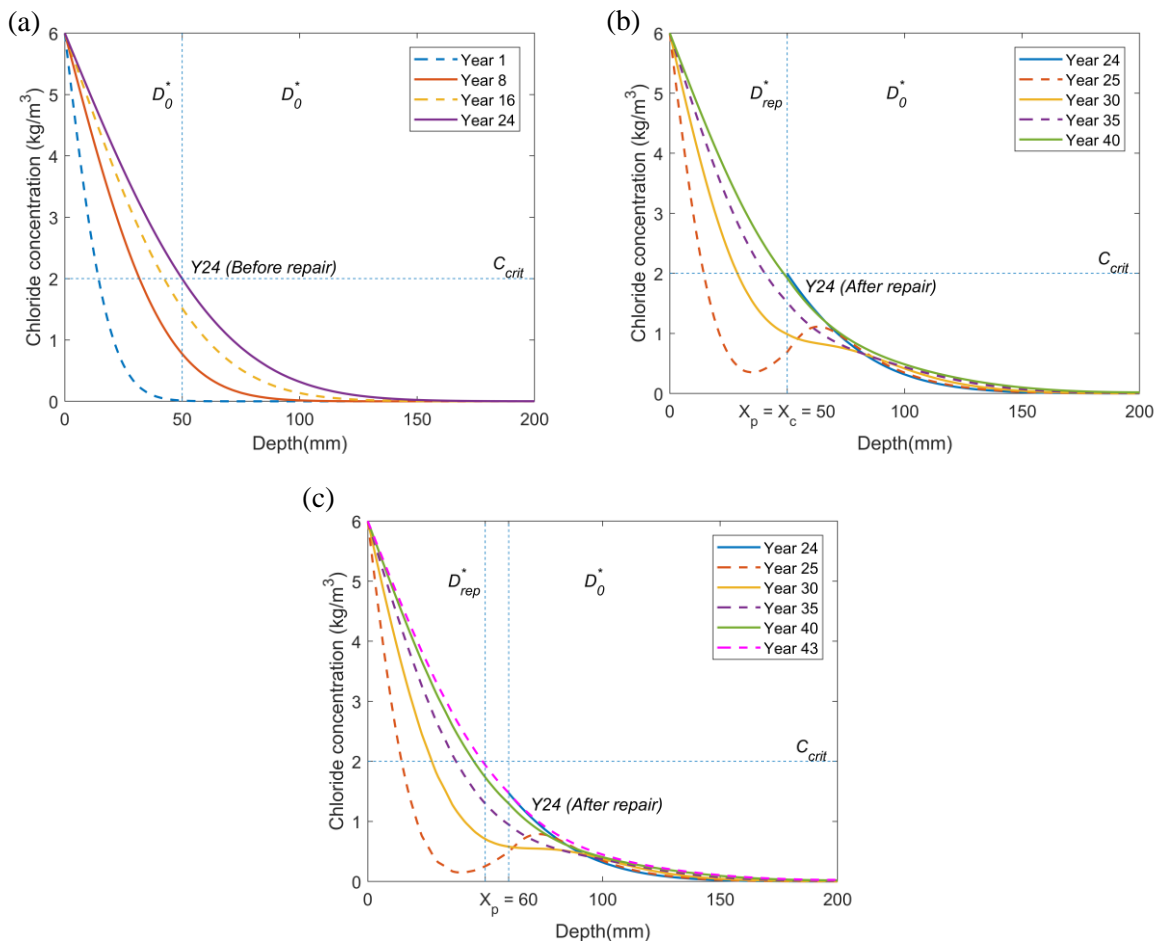
306 **3.2 Maintenance model and results**

307 Chloride penetration is now modeled considering that concrete repair occurs during service life of the
308 structure. Chloride concentration is again assessed at the cover depth. A maintenance plan may
309 consider several repair factors including expected number of repair cycles, concrete replacement depth
310 and repair time interval. Two approaches for maintaining concrete structures subjected to chloride
311 ingress are suggested, including the corrective repair strategy and scheduled repair interval strategy.

312 **3.2.1 Corrective repair strategy**

313 The corrective repair strategy removes a chloride-contaminated concrete layer each time the chloride
314 concentration at the cover thickness reaches a critical value (C_{crit}) and replaces it with a similar
315 material. In the following example, this maintenance model is used to calculate these time intervals
316 between repairs following the corrective repair strategy. The calculation further demonstrates how the
317 depth of each repair affects the calculated repair intervals. Two concrete repair depths (X_{p1}, X_{p2}) are
318 chosen; the first repair depth, X_{p1} , is equal to the existing cover depth ($X_{p1} = X_c = 50$ mm), and the
319 second repair depth, X_{p2} , is 60 mm ($X_{p2} = 60$ mm), which is deeper than the existing concrete cover
320 layer. The replacement material is assumed to have an initial chloride concentration of 0 kg/m³ at the
321 time of each repair. Although the repaired concrete is assumed to be the same quality as the old
322 concrete, the diffusion coefficient in each concrete layer is different due to the aging factor $f_2(t)$. The

323 diffusion coefficient in the original concrete, D_0^* , decreases as concrete aging, while D_{rep}^* assumes its
 324 initial value when the material is put into service. Fig. s (2a, 2b, and 2c) illustrate the chloride
 325 diffusion versus depth in a concrete structure before and after repair with X_{p1} and X_{p2} .
 326 Figure 2a shows the progress of chloride penetration until C_{crit} is reached in year 24. Concrete repair
 327 is assumed to occur in that year. In Figures 2b and 2c, the residual chloride concentration after repair is
 328 shown in year 24 (after repair) curve and the progress of concrete penetration after repair is shown
 329 from year 25-40. In year 25, chloride penetration in the new concrete has two sources: the concrete
 330 surface exposed to the environment and, after repair, the residual chloride in the aged concrete layer.
 331 After repair, residual chloride in the aged concrete redistributes between the abutting layers until
 332 reaching an equilibrium between years 30-35. In the following years, only environmental chloride
 333 concentration contributes to chloride penetration.



334
335

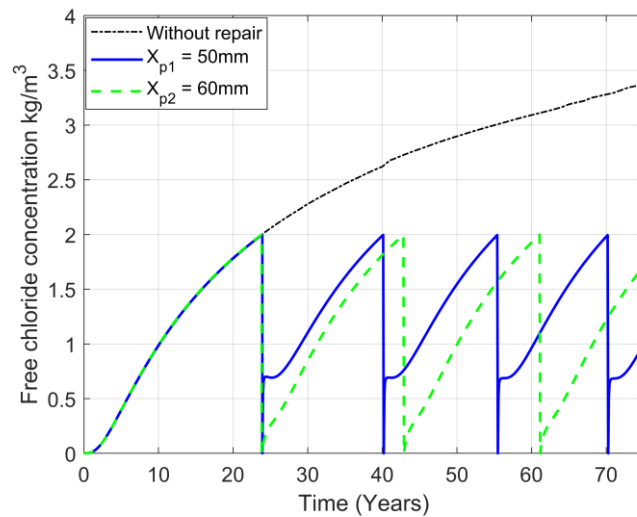
336
337

Figure 2. Chloride profiles: (a) before repair, (b) after repair with X_{p1} , (c) after repair with X_{p2}

338 These simulation results are physically compatible with the mechanism of chloride transport in
 339 repaired concrete structures analyzed in Section 2.2, including chloride diffusion from two sources in
 340 the after-repair condition. The example demonstrates that the application of cover rebuilding reduces
 341 the critical chloride concentration at the reinforcement depth and thereby delay corrosion initiating
 342 from concentration above C_{crit} .

343 The two repair depths have similar chloride progressions after repair, but chloride concentrations after
 344 the deeper concrete repair, ($X_{p2} = 60\text{ mm}$), are consistently lower than those with the shallower
 345 concrete repair, ($X_{p1} = 50\text{ mm}$). That is because the deeper replacement depth slows the chloride
 346 redistribution from the aged concrete to the critical depth due to lesser chloride ions remaining in the
 347 old concrete and longer diffusion distance. Based on the results, a second repair would be
 348 recommended around year 40 for the shallower repair and year 43 for a deeper repair. These second
 349 repairs and additional repairs each time C_{crit} is reached were also simulated to observe the repair
 350 schedule under the corrective repair strategy with different repair depths. The results for 70 years are
 351 shown in Figure 3.

352 It is worth noting the difference in chloride redistribution after concrete repair between depths
 353 X_{p1} and X_{p2} . The redistribution for X_{p1} is more rapid immediately after repair compared to that for
 354 X_{p2} , causing a sudden increase in chloride concentration in the new concrete at critical depth, after
 355 which point redistribution occurs more gradually.



356 **Figure 3.** Time-dependent evolution chloride concentration at the cover depth without and with repair
 357

358 The repair application time and the number of repairs of X_{p1} and X_{p2} for the considered structural
 359 lifetime are presented in Table 2. While the repair strategy of X_{p1} is triggered four times in 70 years to
 360 mitigate the risk of corrosion initiation, the concrete structure with X_{p2} requires only three repairs in
 361 its lifespan. It is observed that a deeper repair depth leads to a fewer number of repairs and so that a
 362 60mm repair depth may be preferred from certain economic and environmental perspectives.
 363 Conversely, the cost and the amount of new concrete placed for a single repair at a deeper depth is
 364 higher. Economic and environmental comparison of the repair strategies is beyond the scope of this
 365 paper.

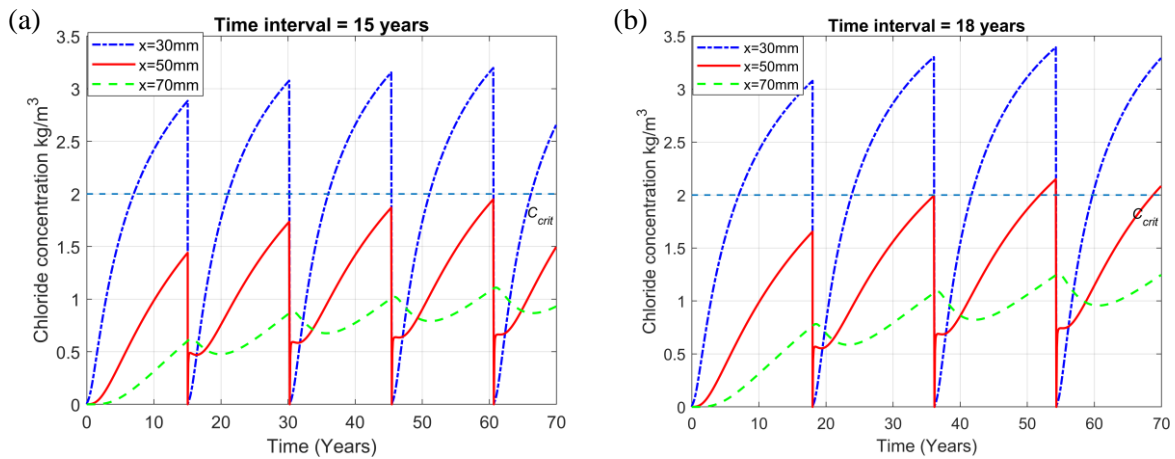
366 **Table 2. Number of repairs for two different repair depths**

Repair depth	Repair application time (year)	Number of
--------------	--------------------------------	-----------

	First	Second	Third	Fourth	Fifth	repairs
50mm	23.9	40	55.2	69.8	>70	4
60mm	23.9	42.8	60.8	>70		3

367 **3.2.2 Scheduled repair interval strategy.**

368 The second approach for a maintenance model schedules structure repairs at regular intervals. Repairs
 369 are therefore carried out at pre-scheduled times. It is necessary to evaluate by modeling, testing, or
 370 expert advice the risk of corrosion occurring at those times. The first repair strategy considered in this
 371 section is the removal and replacement of $X_p = 50$ mm of surface concrete at 15- and 18-year intervals,
 372 as shown in Figures 4a, b.



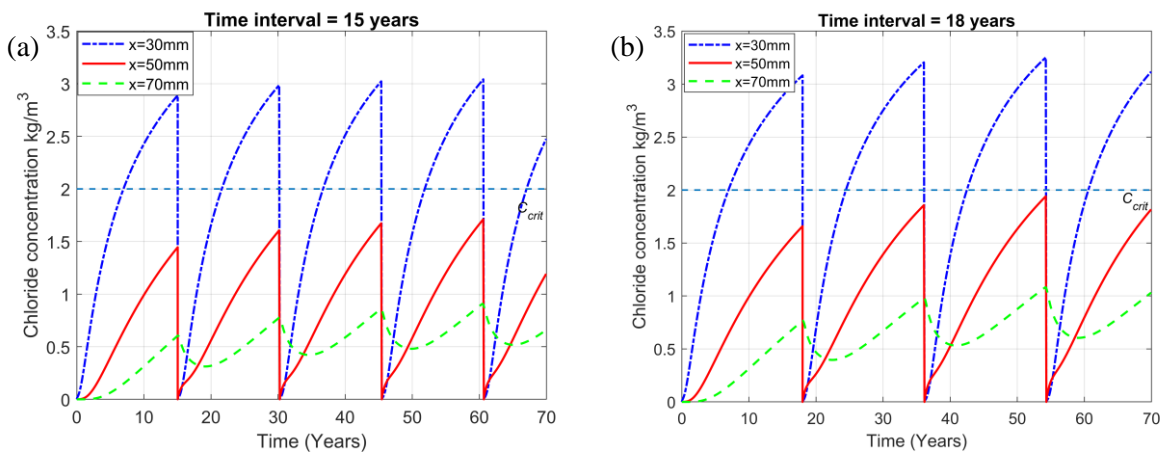
373
 374 **Figure 4.** Time-dependent evolution of the chloride concentration at three depths ($x = 30, 50$ and 70 mm) with
 375 $X_p = 50$ mm and (a) 15-year repair interval, or (b) 18-year repair interval

376 All cases present the evolution of the chloride content at different depths in a RC structure, including
 377 at $x = 30$ mm, 50 mm, and 70 mm. At the position $x = 30$ mm, the chloride concentration increases
 378 rapidly because it is closer to the concrete surface than the rest of the survey points. Immediately after
 379 repair, the entire cover layer of concrete with the chloride contamination will be removed, thus
 380 obviously the chloride content at the 30 mm and 50 mm depths will decrease to zero, and then rapidly
 381 increase again when x decreases. The chloride level at the 70 mm depth falls quickly after repair due to
 382 the new concrete layer's chloride absorption, but not to zero level, and then progressively increases
 383 again. It indicates that the existing chloride content in the unrepaired concrete will flow in two
 384 directions: away and toward the concrete surfaces.

385 To assess the usability of the repair strategy, the chloride concentration is assessed at the cover depth
 386 of 50 mm and compared with a threshold chloride content of 2 kg/m^3 (as seen on the profile $x = 50$ mm
 387 in Figures 4a, b). It is observed that the chloride concentration by the repair strategy of the 50 mm
 388 replacement depth and the 15-year maintenance interval satisfy the requirement of reducing corrosion

389 initiation risks for multiple repairs (Figure 4a). Meanwhile, with $X_p = 50\text{mm}$ but the 18-year
 390 maintenance interval, this repair plan is not suitable in preventing the risk of corrosion initiation
 391 because the chloride concentration exceeds the critical value before the third repair action is
 392 implemented (see Figure 4b), which may affect the durability issue in RC structures.

393 Next, a deeper repair depth, $X_p = 60\text{ mm}$, is considered, with repair intervals of 15 and 18 years.
 394 Figures 5a and 5b show that when considering the critical chloride concentration of 2 kg/m^3 at the
 395 cover depth (as seen on the profile $x = 50\text{ mm}$ in Figures 5a, b), both repair intervals are appropriate
 396 for corrosive RC structures with multiple repairs. In which case the less frequent repair interval of 18
 397 years may be preferred from an economic aspect and environmental point of view as it has less
 398 expected number of repairs. Hence, a comprehensive maintenance strategy should evaluate both repair
 399 intervals and replacement depths, balancing their costs, environmental impacts, and other factors, to
 400 achieve an optimal strategy of protecting structures from corrosion. The deterministic example
 401 highlights how different repair depths affect the maintenance schedule, however the impact of
 402 concrete cover should not be underestimated because of its contribution in extending the time of
 403 corrosion initiation [39].



404
 405 **Figure 5.** Time-dependent evolution of the chloride concentration at three depths ($x = 30, 50$ and 70 mm) with
 406 $X_p = 60\text{ mm}$ and (a) 15-year repair interval, or (b) 18-year repair interval

407 **4. Probability-based maintenance model for repairing concrete** 408 **structures**

409 The chosen data about material properties, environmental conditions, cover depth, replacement depth
 410 and other model parameters used in the deterministic maintenance model allows us to conveniently
 411 optimize the inspection and repair plan during the structure lifespan. However, this is not sufficient
 412 from an engineering point of view because some of the aforementioned parameters are random.
 413 Therefore, a probabilistic analysis should be performed to obtain a comprehensive estimation of

414 chloride penetration in repairing concrete structures. This section aims to determine appropriate
415 strategies in establishing maintenance plans to prolong the structure service life when considering
416 uncertainties.

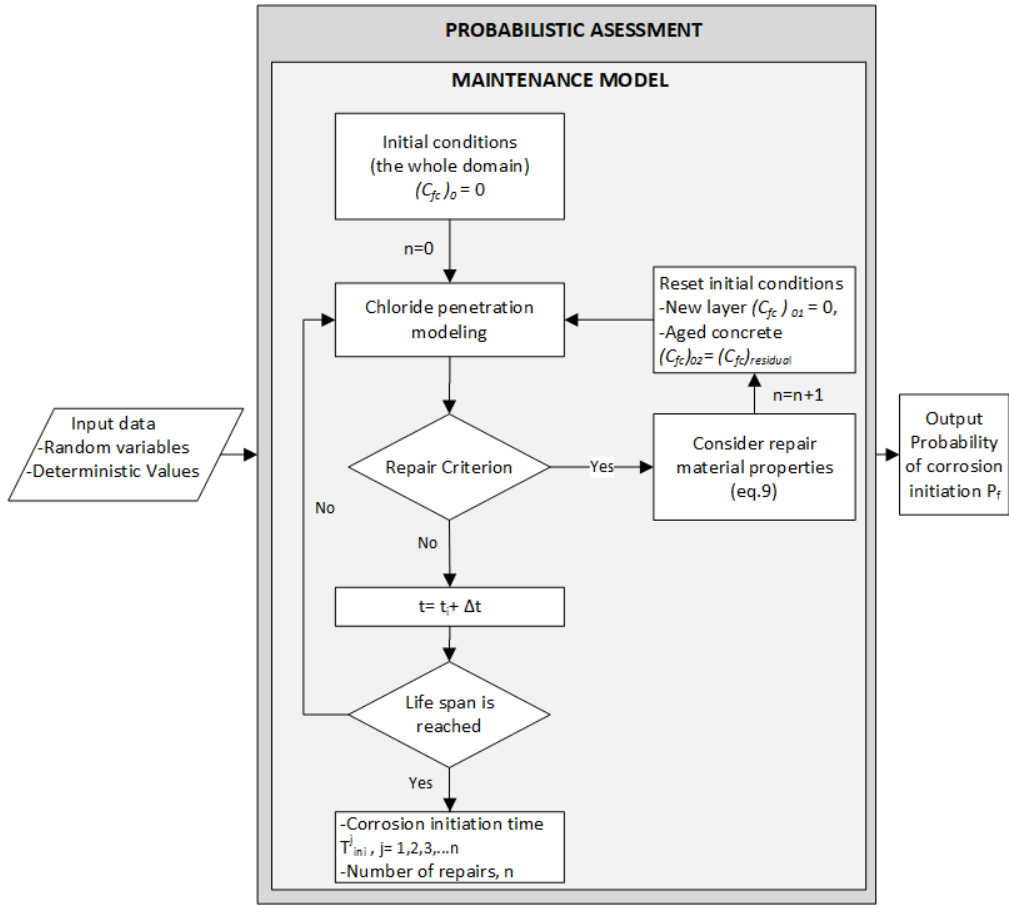
417 ***4.1 Methodology for probabilistic analysis***

418 Among the conventional methods used for probabilistic assessment, Monte Carlo Simulation (MCS) is
419 commonly used due to its simple computer implementation [16,17,40]. However, for each random
420 input variable in a model, this technique generates a random sample of N points, thus it is
421 computationally expensive when dealing with uncertainty problems. The use of MCS is even more
422 time-consuming when complex numerical models are involved, particularly in the iterative
423 computation for predicting the time-dependent chloride transport in repairing RC structures with two
424 different material layers. Hence, another sampling method called Latin Hypercube Sampling (LHS)
425 was developed to overcome those constraints. This sampling strategy disperses the sample points more
426 uniformly among all possible values, thus reducing the number of samplings generated with
427 satisfactory result. As a result, LHS ensures that the entire range of each random variable is completely
428 covered. In a wide range of scenarios, LHS has been demonstrated to be more efficient than basic
429 random sampling. Because of its popularity and stability, LHS has been frequently employed in the
430 design of experiments and probabilistic analysis for evaluating the probability of failure [41,42].
431 Therefore, LHS is used in this paper to carry out probabilistic analysis in maintenance modeling.

432 The computational strategy for the probability-based maintenance model will combine LHS with finite
433 element and finite difference approaches. Figure 6 depicts the proposed approach with a flowchart. It
434 is made up of two main loops. The external loop illustrates the probabilistic strategy of utilizing LHS
435 to generate the required number of input random variables to estimate the probability of corrosion
436 initiation; while the internal loop represents the numerical code in the FreeFEM++ used to model
437 chloride transport.

438 In the internal loop, the “repair criterion” is set command based on two main approaches, as presented
439 in the deterministic example. The first approach is that repair is carried out when the chloride content
440 at the rebar cover reaches a specific critical chloride value. Meanwhile, the second approach considers
441 the application of pre-scheduled repair intervals for structures as the condition to perform a repair.
442 These approaches allow us to compute the probability of corrosion initiation, which is one of the main
443 criteria in reliability assessment for corroding RC structures. The calculation flowchart for chloride
444 ingress after repair with time-dependent chloride diffusion is also presented. As seen from Figure 6,
445 when the repair criterion is satisfied, the repair material properties will be considered, and initial
446 conditions are set for each domain (new material layer and original, aged concrete) that take into
447 account a spatially domain-dependent and time-dependent diffusion coefficients of two concrete

448 layers, as mentioned in section 2.2. Accordingly, a new RC structure service life is predicted, this
 449 process is reiterated until the structure's life span is reached, then the expected number of repairs, n , is
 450 recorded.



451

452

Figure 6. Flow chart of probabilistic maintenance modeling

453 4.2. Probability of corrosion initiation

454 The time of corrosion initiation, T_{ini} , is defined as the instant at which the chloride concentration at the
 455 cover thickness reaches a critical level, C_{crit} . The chloride concentration at which the passive layer on
 456 steel bars is broken down, leading to the start of steel's corrosion, is represented by this critical value.
 457 C_{crit} in this paper is assumed to be a random variable because of the uncertainty of concrete material
 458 properties. For RC structures repaired by cover rebuilding technique, the corrosion initiation time may
 459 occur on several occasions depending on the exposure conditions, lifetime assessment and number of
 460 repairs. It is determined by evaluating the time-dependent chloride concentration at the cover depth
 461 using the solutions of nonlinear partial differential equations presented in section 2. The cumulative
 462 distribution function (CDF) of time to corrosion initiation, $F_{T_{ini}}(t)$, is defined as:

463

$$F_{T_{ini}}(t) = \text{Prob} \{T_{ini} \leq t\} = \int_{T_{ini} \leq t} f(\underline{x}) d\underline{x} \quad (11)$$

464 where \underline{x} is the vector of the random variables considered and $f(\underline{x})$ is the joint probability density
 465 function of \underline{x} . The limit state function becomes:

$$466 \quad g(\underline{x}, t) = C_{fc}(\underline{x}, t, X_c) - C_{crit}(\underline{x}) \quad (12)$$

467 where $C_{fc}(\underline{x}, t, X_c)$ is the chloride concentration at the cover depth and the time t , X_c is the concrete
 468 cover thickness. The probability of failure, p_f , is transformed into the probability of corrosion
 469 initiation in this case, and is calculated by integrating the joint probability function on the failure
 470 domain [43]:

$$471 \quad p_f = \int_{g(\underline{x}, t) \leq 0} f(\underline{x}) d\underline{x} \quad (13)$$

472 **4.3. Probabilistic data**

473 In the probabilistic assessment of this problem, there are two main sources of uncertainty to be
 474 considered, one related to the chloride diffusivity in concrete and the other related to assessing criteria.
 475 The probabilistic models of all considered random variables are shown in Table 3. In the first group
 476 are considered the surface chloride concentration and the concrete diffusivity parameters (D_{ref} , U_c ,
 477 m_c) for original and repair materials. The surface chloride, C_s , which represents the degree of chloride
 478 attack, is uncertain due to the randomness of chloride content in surrounding environment. The
 479 uncertainty in concrete diffusivity parameters, which represent the chloride resistance of concrete
 480 material, exists due to different or random properties of concrete material properties (the w/c ratio, the
 481 concrete mix proportions, the type of cement, the type of curing). Although concrete structures have
 482 the same mix design, their material properties may differ due to different manufacturing processes,
 483 quality control and environments. For the second source of uncertainties, cover depth, replacement
 484 depth and critical chloride value are the three random variables considered herein. The variations in
 485 the cover depth and replacement depth are attributed to construction/repair methods and labor during
 486 execution. The uncertainty in the critical chloride value is due to its sensitivity to chemical properties
 487 of concrete components such as sand, gravel, and cement.

488 Previous studies [11,14] have shown that cover rebuilding technique benefits from a replacement
 489 depth below cover depth. This is also implied according to the deterministic analysis in section 3.
 490 Therefore, in the probabilistic assessment, the replacement depth, X_p , which is considered as a random
 491 variable, is established to be deeper than the cover thickness. To solve this issue, X_r is appeared as the
 492 extra replacement depth behind the rebar cover. Depending on construction methods and quality
 493 control during execution, X_r would be variant and follows a normal distribution with a mean value of
 494 10mm and a coefficient of variation $[COV]_{X_r} = 0.1$. The total replacement depth X_p could be presented
 495 as $X_p = X_c + X_r$.

496 **Table 3: Probabilistic models of random variables**

Variable	Mean	COV	Distribution	References
D_{ref}	$3 \times 10^{-11} \text{ m}^2/\text{s}$	0.20	Log-normal	[23], [44]
U_c	41.8 kJ/mol	0.10	Beta on [32,44.6]	[32]
m_c	0.15	0.30	Beta on [0,1]	[35]
C_{crit}	0.5 wt% cement	0.20	Normal	[45]
C_s	7.35 kg/m^3	0.7	Log-normal	[46]
X_c	40 mm	0.25	Normal (truncated at 10 mm)	[45,47]
X_r	10 mm	0.1	Normal (truncated at 0 mm)	

497 **5. Illustrated example of probabilistic assessment**

498 **5.1 Problem description and basic assumptions**

499 The main goal for this example is a comprehensive assessment of the probability of corrosion
500 initiation for repairing concrete structures by considering the uncertainties described in previous
501 section. Two main approaches are adopted to determine repair conditions and application times, which
502 are named “corrective” and “preventive” repair models. The intention of this section is to explore the
503 results obtained from the probabilistic analysis so that a reasonable maintenance plan can be achieved.
504 The problem description and necessary assumptions are similar with the deterministic example. For
505 the probabilistic study, samples for each random variable are generated based on the Latin Hypercube
506 sampling technique, and the random variables are assumed to be statistically independent. This
507 scenario would overestimate the deterioration process and probability of initiating corrosion. However,
508 actual data is needed to assess correlations between the parameters listed in Table 3 and therefore
509 improve the performance of the maintenance model. The remaining constant parameters used in the
510 maintenance model are presented in Table 1. For the sake of simplicity, it is assumed that the same
511 material is used for construction and repair in all cases.

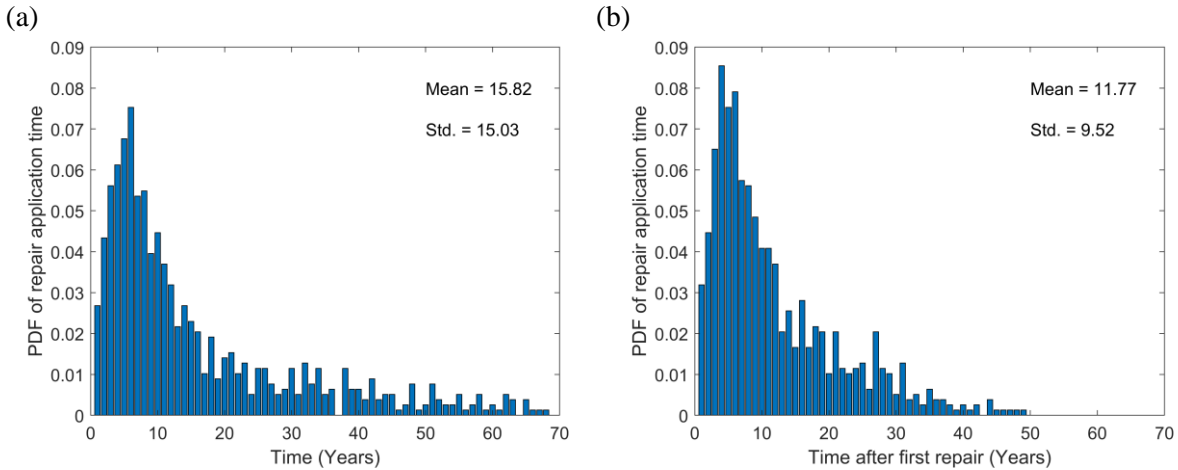
512 **5.2 Corrective Repair Model – Results and Discussion**

513 The definition of the corrective repair approach was described in the deterministic example, section
514 3.2.1. The chosen repair strategy herein is to apply a replacement layer approximately 10 mm deeper
515 than the cover thickness. And the probabilistic analysis considers of the uncertainty of random
516 variables.

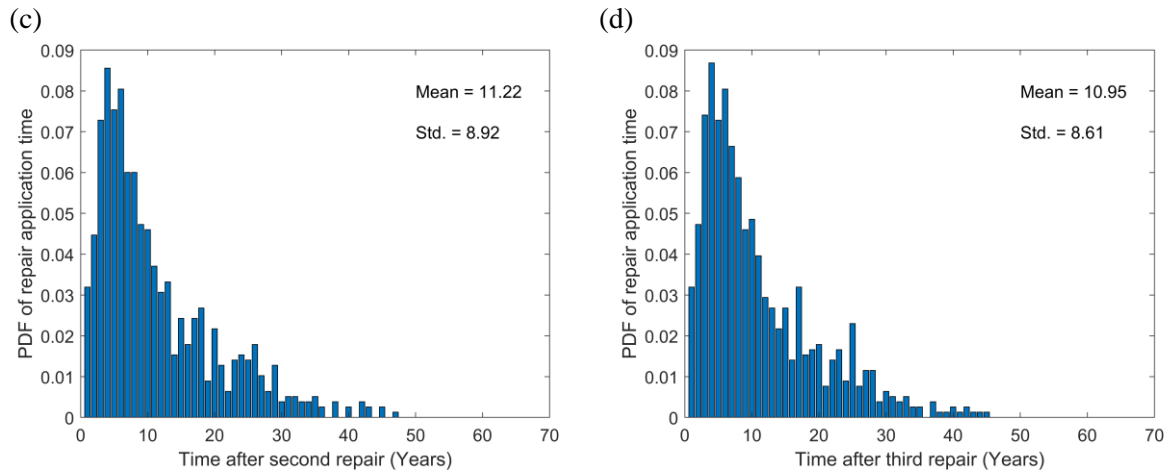
517 The probabilistic times to repair (or the probabilistic time to corrosion initiation) can be represented in
518 terms of the probability density function (PDF) of repair application time. From the PDF in Figure 8a,
519 the mean and the standard deviation of the first application time is calculated as equal to 15.82 and
520 15.03 years, respectively. The means of the three following repair times are 11.77, 11.22, and 10.95

521 years, respectively. To be specific, it takes an average 11.77 years from the first repair application time
 522 to trigger the second repair. Similarly, it takes an average 11.22 years from the second repair to trigger
 523 the third repair and so on. It is observed that the time for corrosion initiation at the early stage (before
 524 any repair measure is applied) is longer than that in the remaining operating period within a 70-year
 525 service life. Except the first repair application time of about 16 years, the repair intervals in following
 526 years are all approximately 11 years. The corrective repair model does not demonstrate significant
 527 variation chloride diffusion in RC structures between repair periods. The results are in agreement with
 528 findings from a previous probabilistic study [14]. This observation could be useful in developing a
 529 maintenance plan for RC structures in marine environments at an early stage.

530 Considering the PDFs in Figure 8, the cumulative density function (CDF) of repair application time
 531 during a 70-year service life can be computed to provide the probability of maintenance activity at any
 532 year (Figure 9). For example, if the time to repair is defined at the year when the probability of
 533 corrosion initiation reaches 50% and it is required to carry out the cover replacement measure, the first
 534 repair will be performed at year 15.82, the subsequent repairs will be implemented at years 27.59,
 535 38.82, 49.77, and 60.72. However, if 40% probability is chosen to trigger a repair, the first repair time
 536 is in year 10.7, and this period will be even shorter, year 8, with 30% probability for the first corrosion
 537 occur. Therefore, the outline of maintenance planning based on probabilistic repairing times should
 538 consider the cost constraints, environmental impacts as well as the expert judgment of the quality of
 539 structures. In addition, an early inspection on site to evaluate the degree of corrosion is highly
 540 recommended to support a decision-making process.

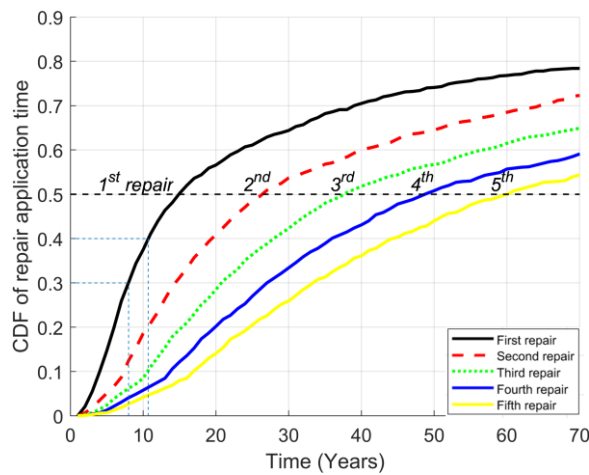


541



542

543 **Figure 8.** PDF of repair application time (a) First repair application (b) Second repair application (c) Third repair
 544 application (d) Fourth repair application



545

546

Figure 9. CDF of repair application time

547 Furthermore, the calculation of the total expected number of repairs is introduced. It is assumed that if
 548 the cumulative period of repairs exceeds the service life of 70 years, the maintenance could be
 549 stopped, hence the expected number of repairs would be determined. As shown in Table 4, with the
 550 repair strategy of $X_p = 60$ mm replacement depth, averagely five repairs are estimated during the
 551 structure's lifetime. Additional repair strategies could also have been explored in this study such as
 552 applying different replacement depths or using better-quality materials. For instance, recent studies
 553 [14,17] proved that self-compacting concrete is more sensitive to the values of the diffusion
 554 coefficient, which can be used as an indicator of material quality.

555

Table 4. The expected number of repairs applications

Criteria	Repair application time						Expected number of repairs
	First	Second	Third	Fourth	Fifth	Sixth	
Mean	15.82	27.59	38.82	49.77	60	>70	5

Standard deviation	15.03	24.38	33.22	41.77	50.35		
--------------------	-------	-------	-------	-------	-------	--	--

556 The maintenance planning based on the corrective repair model could be used as a basic result for the
557 preventive repair model that is proposed and explained in the next section.

558 **5.3 Preventive Repair Model – Results and Discussion**

559 This probabilistic model concerns the maintenance policy of applying a scheduled repair interval at a
560 given repair depth for RC structures as the condition to perform a repair. For instance, the repair
561 application will be implemented immediately whenever the structure lifetime reaches a specific time of
562 repair. Within a 70-year service life, the repair intervals (the time between two repairs) suggested in
563 this example are 15, 10, 10, 10, 10, and 10 years, respectively (i.e. in years 15, 25, 35, 45, 55, and 65).
564 Again, the chosen repair strategy is to apply a replacement layer approximately 10 mm deeper than the
565 cover thickness. These chosen intervals are not randomly determined, but they are based on the results
566 obtained from the corrective repair model, that the scheduled repair times are determined to be less
567 than the mean values of repair application times. The mean repair time value could be approximated as
568 the time to reach a probability of 0.5 in Figure 9. Since this approximation should reduce the risks of
569 corrosion initiation, this probabilistic model is referred as a preventive repair model. Other threshold
570 probabilities could be selected depending on the risks allowed for the end user. For instance, small
571 probabilities are related to reinforced concrete assets with higher consequences in case of failure and
572 vice-versa. The selection of the repair depth will also influence the probability of corrosion initiation
573 after the first repair. The effects of considering different repair schedules and depths for preventive
574 repair will be illustrated and discussed in this section.

575 Figure 10 presents the simulated time-dependent chloride concentration profiles at the critical depth. It
576 is noted that the cover depth and the replacement concrete depth are considered random. Thus, for
577 each sample, the chloride content is captured at varying cover depths and is compared with the random
578 value of threshold chloride concentration for corrosion initiation. Eqs. (11) to (13) are applied in this
579 case to compute the probability of corrosion initiation as shown in Figure 11. The chloride profiles of
580 all samples as well as the mean profile, the lower and upper bounds of chloride content are presented
581 in Figure 10. It is observed that at the years of repair (i.e. year 15, 25, 35, 45, 55, and 65) the mean
582 chloride content is higher than the mean value of the chloride limit to start corrosion (2 kg/m^3).
583 However, Figure 11 shows that the probability of corrosion occurring would be 51.9% only at the first
584 repair time (year 15), while the probabilities are below 50% for all remaining repair periods. Hence,
585 the assessment of corrosion risk and the reliability estimation for a maintenance plan based on mean
586 values can be misleading in this case.

587
588

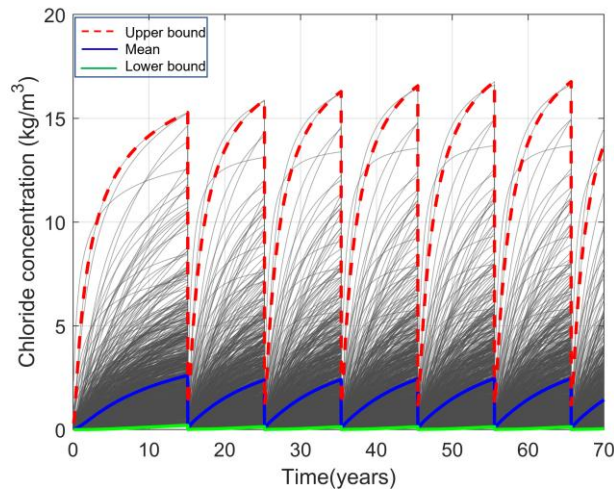


Figure 10: Time-dependent chloride concentration profiles at rebar cover

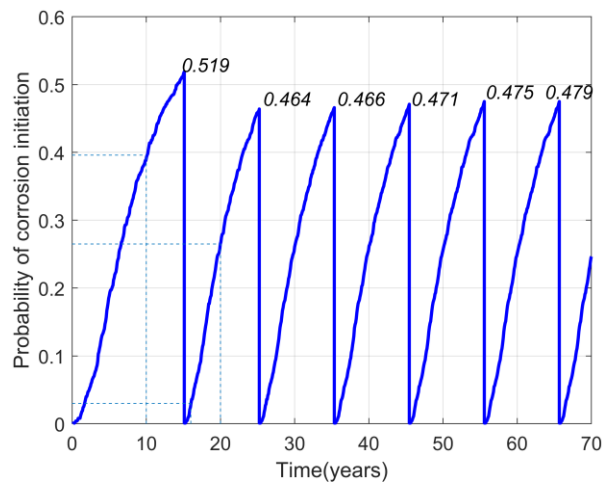


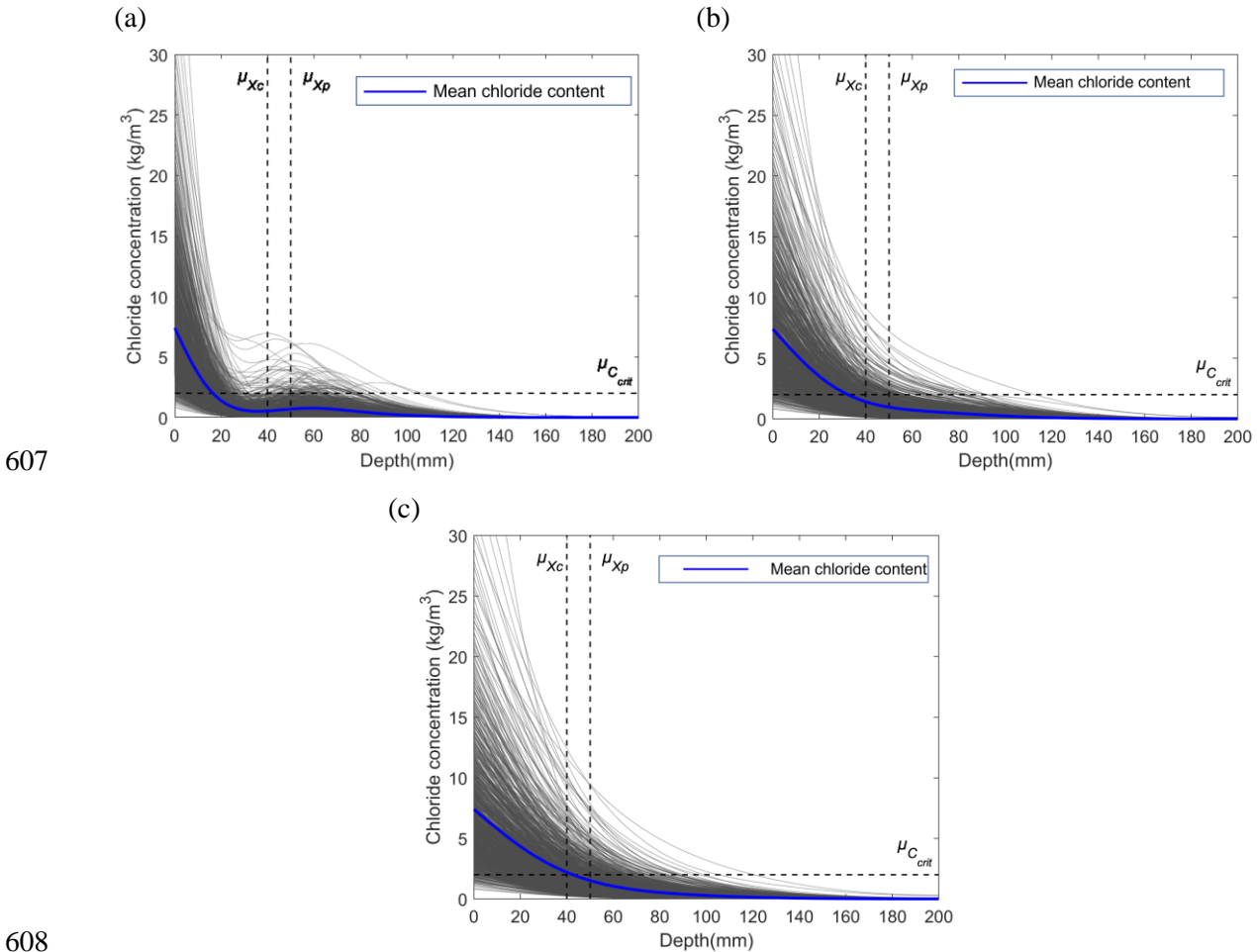
Figure 11: Probability of corrosion initiation

589
590
591

592 In Figure 11, the probability of corrosion initiation at any year can be recorded. For example, in year
593 10 (5 years before the first repair), there is a 39.6% possibility of corrosion occurring, while its
594 probabilities in year 16 (1 year after the first repair), year 20 (5 year after the first repair), and year 25
595 (10 year after the first repair) are 3%, 26.5% and 46.4% respectively. It can be concluded that the
596 further away from the most recent repair, the higher the probability of corrosion.

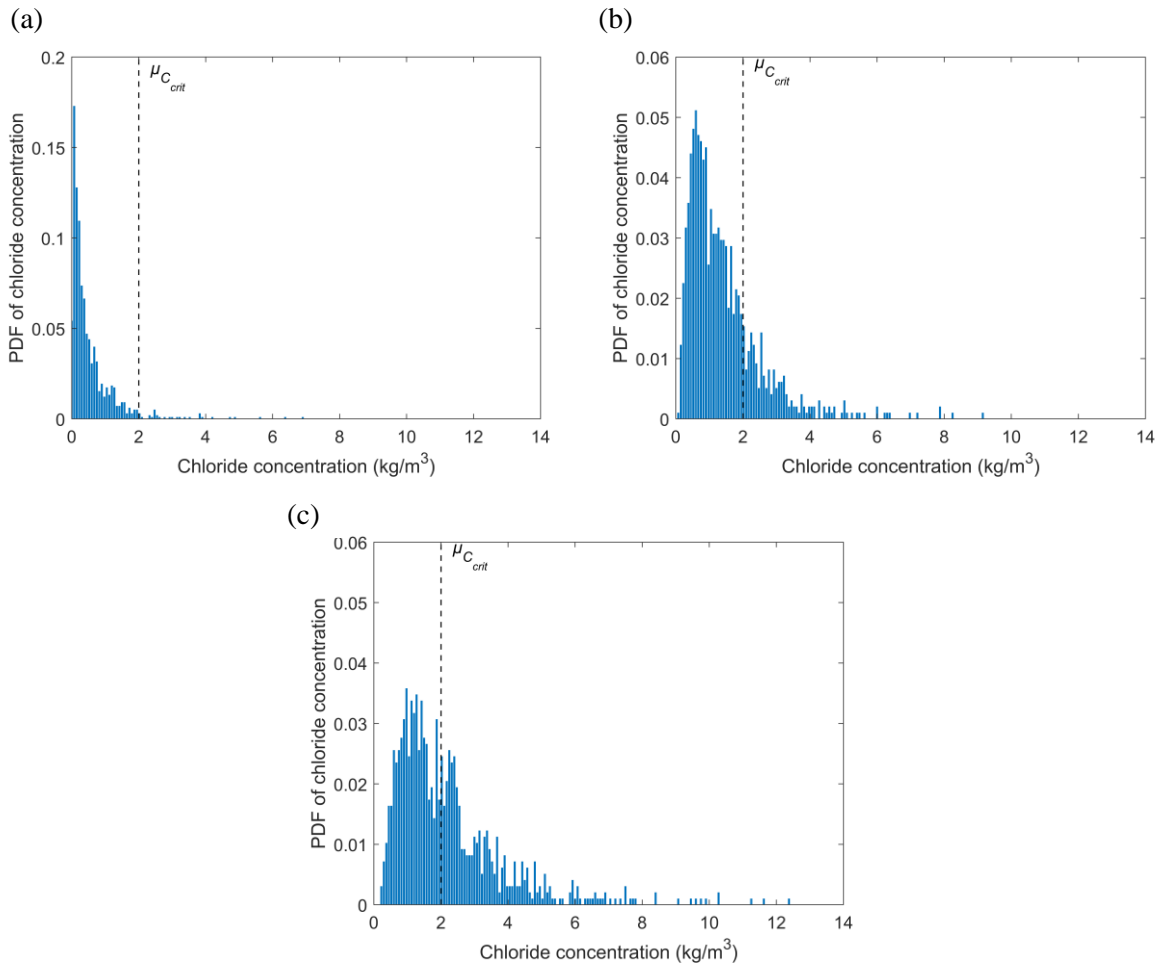
597 Figure 12 shows the chloride profiles at (a) one year after the first repair, (b) 5 years after the first
598 repair, and (c) 10 years after the first repair. In Figure 12a, the chloride content in year 16 (1 year after
599 the first repair) is illustrated by curves that fluctuate in value around the interfacial position between
600 two concrete layers, or at $x = X_p$. This is due to the redistribution of chloride ions existing in the
601 original concrete to the new concrete layer after the first repair. Moreover, at year 16, the mean of
602 chloride content at the position of rebar cover is much lower than the mean of critical value (2 kg/m^3),
603 due to the effective chloride absorption of new concrete layer in the early years after repair. For the
604 rest of survey years, the decrease of chloride concentration versus depth tend to remain (Figures 12b,

605 c). It is noticed that in year 25 (10 years after the first repair), the mean of chloride content at the
 606 critical depth is over the average critical value, it is when the second repair should be scheduled.



609 **Figure 12:** Chloride profiles (a) one year after the first repair (b) 5 years after the first repair (c) 10 years after
 610 the first repair

611 From Figure 12, the PDF of chloride content at the cover depth can be determined, as shown in Figure
 612 13a, b and c. The mean line of critical chloride value (2kg/m^3) is considered as the landmark value for
 613 comparison. It is observed that the year with more PDF values being to the right of the threshold value
 614 will have a higher probability of corrosion initiation because it represents that the chloride content
 615 exceeds the limit state. Accordingly in a 10-year repair cycle from year 15 to year 25, the probability
 616 of starting corrosion at year 16 (1 year after the first repair) is lowest (Figure 13a), followed by year 20
 617 (5 year after the first repair) (Figure 13b) and 25 (10 year after the first repair) (Figure 13c). This is
 618 consistent and compatible with Figure 11.



619

620

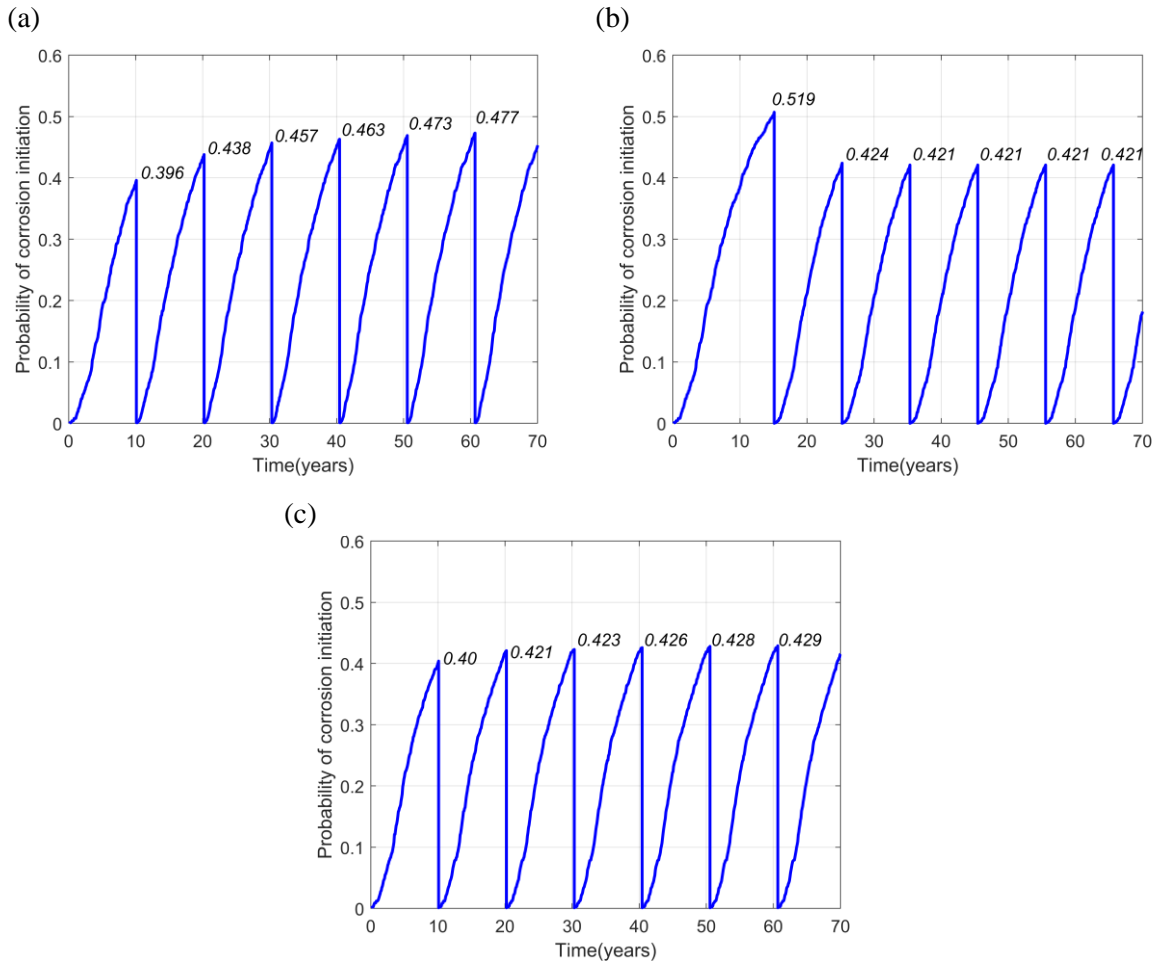
621
622

Figure 13: PDF of chloride content at cover depth (a) one year after the first repair (b) 5 years after the first repair (c) 10 years after the first repair

623
624
625
626
627
628
629
630
631
632
633
634
635
636
637

To reduce the probability of corrosion initiation, some maintenance strategies are introduced, such as shortening repair application times or applying a deeper replacement concrete or using both strategies simultaneously. Figure 14 shows the probability of corrosion initiation when applying different preventive repair strategies, including (a) 10-year repair intervals, (b) 20mm extra replacement depth, and (c) 10-year repair intervals and 20mm extra replacement depth. In Figure 14a, there is only about 40% possibility of initiating corrosion at the first repair time if the repair is carried out every ten years. The probabilities of corrosion initiation at the following repair times are slightly reduced compared to the data in Figure 11. This strategy requires six cover rebuilding repair applications. Considering both the reliability criteria and the number of repairs in a 70-year service life, the strategy of 10-year repair intervals appears to be more effective than the previous preventive strategy. Meanwhile, for the case in Figure 14b with the original repair intervals but deeper extra concrete patch of 20mm ($\mu_{Xr} = 20 \text{ mm}$, $[\text{COV}]_{Xr} = 0.1$), the corrosion probability at the first repair is not changed with 51.9%, but better reliability is achieved for the later repairs compared to the original case where $\mu_{Xr} = 10 \text{ mm}$. Hence, if both strategies are applied for RC structures, the probability of corrosion initiation would be lower (Figure 14c). However, other factors, such as maintenance costs and environmental impacts, should

638 also be involved in infrastructure life cycle assessment to formulate a cost-effective and
 639 environmentally friendly maintenance strategy.



642 **Figure 14:** Probability of corrosion initiation of two cases (a) 10-year repair intervals (b) 20mm extra
 643 replacement depth (c) 10-year repair intervals and 20mm extra replacement depth

644 **Conclusions and future work**

645 In this study, a probabilistic prediction model of structural maintenance in a marine environment was
 646 proposed by combining Latin Hypercube Sampling technique with a time-dependent model of chloride
 647 transport in repaired concrete structures. Besides the calculation framework for service life and service
 648 life extension of corroded RC structures with repair, the effects of maintenance actions on the chloride
 649 transport behaviors within concrete were also discussed. By using the proposed computation approach,
 650 the following conclusions can be drawn:

- 651 1. Choosing a concrete replacement depth that is deeper than the existing cover depth is
 652 recommended for repairing RC structures subjected to chloride-induced corrosion, because it
 653 can slow down the chloride redistribution from the aged concrete to the critical depth. Also, a
 654 deeper repair depth leads to fewer repairs and larger pre-schedule repair intervals compared to

655 the shallower concrete repair. For instance, in section 3.2 was found that increasing this repair
656 depth by 10 mm leads to one repair less during the structural lifetime.

- 657 2. Both corrective and preventive repair strategies were performed in the probabilistic
658 assessment. For the corrective repair strategy, repair is carried out when the chloride content at
659 the rebar cover reaches a specific critical chloride value, and the total expected number of
660 repairs can be used as a factor for comparing different repair strategies. It was observed that,
661 excepting the first longer repair application time, probability of corrosion initiation was lower
662 than 0.52 between repair periods. In the case of time constraint and expensive computational
663 cost, the simulation of service life prediction for a few initial repair stages is sufficient as a
664 basic result for formulating the maintenance schedule by the preventive approach.
- 665 3. For the preventive repair model, the scheduled repair times could be determined to be less than
666 the mean values of repair application times obtained from the corrective repair model. This
667 maintenance approach seems to be efficient for service life prediction and maintenance
668 planning because an acceptable probability of corrosion initiation can be achieved by adopting
669 some practical repair strategies. For example, the minimum probability of corrosion initiation
670 is recorded to be below 0.43 for all repair times when applying 10-year repair intervals and
671 20mm extra replacement depth. In conclusion, the application of shorter repair intervals could
672 contribute to the decrease of corrosion risk for the first repair, while the selection of the repair
673 depth could mainly influence the probability of corrosion initiation after the first repair.
- 674 4. The proposed model, however, still has some limitations. It was applied for marine structures
675 exposed to atmospheric or submerged zones with constant chloride surface concentration.
676 Further research should thus aim to develop a comprehensive stochastic model considering
677 more realistic exposure conditions, chloride diffusion-convection coupling system in concrete
678 and time-dependency of surface chloride concentration. Also, real data (related to concrete
679 chloride diffusivity and assessing criteria) is required to estimate correlations between the
680 parameters used in the model and to improve the performance of the maintenance model.
681 Accordingly, the model-based approach for maintenance schedule could be compared with a
682 practical engineering case and applied in practice. This work is under progress.

683 **Acknowledgments**

684 This study was carried out in the framework of the Strengthening the Territory's Resilience to Risks of
685 Natural, Climate and Human Origin (SIRMA) project, which is co-financed by the European Regional
686 Development Fund (ERDF) through INTERREG Atlantic Area Program with application code:
687 EAPA_826/2018. The sole responsibility for the content of this publication lies with the authors. It
688 does not necessarily reflect the opinion of the European Union. Neither the INTERREG Europe
689 program authorities are responsible for any use that may be made of the information contained therein.

690 The authors also acknowledge the insightful and constructive comments from the five anonymous
691 reviewers.

692 **References**

- 693 [1] I. Stipanovic Oslakovic, D. Bjegovic, D. Mikulic, Evaluation of service life design models on
694 concrete structures exposed to marine environment, *Mater. Struct.* 43 (2010) 1397–1412.
695 <https://doi.org/10.1617/s11527-010-9590-z>.
- 696 [2] K. Tuutti, Corrosion of steel in concrete, Swedish Cement and Concrete Research Institute,
697 Stockholm., 1982.
- 698 [3] Q. Liu, Z. Hu, X. Wang, H. Zhao, K. Qian, L. Li, Z. Meng, Numerical study on cracking and its
699 effect on chloride transport in concrete subjected to external load, *Constr. Build. Mater.* 325
700 (2022) 126797. <https://doi.org/10.1016/j.conbuildmat.2022.126797>.
- 701 [4] Y. Ma, Z. Guo, L. Wang, J. Zhang, Experimental investigation of corrosion effect on bond
702 behavior between reinforcing bar and concrete, *Constr. Build. Mater.* 152 (2017) 240–249.
703 <https://doi.org/10.1016/j.conbuildmat.2017.06.169>.
- 704 [5] E. Bastidas-Arteaga, A. Chateauneuf, M. Sánchez-Silva, Ph. Bressolette, F. Schoefs, A
705 comprehensive probabilistic model of chloride ingress in unsaturated concrete, *Eng. Struct.* 33
706 (2011) 720–730. <https://doi.org/10.1016/j.engstruct.2010.11.008>.
- 707 [6] M. Liu, D.M. Frangopol, Optimal bridge maintenance planning based on probabilistic
708 performance prediction, *Eng. Struct.* 26 (2004) 991–1002.
709 <https://doi.org/10.1016/j.engstruct.2004.03.003>.
- 710 [7] X. Cheng, J. Xia, R. Wu, W. Jin, C. Pan, Optimisation of sacrificial anode cathodic protection
711 system in chloride-contaminated reinforced concrete structure, *J. Build. Eng.* 45 (2022) 103515.
712 <https://doi.org/10.1016/j.jobbe.2021.103515>.
- 713 [8] BSEN1504-9. Products and systems for the protection and repair of concrete structures.
714 Definitions, requirements, quality control and evaluation of conformity - General principles for
715 use of products and systems, (2008).
- 716 [9] E. Bastidas-Arteaga, F. Schoefs, Stochastic improvement of inspection and maintenance of
717 corroding reinforced concrete structures placed in unsaturated environments, *Eng. Struct.* 41
718 (2012) 50–62. <https://doi.org/10.1016/j.engstruct.2012.03.011>.
- 719 [10] C.A. Eldho, S. Jones, O. Nanayakkara, J. Xia, Performance of Concrete Patch Repairs: From a
720 Durability Point of View, in: *Proc. 5th Int. Conf. Durab. Concr. Struct.*, Purdue University Press,
721 2016. <https://doi.org/10.5703/1288284316112>.
- 722 [11] A. Rahimi, C. Gehlen, T. Reschke, A. Westendarp, Approaches for Modelling the Residual
723 Service Life of Marine Concrete Structures, *Int. J. Corros.* 2014 (2014) 1–11.
724 <https://doi.org/10.1155/2014/432472>.

- 725 [12] S. Soleimani, P. Ghods, O.B. Isgor, J. Zhang, Modeling the kinetics of corrosion in concrete patch
726 repairs and identification of governing parameters, *Cem. Concr. Compos.* 32 (2010) 360–368.
727 <https://doi.org/10.1016/j.cemconcomp.2010.02.001>.
- 728 [13] Truong Q.C., El Soueidy CP., Bastidas-Arteaga E., Li Y., Optimizing Cover Rebuilding
729 Maintenance for Reinforced Concrete Structures Exposed to Chloride Attack, in: *Proc. 1st Conf.*
730 *Eur. Assoc. Qual. Control Bridg. Struct.*, Springer, Cham, 2022. [https://doi.org/10.1007/978-3-](https://doi.org/10.1007/978-3-030-91877-4_18)
731 [030-91877-4_18](https://doi.org/10.1007/978-3-030-91877-4_18).
- 732 [14] A. Petcherdchoo, Probability-Based Sensitivity of Service Life of Chloride-Attacked Concrete
733 Structures with Multiple Cover Concrete Repairs, *Adv. Civ. Eng.* 2018 (2018) 1–17.
734 <https://doi.org/10.1155/2018/4525646>.
- 735 [15] H.-W. Song, H.-B. Shim, A. Petcherdchoo, S.-K. Park, Service life prediction of repaired
736 concrete structures under chloride environment using finite difference method, *Cem. Concr.*
737 *Compos.* 31 (2009) 120–127. <https://doi.org/10.1016/j.cemconcomp.2008.11.002>.
- 738 [16] Y. Ma, Z. Guo, L. Wang, J. Zhang, Probabilistic Life Prediction for Reinforced Concrete
739 Structures Subjected to Seasonal Corrosion-Fatigue Damage, *J. Struct. Eng.* 146 (2020)
740 04020117. [https://doi.org/10.1061/\(ASCE\)ST.1943-541X.0002666](https://doi.org/10.1061/(ASCE)ST.1943-541X.0002666).
- 741 [17] P. Lehner, M. Hornáková, K. Hrabová, Sensitivity Analysis of Stochastic Calculation of SCC
742 Regarding Aggressive Environment, *Materials.* 14 (2021) 6838.
743 <https://doi.org/10.3390/ma14226838>.
- 744 [18] C.G. Nogueira, E.D. Leonel, H.B. Coda, Probabilistic failure modelling of reinforced concrete
745 structures subjected to chloride penetration, *Int. J. Adv. Struct. Eng.* 4 (2012) 10.
746 <https://doi.org/10.1186/2008-6695-4-10>.
- 747 [19] B. Yu, C. Ning, B. Li, Probabilistic durability assessment of concrete structures in marine
748 environments: Reliability and sensitivity analysis, *China Ocean Eng.* 31 (2017) 63–73.
749 <https://doi.org/10.1007/s13344-017-0008-3>.
- 750 [20] F. Hecht, New development in FreeFem++, *J. Numer. Math.* (2012) 251–266.
- 751 [21] E. Zacchei, E. Bastidas-Arteaga, Multifactorial Chloride Ingress Model for Reinforced Concrete
752 Structures Subjected to Unsaturated Conditions, *Buildings.* 12 (2022) 107.
753 <https://doi.org/10.3390/buildings12020107>.
- 754 [22] A. Ababneh, F. Benboudjema, Y. Xi, Chloride Penetration in Nonsaturated Concrete, *J. Mater.*
755 *Civ. Eng.* 15 (2003) 183–191. [https://doi.org/10.1061/\(ASCE\)0899-1561\(2003\)15:2\(183\)](https://doi.org/10.1061/(ASCE)0899-1561(2003)15:2(183)).
- 756 [23] R.V.S. Anna V. Saetta and Renato V. Vitaliani, Analysis of Chloride Diffusion into Partially
757 Saturated Concrete, *ACI Mater. J.* 90 (1993). <https://doi.org/10.14359/3874>.
- 758 [24] B. Martín-Pérez, S.J. Pantazopoulou, M.D.A. Thomas, Numerical solution of mass transport
759 equations in concrete structures, *Comput. Struct.* 79 (2001) 1251–1264.
760 [https://doi.org/10.1016/S0045-7949\(01\)00018-9](https://doi.org/10.1016/S0045-7949(01)00018-9).
- 761 [25] F. Qu, W. Li, W. Dong, V.W.Y. Tam, T. Yu, Durability deterioration of concrete under marine

- 762 environment from material to structure: A critical review, *J. Build. Eng.* 35 (2021) 102074.
763 <https://doi.org/10.1016/j.jobbe.2020.102074>.
- 764 [26] E. Bastidas- Arteaga, C.-P.E. Soueidy, O. Amiri, P.T. Nguyen, Polynomial chaos expansion for
765 lifetime assessment and sensitivity analysis of reinforced concrete structures subjected to chloride
766 ingress and climate change, *Struct. Concr.* (2020) 1–12. <https://doi.org/10.1002/suco.201900398>.
- 767 [27] Q. Liu, L. Sun, X. Zhu, L. Xu, G. Zhao, Chloride transport in the reinforced concrete column
768 under the marine environment: Distinguish the atmospheric, tidal-splash and submerged zones,
769 *Structures.* 39 (2022) 365–377. <https://doi.org/10.1016/j.istruc.2022.03.041>.
- 770 [28] X. Chen, A. Yu, G. Liu, P. Chen, Q. Liang, A multi-phase mesoscopic simulation model for the
771 diffusion of chloride in concrete under freeze–thaw cycles, *Constr. Build. Mater.* 265 (2020)
772 120223. <https://doi.org/10.1016/j.conbuildmat.2020.120223>.
- 773 [29] M.M. L. O. Nilsson and L. Tang, Effect of Non-Linear Chloride Binding on the Prediction of
774 Chloride Penetration Into Concrete Structures, *ACI Symp. Publ.* 145 (1994).
775 <https://doi.org/10.14359/4554>.
- 776 [30] T. Luping, L.-O. Nilsson, Chloride binding capacity and binding isotherms of OPC pastes and
777 mortars, *Cem. Concr. Res.* 23 (1993) 247–253. [https://doi.org/10.1016/0008-8846\(93\)90089-R](https://doi.org/10.1016/0008-8846(93)90089-R).
- 778 [31] G.K. Glass, N.R. Buenfeld, The influence of chloride binding on the chloride induced corrosion
779 risk in reinforced concrete, *Corros. Sci.* (2000) 16.
- 780 [32] C.L. Page, N.R. Short, A. El Tarras, Diffusion of chloride ions in hardened cement pastes, *Cem.*
781 *Concr. Res.* 11 (1981) 395–406. [https://doi.org/10.1016/0008-8846\(81\)90111-3](https://doi.org/10.1016/0008-8846(81)90111-3).
- 782 [33] Z.P. Baant, L.J. Najjar, Drying of concrete as a nonlinear diffusion problem, *Cem. Concr. Res.* 1
783 (1971) 461–473.
- 784 [34] Y. Xi, Z.P. Bažant, H.M. Jennings, Moisture diffusion in cementitious materials Adsorption
785 isotherms, *Adv. Cem. Based Mater.* 1 (1994) 248–257. [https://doi.org/10.1016/1065-7355\(94\)90033-7](https://doi.org/10.1016/1065-7355(94)90033-7).
- 787 [35] D. Val, Service-life performance of RC structures made with supplementary cementitious
788 materials in chloride-contaminated environments, in: *Concr. - Int. RILEM-JCI Semin. Concr.*
789 *Durab. Serv. Life Plan. Curing Crack Control Perform. Harsh Environ., RILEM Publications*
790 *SARL, Dead Sea, Israel, 2006: pp. 363–373.* <https://doi.org/10.1617/291214390X.038>.
- 791 [36] A.C. Estes, D.M. Frangopol, Minimum expected cost-oriented optimal maintenance planning for
792 deteriorating structures: application to concrete bridge decks, *Reliab. Eng. Syst. Saf.* 73 (2001)
793 281–291. [https://doi.org/10.1016/S0951-8320\(01\)00044-8](https://doi.org/10.1016/S0951-8320(01)00044-8).
- 794 [37] M. Raupach, Patch repairs on reinforced concrete structures – Model investigations on the
795 required size and practical consequences, *Cem. Concr. Compos.* 28 (2006) 679–684.
796 <https://doi.org/10.1016/j.cemconcomp.2006.05.016>.
- 797 [38] J. Shi, B. Liu, J. Qin, J. Jiang, X. Wu, J. Tan, Experimental study of performance of repair mortar:
798 Evaluation of in-situ tests and correlation analysis, *J. Build. Eng.* 31 (2020) 101325.

- 799 <https://doi.org/10.1016/j.jobe.2020.101325>.
- 800 [39] E. Bastidas-Arteaga, M.G. Stewart, Economic assessment of climate adaptation strategies for
801 existing reinforced concrete structures subjected to chloride-induced corrosion, *Struct. Infrastruct.*
802 *Eng.* 12 (2016) 432–449. <https://doi.org/10.1080/15732479.2015.1020499>.
- 803 [40] J. Lizarazo-Marriaga, C. Higuera, I. Guzmán, L. Fonseca, Probabilistic modeling to predict fly-
804 ash concrete corrosion initiation, *J. Build. Eng.* 30 (2020) 101296.
805 <https://doi.org/10.1016/j.jobe.2020.101296>.
- 806 [41] S. Dastmalchi, H.V. Burton, Effect of modeling uncertainty on multi-limit state performance of
807 controlled rocking steel braced frames, *J. Build. Eng.* 39 (2021) 102308.
808 <https://doi.org/10.1016/j.jobe.2021.102308>.
- 809 [42] A. Olsson, G. Sandberg, O. Dahlblom, On Latin hypercube sampling for structural reliability
810 analysis, *Struct. Saf.* 25 (2003) 47–68. [https://doi.org/10.1016/S0167-4730\(02\)00039-5](https://doi.org/10.1016/S0167-4730(02)00039-5).
- 811 [43] A.V. Monteiro, A. Gonçalves, Probabilistic assessment of the depassivation limit state of
812 reinforced concrete structures based on inspection results, *J. Build. Eng.* 49 (2022) 104063.
813 <https://doi.org/10.1016/j.jobe.2022.104063>.
- 814 [44] D.V. Val, P.A. Trapper, Probabilistic evaluation of initiation time of chloride-induced corrosion,
815 *Reliab. Eng. Syst. Saf.* 93 (2008) 364–372. <https://doi.org/10.1016/j.res.2006.12.010>.
- 816 [45] DuraCrete. Probabilistic performance based durability design of concrete structure, (2000).
- 817 [46] R. McGee, Modelling of durability performance of Tasmanian bridges, Melchers RE Stewart MG
818 Ed. *Appl. Stat. Probab. Civ. Eng.* (2000) 297–306.
- 819 [47] K. Vu, M. Stewart, Structural reliability of concrete bridges including improved chloride-induced
820 corrosion models, *Struct Saf.* 22 (2000) 313–333.

821