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Quynh Chau Truong

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THESE DE DOCTORAT

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ECOLE DOCTORALE N° 602
Sciences de l'Ingénierie et des Systèmes
Spécialité : « Génie Civil »

Par

Quynh Chau TRUONG

Probabilistic Modeling and Optimization of Repair Strategies for Sustainable Maintenance of Reinforced Concrete Structures subjected to Chloride Ingress

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PhD thesis

Présentée par

Quynh Chau TRUONG

*Title: Probabilistic Modeling and Optimization of Repair
Strategies for Sustainable Maintenance of Reinforced Concrete
Structures Subjected to Chloride Ingress*

2023

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Abstract

The corrosion of reinforced concrete structures is a widespread problem that can compromise their safety and durability. Therefore, it is crucial to have effective repair and maintenance management to ensure their long-term performance and safety. This thesis aims to develop probabilistic models and optimize repair strategies for reinforced concrete structures exposed to chloride ingress, with the goal of achieving sustainable maintenance practices. To achieve this, a comprehensive maintenance model is developed to simulate chloride ingress when the structure is repaired by concrete cover rebuilding. The model considers factors such as repair thickness and intervals, concrete quality, and environmental conditions. This enables the evaluation of the efficiency of repair strategies to improve maintenance operations.

A probabilistic approach, based on Latin Hypercube Sampling and Polynomial Chaos Expansions, is proposed to account for uncertainty, predict the risk of corrosion, and optimize the maintenance schedule. Furthermore, a decision framework that considers costs, environmental impact, and safety indicators is introduced to select a sustainable repair strategy. Throughout the manuscript, several numerical examples and case studies illustrate the above points. Overall, the proposed models and frameworks can assist engineers and practitioners in developing effective and sustainable maintenance and repair strategies to improve the safety and longevity of corroding reinforced concrete structures.

Keywords: chloride penetration, maintenance model, reinforced concrete, corrosion, cover rebuilding, cost-effectiveness, sustainability

Résumé

La corrosion des structures en béton armé est un problème répandu qui peut compromettre leur sécurité et leur durabilité. Il est donc crucial d'avoir une gestion efficace de la réparation et de la maintenance pour assurer leur performance et leur sécurité à long terme. Cette thèse vise à développer des modèles probabilistes et à optimiser les stratégies de réparation pour les structures en béton armé exposées à la pénétration des chlorures, dans le but d'atteindre des pratiques de maintenance durables. Pour y parvenir, un modèle de maintenance a été développé pour simuler les processus de transfert de chlorures lorsque la structure est réparée avec la reconstruction de la couverture. Le modèle prend en compte des facteurs tels que l'épaisseur et les intervalles de réparation, la qualité du béton et les conditions environnementales. Cela permet d'évaluer l'efficacité des stratégies de réparation pour améliorer les opérations de maintenance.

Une approche probabiliste, basée sur l'échantillonnage Latin Hypercube et le chaos polynomial, a été proposée pour tenir compte de l'incertitude, prédire le risque de corrosion et optimiser les actions de maintenance. De plus, un cadre décisionnel qui prend en compte des indicateurs de coûts, d'impact environnemental et de sécurité a été mis en œuvre pour sélectionner une stratégie de réparation durable. Tout au long du manuscrit, plusieurs exemples numériques et études de cas illustrent les points susmentionnés. Dans l'ensemble, les modèles et cadres proposés peuvent aider les ingénieurs et les praticiens à développer des stratégies de maintenance et de réparation efficaces et durables pour améliorer la sécurité et la durée de vie des structures en béton armé corrodées.

Mots clés: pénétration des chlorures, modèle d'entretien, béton armé, corrosion, reconstruction de la couverture, durabilité.

Publications

Articles

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2. Truong Q.C., Van Nguyen C., El Soueidy CP., Bastidas-Arteaga E. (2023). Service Life Prediction of Reinforced Concrete Coastal Structures in Vietnam Subjected to Chloride Ingress. In: Reddy, J.N., Wang, C.M., Luong, V.H., Le, A.T. (eds) *ICSCEA 2021*. Lecture Notes in Civil Engineering, vol 268. Springer, Singapore. https://doi.org/10.1007/978-981-19-3303-5_40
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Table of Contents

Abstract

Résumé

Publications

Table of Contents

Introduction	15
Background.....	15
Objectives and Scope.....	17
Document Organisation.....	17
Chapter 1 Literature Review.....	21
1.1. Introduction	21
1.2. Life-cycle of RC structures subjected to corrosion.....	22
1.3. Modeling chloride ingress into concrete considering the repair/maintenance actions.....	24
1.3.1. Chloride ingress model	24
1.3.2. Chloride ingress model in repaired concrete	25
1.4. Probabilistic approaches for maintenance modeling and optimization	27
1.4.1. Time-dependent reliability analysis	29
1.4.2. Risk assessment	29
1.4.3. Markov Chain Models.....	30
1.4.4. Bayesian method	30
1.5. Methods for reliability analysis	31
1.5.1. Basic concept for reliability analysis	31
1.5.2. Reliability methods.....	32
1.6. Framework for the proposed maintenance management	35
1.6.1. Safety requirements.....	35
1.6.2. Cost considerations	36
1.6.3. Sustainability	37
1.7. Conclusions.....	38
Chapter 2 Modeling Chloride Ingress into Repaired Concrete.....	41
2.1. Introduction	41
2.2. Deterioration model	42
2.2.1. Governing equations for chloride ingress model	42
2.2.2. Numerical modeling of chloride ingress	44
2.2.3. Model validation	45
2.3. Chloride ingress model considering the repair/maintenance actions	48
2.3.1. One-dimensional maintenance model	49
2.3.2. Two-dimensional maintenance model	51
2.3.3. Numerical solution and modeling process in repaired concrete in FreeFem++	52
2.4. Case study 1: Deterministic 1D model of chloride ingress in repaired concrete	53

2.4.1.	<i>Problem description</i>	53
2.4.2.	<i>Maintenance model and results</i>	54
2.5.	Case study 2: Deterministic 2D model of chloride ingress in repaired concrete	58
2.5.1.	<i>Problem description</i>	58
2.5.2.	<i>Analysis of the results</i>	60
2.6.	Case study 3: Influence of temperature and humidity on corrosion process and maintenance plan.....	67
2.6.1.	<i>Problem description</i>	67
2.6.2.	<i>Results and discussions</i>	68
2.7.	Summary and conclusions	71
Chapter 3	<i>Probability-based Maintenance Modeling and Planning for Chloride-Contaminated Concrete Structures</i>	73
3.1.	Introduction	73
3.2.	Probabilistic assessment for reliability analysis	74
3.2.1.	<i>Probability of corrosion initiation</i>	74
3.2.2.	<i>Methodology of probabilistic analysis</i>	75
3.2.3.	<i>Framework of probabilistic maintenance modeling</i>	76
3.2.4.	<i>Probabilistic data</i>	77
3.3.	Case study 1: LHS -based probabilistic assessment of 1D maintenance model	78
3.3.1.	<i>Problem description</i>	78
3.3.2.	<i>Results and discussion</i>	79
3.4.	Surrogate-based probabilistic assessment of maintenance model	87
3.4.1.	<i>Polynomial Chaos Expansions</i>	87
3.4.2.	<i>PCE-based Global Sensitivity Analysis</i>	89
3.5.	Case study 2: PCE-based probabilistic assessment for 1D maintenance model.....	89
3.5.1.	<i>Problem description</i>	89
3.5.2.	<i>Results and discussions</i>	90
3.6.	Case study 3: Optimizing maintenance schedule through PCE-based probabilistic assessment.....	95
3.6.1.	<i>Repair criterion for maintenance planning</i>	95
3.6.2.	<i>Problem description</i>	95
3.6.3.	<i>Results and discussions</i>	98
3.7.	Summary and conclusions	105
Chapter 4	<i>Selecting a Cost-effective and Sustainable Repair strategy for Chloride-contaminated Concrete Structures</i>	109
4.1.	Introduction	109
4.2.	Repair and sustainable maintenance management for corroded structures	110
4.3.	Cost-effective maintenance management	111
4.3.1.	<i>Life-cycle cost estimation methodology</i>	112
4.3.2.	<i>Life-cycle maintenance cost estimation</i>	113
4.4.	Sustainable maintenance management.....	114
4.4.1.	<i>Costs</i>	116
4.4.2.	<i>CO₂ emissions</i>	116
4.4.3.	<i>Waste generation</i>	117
4.4.4.	<i>Methods for sustainable maintenance management</i>	118
4.5.	Case study 1.....	119
4.5.1.	<i>Problem description and basic considerations</i>	119
4.5.2.	<i>Maintenance schedule of each repair strategy</i>	120
4.5.3.	<i>Results and discussion</i>	121
4.6.	Case study 2.....	123

4.7. Conclusions.....	126
Closure	129
<i>Summary and Conclusions</i>	129
<i>Perspectives</i>	132
References	135
Appendix A <i>Methods for reliability analysis</i>	147
A.1. Time-dependent reliability analysis	147
A.2. Methods for reliability assessment	147
A.2.1 Monte-Carlo Simulation.....	148
A.2.2 Latin Hypercube Sampling	148
Appendix B <i>The VIKOR method for multi- criteria optimisation</i>	151
Résumé étendu en Français	153
1. Contexte	153
2. Objectifs et champ d'application	155
3. Modèle de détérioration.....	155
3.1 <i>Équations de base du modèle de pénétration des chlorures</i>	155
3.2 <i>Validation du modèle</i>	157
4. Modèle de pénétration des chlorures tenant compte des actions de réparation/entretien.....	160
5. Etude probabiliste pour l'analyse de fiabilité	162
5.1 <i>Probabilité de déclenchement de la corrosion</i>	162
5.2 <i>Méthodologie de l'analyse probabiliste</i>	163
5.3 <i>Cadre de la modélisation probabiliste de la maintenance</i>	164
6. Réparation et gestion durable de l'entretien des structures en béton armé polluées par les chlorures	165
6.1 <i>Coûts</i>	166
6.2 <i>Émissions de CO₂</i>	167
6.3 <i>Production de déchets</i>	168
6.4 <i>Méthodes de gestion durable de la maintenance</i>	169
7. Principaux résultats et discussions.....	170
7.1 <i>Modélisation de la pénétration des chlorures dans le béton réparé</i>	170
7.2 <i>Modélisation et planification de la maintenance basée sur les probabilités pour les structures en béton contaminées par les chlorures</i>	171
7.3 <i>Sélection d'une stratégie de réparation rentable et durable pour les structures en béton contaminées par les chlorures</i>	172
8. Conclusions et perspectives	173

Introduction

Background

Reinforced concrete (RC) civil infrastructure systems are important assets for the socio-economic development of any country. The design and maintenance of these systems for a particular service lifetime have been recognised as critical issues worldwide (Bastidas-Arteaga et al., 2011). RC structures are characterised by high durability; however, during their service life, they are exposed to internal and external influences that affect performance, serviceability, and safety. Among others, corrosion of steel in concrete due to chloride penetration is the main cause of RC deterioration and eventual shortening of assets' service life (Stipanovic Oslakovic, Bjegovic and Mikulic, 2010). After construction, The corrosion-induced damage process can be divided into two main phases: the initiation phase and the propagation phase. The high alkalinity of the cement paste surrounding the reinforcing steel creates a reactive, passive condition. However, a high amount of chloride next to the reinforcing bars could break through this passive film and corrosion initiates (Tuutti, 1982; Liu *et al.*, 2022). The propagation period follows corrosion initiation and is defined as the period in which active corrosion occurs. According Tuutti (1982), the period between the initial condition of a structure and the onset of corrosion is critical and determines the service life of the structure.

High chloride environments are known to accelerate corrosion deterioration and may reduce structure service life in some cases from an expected range of 50-100 years to a range of 10-30 years (Engelund, Sorensen and Sorensen, 1999; Zhao and Zin, 2006). In addition, chloride-induced damage can be a slow and silent process that can lead to extensive damage (e.g. pitting corrosion) before it is noticed. Structure maintenance and repair can ensure and extend structure serviceability and maintain its safety and, therefore, its lifespan (Liu and Frangopol, 2004; Cheng *et al.*, 2022). Indeed, in a preventive perspective, maintenance and repair operations should be implemented before or at least as soon as corrosion begins.

Therefore, early detection and monitoring of corrosion are essential to avoid costly and extensive repairs. To this end, visual inspections, destructive testing methods (DT), and other advanced non-destructive testing (NDT) techniques can be used, but these methods can be expensive and time-consuming, in some cases these techniques could lead to misdiagnosis (visual inspection), or cause additional damage to the structure. Therefore, developing a robust model to predict the service life of chloride-polluted RC structures and effective strategies for periodic inspection and repair play an important role in achieving the goal of durability and ensuring the continued operation of repaired structures (Liu and Frangopol, 2004; Bastidas-Arteaga and Schoefs, 2012). Several models of varying complexity have been proposed to predict and describe degradation (Song *et al.*, 2009; Soleimani *et al.*, 2010), but deterministic application can be misleading due to uncertainties in key parameters and models. Probabilistic degradation models have been developed to account for such uncertainties. However, modeling chloride ingress in repaired RC structures is a complex task because not only the original degradation process but also the condition after repair must be considered (Barkey, 2004; Shi, 2020), since repair is not a guarantee of complete restoration of the structure to its original condition. Therefore, probabilistic modeling of degradation processes in repaired RC structures should be treated as a separate and important research topic. Based on this, probabilistic maintenance modeling can assist in the selection of repair strategies and materials and provide a more realistic estimate of the remaining life of RC structures.

In addition, these maintenance activities generate costs and environmental impacts. While economic considerations are often at the forefront of management decisions, it is important to consider environmental and social issues as well (Bastidas-Arteaga and Schoefs, 2015; Petcherdchoo, 2015). The production of cement, which is an important component of concrete, contributes significantly to greenhouse gas emissions; and waste generated during repair work can have a negative impact on the environment. Therefore, an effective management of corroded RC structures requires a sustainable approach that balances economic, environmental, and social factors. For example, the selection of appropriate maintenance strategies and materials can significantly affect the environmental impact of RC structures during their life cycle. The traditional reactive maintenance approach of waiting until damage occurs before taking action can be costly and environmentally damaging. A proactive maintenance approach, on the other hand, which includes regular inspections and preventive measures, can reduce the need for extensive repairs and the associated environmental impact (Bastidas-Arteaga and Schoefs, 2015). Therefore, it is essential to conduct a comprehensive study that considers all of these factors to formulate a decision-making framework, that identifies optimal solutions to maximize sustainability and minimize costs. Within this context, the methodology developed in this study integrates the following aspects:

- a comprehensive model of chloride penetration in concrete before/after repair,
- an efficient probabilistic approach for optimization of repair strategies, and
- the integration of cost and environmental impact in maintenance management.

Objectives and Scope

The objective of this study is to develop a probabilistic modeling and optimization of repair strategies for sustainable maintenance of RC structures subjected to chloride ingress. The study will focus on achieving the following specific objectives:

- Evaluate current methods that can solve the considered aspects of the management of corroding RC structures, highlight their advantages and disadvantages, and identify an effective approach to solve this problem.
- Build a comprehensive model for chloride penetration in repaired concrete, which is used for investigating the effectiveness of different repair strategies, maintenance planning and optimization.
- Integrate the maintenance model into a probabilistic framework to account for the uncertainties associated with deterioration and repair processes.
- Develop a method to optimize the performance of the maintenance strategy and maintenance planning.
- Integrate cost and environmental constraints into the decision-making process using appropriate approach to evaluate the sustainability of maintenance strategies.

Document Organisation

Chapter 1 discusses the effects of chloride-induced corrosion on the safety of RC structures and provides a critical overview of the current state of maintenance management of corroding RC structures. This literature review is used to identify an appropriate approach and determine research needs for the thesis.

Chapter 2 presents the development of the chloride ingress model in repaired concrete, with the objectives of formulating a maintenance model and investigating the effects of different repair strategies and environmental conditions on structural performance. This chapter also includes a description of the numerical approach chosen and some numerical examples.

Chapter 3 presents a probability-based approach for modeling and planning maintenance of chloride-polluted concrete structures. It covers the basic elements of reliability theory, a methodology for the probabilistic maintenance model, and the application of the surrogate model for probabilistic assessment and global sensitivity analysis. The chapter also includes

case studies to demonstrate the effectiveness of the proposed approach in controlling the probability of corrosion initiation and optimizing maintenance schedules.

Chapter 4 discusses the selection of a cost-effective and sustainable repair strategy for corroded concrete structures, including a description of the conceptual formulation, life-cycle cost analysis, the criteria considered in the sustainability analysis, and a tool for decision-making under multicriteria constraints. The chapter includes some case studies to illustrate the proposed methodology.

Chapter 1 *Literature Review*

1.1. Introduction

Corrosion of reinforcing steel in concrete can lead to structural damage, reduced load-bearing capacity, and even collapse in extreme cases. Therefore, it is important to manage and maintain corroded RC structures to ensure their longevity and structural integrity. Proper maintenance is a critical component of this management, as it can help prevent and mitigate corrosion-related damage, minimize repair costs, and extend the service life of the structure. Regular inspections, timely repairs and appropriate protective measures can help control corrosion in RC structures and ensure their continued safe use. The effectiveness of a repair system depends on several factors, including the type of structure, the operating conditions, the degree of deterioration, and the financial constraints of the project. Ultimately, repair systems should be evaluated based on cost, risk of failure, long-term performance, and other issues such as the environmental impact.

The expense of maintaining a deteriorating structure throughout its lifetime can significantly exceed the initial construction cost. Although the construction cost can be substantial, it is a one-time expense, while maintenance costs can span decades. There are numerous deteriorating structures in the United States, and national expenditures to maintain them represent a significant portion of the budget. For instance, there are more than 617,000 bridges across the United States. Currently, 42% of all bridges are at least 50 years old, and 46,154, or 7.5% of the nation's bridges, are considered structurally deficient, meaning they are in "poor" condition. Nearly 231,000 bridges still need repair and preservation work (ASCE Report Card, 2021). The estimated cost of addressing the bridge backlog is about \$125 billion. With such massive expenditures, any efficiencies or optimization achieved can result in significant cost savings.

Estimating the lifetime cost of a structure can be challenging without sufficient knowledge of the structure and its deterioration behavior. To optimize the maintenance of a corroding structure, a specific methodology can be applied, including:

1. Define the structure and its failure, i.e., understand the life cycle of RC structures subjected to corrosion, identify the critical stages to be considered in maintenance;
2. Develop a deterioration model that considers factors such as corrosion rates, the effects of environmental conditions, and further corrosion after periodic repairs;
3. Determine the repair options available, the associated costs, and their impact on the structure's performance;
4. Conduct the probabilistic assessment for repair and maintenance modeling;
5. Formulate an optimization problem that considers the optimization criteria, the failure constraints, the durability of the structure, and any imposed constraints such as financial and environmental; and
6. Establish a decision-making framework for selecting the best cost-effective and sustainable repair solution.

The main purpose of this chapter is to provide an overview of the methodology for optimizing the maintenance of corroding RC structures based on a probabilistic approach. The chapter is divided into several sections, starting with Section 1.2, which discusses the different phases of the life cycle of RC structures affected by corrosion. Section 1.3 provides a detailed overview of the mechanism of chloride transport in repaired concrete and previous research on the maintenance model. Subsequently, Section 1.4 presents various probabilistic approaches for modeling maintenance and optimizing repair strategies. Then some commonly used methods for reliability analysis are discussed in section 1.5. Finally, the conceptual framework for an effective maintenance management is discussed in section 1.6.

1.2. Life-cycle of RC structures subjected to corrosion

RC structures are commonly used in construction because of their high strength and durability. However, these structures can be susceptible to corrosion, which can significantly shorten their service life and compromise their structural integrity (Kumar Mehta, 1997; Stipanovic Oslakovic, Bjegovic and Mikulic, 2010). The life cycle of a RC structure exposed to corrosion can be analyzed from a structural point of view, taking into account the different phases of corrosion that must be considered in maintenance management, as shown in Figure 1.1 below:

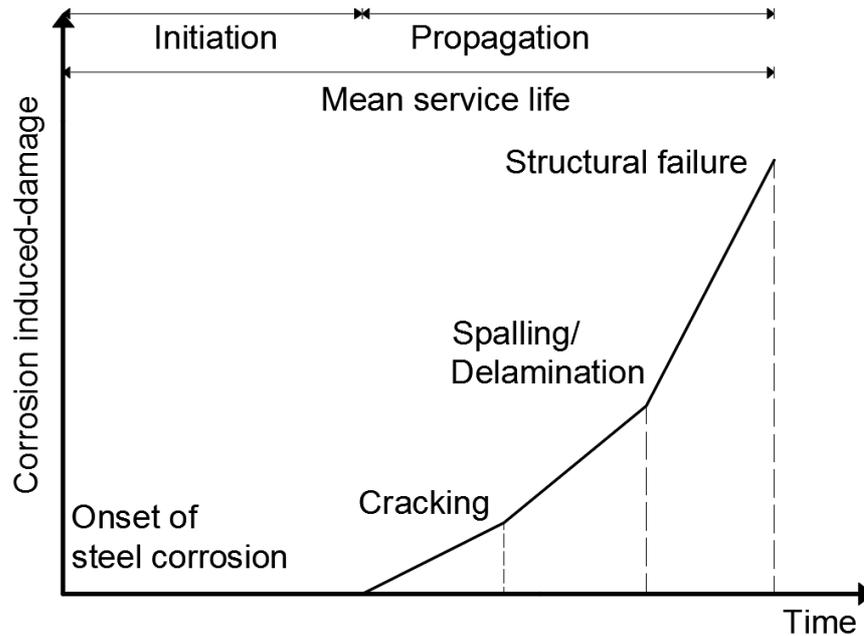


Figure 1.1: The corrosion process of RC structures

Phase 1: Corrosion initiation phase

It is the first phase of the corrosion process and is characterized by the beginning of corrosion activity on the reinforcing steel. Immediately after construction, the RC structure is exposed to various environmental factors that can contribute to corrosion, including moisture, oxygen, and chloride ions. The rate and severity of corrosion depends on the degree of exposure to these factors, as well as the pH of the concrete and the temperature (Kumar Mehta, 1997). Among corrosive elements, chloride-induced corrosion is one of the most common mechanisms of corrosion initiation in RC structures (Tuutti, 1982). Chlorides can penetrate the concrete cover by diffusion or by contact with salt water or de-icing salts. Once chloride ions reach the steel surface, they can break down the passive layer and initiate corrosion (e.g. Bastidas-Arteaga, 2009; Rodrigues *et al.*, 2021). Chloride-induced corrosion can be particularly aggressive, as it can occur even in concrete structures with a relatively thick concrete cover. The period between the original condition of a structure and the onset of corrosion is called *the time of corrosion initiation* (T_{ini}), and the limit state function at this stage is called the serviceability limit state (Andrade, 2017), which is evaluated by the probability of corrosion initiation.

Phase 2: Corrosion propagation phase

Propagation is the second phase of corrosion that occurs after the initiation phase. It is the period in which the active corrosion occurs (Tuutti, 1982). The corrosion process in this phase can reduce the cross-section of the reinforcement, which reduces structural capacity. This process leads to internal damage in the concrete, starting at the surface of the reinforcement and

leading to cracking, delamination and spalling in the concrete concrete as corrosion progresses (e.g. Adasooriya, Samarakoon and Gudmestad, 2017; Rodrigues *et al.*, 2021). If left unchecked, corrosion can eventually lead to complete failure of the structure (Bastidas-Arteaga and Schoefs, 2012). This can occur in a variety of ways, including sudden catastrophic failure or slow degradation of the steel's strength over time. Structural failure can be particularly dangerous when bridges, buildings and other critical infrastructure are involved. Early detection and intervention are critical to prevent the propagation phase of corrosion. The limit state function at this stage is the ultimate limit state (Teply and Vorechovská, 2012), which considers the probability of failure.

The phases of corrosion initiation and propagation are critical to understanding the complex corrosion process that occurs in RC structures. Effective corrosion control and maintenance strategies must be designed to address both phases of the corrosion process and account for the many factors that can contribute to the development and propagation of corrosion activity. From a conservative point of view, it is widely accepted that the optimal time for repairs is before the initiation phase is complete, when the concentration of chlorides near the reinforcement reaches the critical threshold and the steel's protective oxide layer is disrupted (Bastidas-Arteaga and Schoefs, 2012). Therefore, the management of chloride-polluted RC structures in this work will focus on the phase of corrosion initiation, rather than the phase of corrosion propagation. The literature of modeling chloride ingress considering the repair/maintenance actions are discussed in the next section.

1.3. Modeling chloride ingress into concrete considering the repair/maintenance actions

1.3.1. Chloride ingress model

The modeling of chloride transport into RC is based on whether exposure conditions lead to saturated or unsaturated concrete. In saturated conditions, Fick's second law was used to simulate chloride diffusion in concrete and considered a sufficient model (Tuutti, 1982; Zacchei and Bastidas-Arteaga, 2022a). The analytical solution for this model is based on the error function, which is valid only under the assumptions that RC structures are saturated, and diffusivity and surface chloride concentration are time-invariant. In comparison, under unsaturated conditions, chloride transport in concrete is a complex interaction between physical and chemical processes, and thus the diffusion and convection phenomena have been accounted for to study this mechanism (Anna V. Sietta, 1993; Martín-Pérez, Pantazopoulou and Thomas, 2001).

Recently, several numerical models have been developed to estimate the chloride distribution in concrete based on finite element method and finite difference method. Among them, Ying et al. (Ying, Xiao and Tam, 2013) proposed a model using a constant chloride diffusion coefficient which neglects its time variability due to concrete aging. Hu et al. (Hu *et al.*, 2017) established one-dimensional (1D) and two-dimensional (2D) models for time-dependent chloride diffusion that considered the effects of temperature, relative humidity, and a time aging factor of concrete. More comprehensive models were developed for various exposure conditions that considered the diffusion-convection coupling system during chloride penetration, seasonal variations in humidity and heat transfer, time-dependent surface boundary, concrete aging, and chloride binding capacity (Ababneh, Benboudjema and Xi, 2003; Bastidas-Arteaga *et al.*, 2011, 2022).

In this study, a time dependent chloride transport model is developed with the assumptions of constant temperature, humidity, and surface chloride concentration. With these assumptions, this model can be applied for marine structures exposed to atmospheric or submerged zones, where diffusion is the main driving force to transport chloride ions from the surrounding environment into concrete and the capillary suction is not considered (Chen *et al.*, 2020; Liu *et al.*, 2022). Indeed, this approach allows the development of a repair model with multiple cover repairs with lower technical difficulty and computational time, especially for performing probabilistic analyses. The detailed formulation of the proposed chloride penetration model is presented in Chapter 2.

1.3.2. Chloride ingress model in repaired concrete

Various techniques can be used to repair corroded RC structures, such as cover rebuilding, patching, electrochemical techniques, surface coatings, and cathodic protection (BSEN 1504-10:2017). Each repair method has its advantages and disadvantages, and the selection of the appropriate method depends on several factors, such as the extent and severity of the corrosion damage, the accessibility of the element, and the budget (Fagerlund, 2004; F. Binder, 2013). Among them, the cover rebuilding or cover replacement method is particularly common and is the focus of this study (Skoglund *et al.*, 2008). It removes the chloride-contaminated concrete and rehabilitates the structure by placing new concrete or repair mortar without damaging the reinforcement bars. The new material is typically placed as wet shotcrete, dry shotcrete, formed concrete, or by manual repair (BSEN 1504-10:2017). Building a comprehensive model to assess the degree of corrosion and predict the service life of RC structures with repairs is a challenging task that has become crucial for engineers to enable proper condition monitoring and appropriate retrofitting (Bastidas-Arteaga and Schoefs, 2012; Ma *et al.*, 2020).

Challenges in modeling chloride ingress in repaired concrete structures

The cover replacement method is an effective repair strategy for chloride-contaminated RC structures, as it uses a new concrete layer to act as a barrier against chloride ion penetration (Skoglund *et al.*, 2008). However, accurately modeling the diffusion of chloride ions through both layers and their interface is complex, due to the difference of chloride concentrations and the variability of properties within each layer and the system as a whole (Barkey, 2004; M. Raupach, 2006; Song *et al.*, 2009). Addressing these challenges requires extensive modeling of variations in properties and physics to investigate the effectiveness of repair strategies for corroded concrete structures.

Recent studies have focused on the modelling of chloride transport in concrete structure elements after repair, and the methods to predict repair application times during the design life of a structure. For instance, Eldho *et al.* (Eldho *et al.*, 2016) studied the performance of the patch repair method and showed that the electrochemical consequences of the method may decrease the effectiveness of patch repairs. More specifically, according to Rahimi *et al.* (Rahimi *et al.*, 2014), the diffusion mechanism of chloride ions in repaired concrete is a combination of chloride penetration from the concrete surface exposed to the environment and distribution of chloride ions that remain in the original and aged concrete. It was further shown that the resistivities of the patch and the original concrete, which can be used as an indicator of material quality, are among the most significant factors that influence the efficiency of maintenance (Soleimani *et al.*, 2010). Soleimani *et al.* (Soleimani *et al.*, 2010) recommended to choose patches with similar properties to avoid mechanical incompatibility problems at the interface between the patch and the original concrete. Concrete replacement depth is also a critical factor that reduces or delays the corrosion in repaired concrete structures by limiting the amount of chloride content at the rebar cover (Song *et al.*, 2009; Petcherdchoo, 2018). Several models have been proposed to determine the time of corrosion initiation in RC structures with multiple repairs throughout its service life. Both deterministic and probabilistic analysis have been performed, taking into account some main performance factors such as the properties of the patch, the replacement depth, or the repair time interval. However, due to the complexity of the chloride-induced corrosion mechanism in RC after one or several repairs, research on this topic with a comprehensive probabilistic approach is still limited. For instance, Petcherdchoo's study (Petcherdchoo, 2018) presented a probability-based sensitivity analysis of service life (the time to repair) of chloride-attacked concrete structures, which was solved by the combination of the Crank-Nicolson-based finite difference approach and the Latin Hypercube sampling technique. However, this methodology did not include some factors in chloride transport, such as the weather conditions (temperature and humidity), the chloride binding capacity, and time-dependency of the chloride diffusion coefficient. Moreover, the cover replacement depth, which

is intrinsically random due to the uncertainty in repair quality control, was considered deterministic.

In addition, the scope of previous studies was largely limited to a one-dimensional (1D) model and has not yet been applied to the two-dimensional problem that concerns several structural components in practice. For example, the corners and edges of crossbeams, square piles and columns, or rectangular concrete structures, are often exposed to two-dimensional (2D) chloride diffusion (Wu *et al.*, 2020; Liu *et al.*, 2022). In these cases, the use of 1D chloride diffusion models underestimates the chloride concentration at the critical locations of concrete structures, thereby overestimating the time for corrosion initiation. Indeed, there is little information on the underlying mechanisms of 2D chloride transport in repaired concrete structures and their long-term performance after periodic repairs. Therefore, accurate prediction of the service life of marine RC components to be repaired requires the use of 2D chloride-induced corrosion models, especially for modelling corners and edges in the concrete components. Further research is needed to investigate the modeling of two-dimensional chloride penetration in repaired concrete, as well as the performance of different repair strategies with a probabilistic approach. In addition, these repairs need to be optimized for maximum serviceability and safety while reducing costs and environmental impacts (Bastidas-Arteaga and Stewart, 2015; Petcherdchoo, 2015).

Based on the literature, this work aims to propose a comprehensive probability-based model of service life prediction for chloride-polluted RC structures with cover rebuilding repair. The study focuses on the assessment of corrosion initiation risk subjected to one-dimensional and two dimensional chloride ingress under marine environments. The model should take into account the complex process of chloride transport in repaired concrete, which involves a system of two layers of material. In addition, the model of time-dependent chloride ingress should consider the influence of general conditions of weather, chloride binding capacity, time-dependency of the chloride diffusion coefficient, and concrete aging factor. Based on these, the repair model could provide a reliable tool for evaluating the response of structures to maintenance actions, for comparing the effectiveness of different repair options, and for efficiently performing probabilistic assessments. The detailed formulation of the repair model, which takes into account all the aspects mentioned above, is presented in the Chapter 2.

1.4. Probabilistic approaches for maintenance modeling and optimization

Figure 1.2 outlines a conceptual framework for a management system that addresses structural decay. The system focuses on producing several optimal alternatives for a given structural component that can be compared to determine the best solution. The first step is to develop

potential maintenance strategies that consider various factors such as (1) the kinematics of the deterioration process, (2) environmental factors, and (3) socio-economic aspects relevant to the problem.

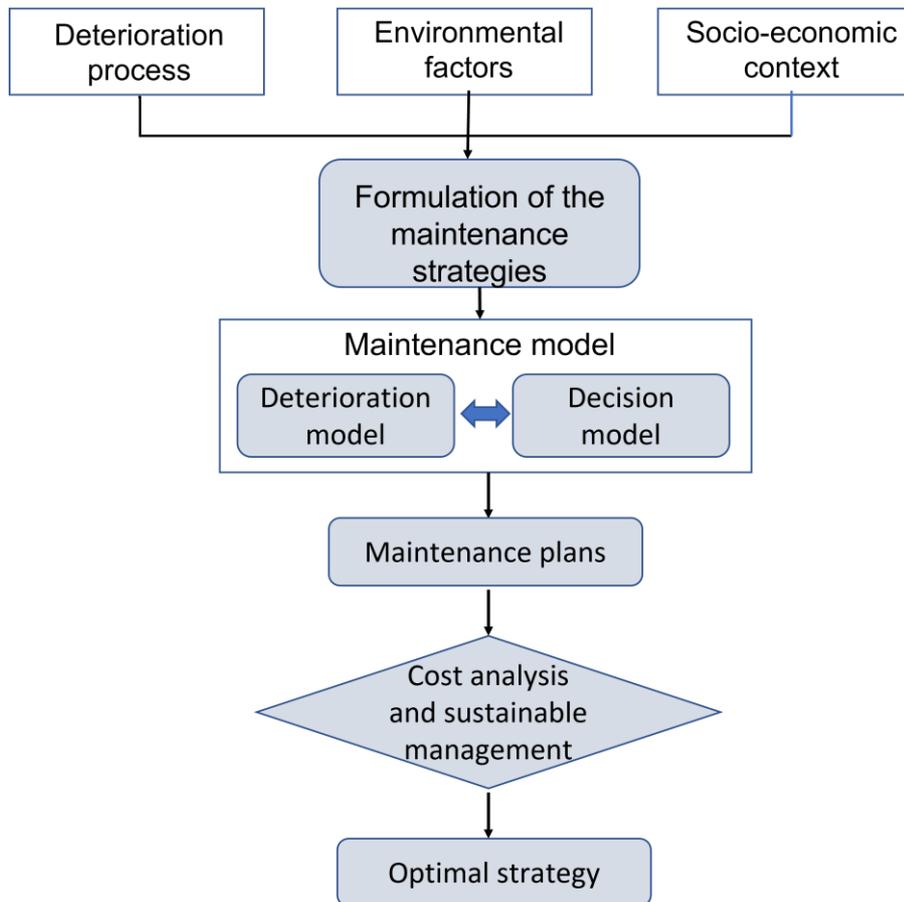


Figure 1.2: Conceptual framework of a management system

The formulation of a maintenance strategy includes the properties of original and repair materials, characteristics of repair methods (i.e., shape, thickness of repair section), and repair criterion, such as repair intervals and repair thresholds. Each strategy is then simulated in the maintenance model to evaluate its characteristics expressed as repair intervals or repair rates. As Kallen (2007) states, a maintenance model must consider two main aspects: the *deterioration model* and the *repair decision model*, as shown in Figure 1.2. The deterioration model evaluates the impact of the deterioration process on structural performance before/after repair, while the decision model introduces decision criteria such as repair intervals and repair thresholds. The results of the maintenance model ensure a certain level of safety, which implicitly relates to the repair threshold. Due to the different characteristics of maintenance strategies, each has a different repair and maintenance plan that meets the requirements for long-term durability and serviceability. Based on this, these maintenance strategies are then compared to find the most cost-effective or sustainable repair strategy.

In addition, for effective management of deteriorating structures, it is necessary to consider the variability and uncertainties that exist in the deterioration process and maintenance actions. Key uncertainty factors include the properties of construction and repair materials, ambient chloride concentrations, loading, and repair performance. Management of deteriorating structures through probabilistic approaches involves the use of statistical tools and techniques to evaluate the probability of failure (probability of corrosion initiation in this case) and determine optimal maintenance strategies. By incorporating probabilistic models into maintenance management, it is possible to achieve better cost efficiency, improve safety, and extend the life of structures. Commonly used probabilistic approaches that address the above issues and enable more accurate and effective management of deteriorating structures include the following:

1.4.1. Time-dependent reliability analysis

Reliability analysis is a probabilistic approach that is used to assess the probability of failure of a structure under specific conditions. It involves the use of mathematical models to estimate the probability of failure based on factors such as the loads, material properties, and structural design (Wang, 2021). In fact, decision-making can be based on different limit states – i.e., serviceability, ultimate, or failure modes for deterioration problem related to chloride ingress. It can also be used to predict the residual service life of a structure and to make informed decisions about repair. The development of the deterioration model proposed in this study enables the implementation of reliability analysis for management and optimization, where the probability of failure at a given time in the life of a repaired structure is calculated, both for the decision criteria of repair interval and repair threshold. This approach has not been explored in depth in previous studies (Sancharoen, Kato and Uomoto, 2008; Petcherdchoo, 2018) because it has been challenging to build an efficient model to study the corrosion process in repaired structures with comprehensive probabilistic considerations.

1.4.2. Risk assessment

In this approach, a risk matrix is used to quantify the risk associated with structural concrete and prioritize maintenance activities accordingly. Non-destructive testing methods such as half-cell potential surveys and surface resistivity tests are well-established techniques for evaluating the corrosion risk of concrete structures (Assouli and Ballivy, 2008). These methods can be used together to provide an overall assessment of the risk of corrosion initiation (CS, 2004a). In addition, intrusive testing can be undertaken when there is a probable risk of corrosion initiation to establish the depth of chloride-ion penetration and subsequent corrosion risk (Castellote and Andrade, 2001; CS, 2004a) (see Table 1.1).

Table 1.1: Interpretation of chloride-concentration (CS, 2004a)

Threshold level by mass of cement (%)	Interpretation
>1.0	High corrosion risk
0.4 – 1.0	Medium corrosion risk
<0.4	Low corrosion risk

This model provides a simplified approach to optimizing the maintenance of critical components and prioritizing the maintenance of structural networks. However, it is limited in scope because it considers only one component, one failure mode, and one type of uncertainty (the uncertainty of the deterioration process). Other important factors, such as the uncertainty in the repair process, or the effectiveness of different repair strategies, that may affect maintenance management are not considered by this model.

1.4.3. *Markov Chain Models*

Markov models have proven to be useful tools in the maintenance management of corroded concrete structures (Kallen, 2007; Possan and Andrade, 2014; Dizaj, Padgett and Kashani, 2021). In this approach, the condition of the structure is classified into discrete states based on the severity of corrosion. These states could range from undamaged to severely damaged, depending on the extent of corrosion. The Markov models use transition probabilities to estimate the probability of a structure transitioning from one condition state to another. One of the advantages of using Markov models in maintenance management is that they allow prediction of the future condition of the structure based on past conditions and maintenance actions. This information can be used to optimize maintenance strategies, such as determining the appropriate timing and type of maintenance required for various corrosion conditions. While Markov models have proven to be highly effective in many applications, they are not without limitations. For example, they assume that the future state of a system depends only on its current state and not on past states. Moreover, the major drawback of this approach is that reliability is not directly incorporated into structure management (Frangopol, Kong and Gharaibeh, 2001). In the context of maintenance management and optimization of corroded EC structures, Bastidas-Arteaga and Schoefs (2012) conducted a comprehensive study using this probabilistic approach. Therefore, the Markov chain models are not pursued further in this study.

1.4.4. *Bayesian method*

Bayesian method is a type of graphical model that represents the probabilistic relationships between variables in a system and the probability of an event occurring. Bayesian

networks allow for the incorporation of prior knowledge and experience into the model, and they can be used to make predictions about the likelihood of future events (Li and Babu, 2019). This approach is particularly useful when dealing with complex systems that have a large number of variables. By using the model, maintenance activities can be prioritized based on the most critical variables, and the probability of failure can be estimated (Cai *et al.*, 2019; Tran, Bastidas-Arteaga and Aoues, 2020; Imounga *et al.*, 2021). There are some limitations to the use of Bayesian networks in the maintenance management of chloride-polluted concrete structures. For example, the accuracy of the model depends on the quality of the data and the assumptions about the relationships between variables (Wang, 2004; Uusitalo, 2007). In addition, the complexity of the model may make it difficult to interpret and communicate to non-experts. However, it is a promising approach for repair and maintenance management that could be conducted in future research works.

Each management approach brings its own advantages and disadvantages. In general, management systems aim to find an optimal maintenance solution that minimizes costs and environmental impact by maintaining an optimal level of safety, either implicitly or explicitly. Considering the complexity of the topic, the research objectives, and the applicability of possible probability models, this study uses time-dependent reliability analysis to manage and optimize repair strategies for corroded RC structures.

1.5. Methods for reliability analysis

1.5.1. Basic concept for reliability analysis

Reliability analysis is a critical process for evaluating the ability of systems or components to remain safe and functional throughout their life cycle. This information is invaluable for making informed decisions, improving resource management, and conducting sensitivity studies. To determine whether or not a system has failed, engineers use a *performance function* $g(\cdot)$:

$$g(\mathbf{X}) = R(\mathbf{X}) - S(\mathbf{X}) \quad (1.1)$$

This function takes into account the system's resistance and load/requirement, represented by $R(\mathbf{X})$ and $S(\mathbf{X})$, respectively. The vector \mathbf{X} contains the random variables of the problem. The safe region is defined as $\{\mathbf{x}: g(\mathbf{x}) > 0\}$, while the failure region is $\{\mathbf{x}: g(\mathbf{x}) \leq 0\}$, where \mathbf{x} is a realization of \mathbf{X} . To estimate the probability of failure (p_f), engineers use the joint probability density function of \mathbf{X} , $f(\mathbf{x})$, and the following equation:

$$p_f = P[g(\mathbf{X}) \leq 0] = \int_{g(\mathbf{X}) \leq 0} f(\mathbf{x}) d\mathbf{x} \quad (1.2)$$

Two main types of limit states are considered in the design and analysis of structures: *serviceability limit state* (SLS) and *ultimate limit state* (ULS) (ACI 318). The SLS ensures that the structure remains functional and usable under normal conditions and within its intended

service life. The SLS is typically associated with deflection, vibration, cracking, and other deformations that could affect the performance of the structure. The ULS, on the other hand, is concerned with the safety of the structure under extreme and infrequent conditions, such as natural disasters or high-intensity loading. The ULS ensures that the structure cannot collapse or be excessively damaged under these conditions (Wang, 2021).

When it comes to chloride-induced corrosion in RC structures, both SLS and ULS are critical considerations. Chloride ions penetrate the concrete and react with the reinforcing steel, corroding and weakening it. At the SLS, the corrosion-induced cracks could lead to water penetration, which could result in additional corrosion and further deterioration of the structure (Tuutti, 1982; Teplý and Vorechovská, 2012). This could lead to a loss of functionality and serviceability, resulting in costly repairs and maintenance. For the problem treated in this study, the SLS is related to *the probability of corrosion initiation*.

In the case of ULS, loss of steel cross-sectional area due to corrosion could compromise the structural integrity of the RC structure. This could lead to sudden collapse or partial failure when the applied load exceeds the remaining resistance (Tuutti, 1982; Teplý and Vorechovská, 2012). The ULS is related to *the probability of failure* for chloride-induced corrosion in RC structures.

According to Bastidas-Arteaga and Schoefs (2012), maintenance strategies often emphasize preventive rather than corrective measures, hence the probability of failure in Equation (1.2) used in this study is the *probability of corrosion initiation*, which is calculated based on the serviceability limit state.

1.5.2. Reliability methods

Reliability methods are statistical techniques used to analyze the probability of failure or other performance problems in a system or structure. These methods are usually divided into three main categories: simulation-based, approximation-based, and surrogate-based.

1.5.2.1. Simulation-based methods

Simulation-based methods, such as Monte Carlo simulation (MCS) and Latin hypercube sampling (LHS), use computational models to simulate the behavior of a system. These methods generate random samples from input distributions and use them to evaluate the system output. MCS and LHS are widely used simulation-based methods in reliability analysis due to their simplicity and flexibility.

1. Monte Carlo Simulation (MCS): Among conventional probabilistic evaluation methods, MCS is widely used because of its simple computer implementation (Zio, 2013). The advantages of MCS include its flexibility in modeling complex systems, its ability to handle non-normal and non-linear distributions, and its ability to produce accurate results when the

number of trials is sufficiently large. Recently, MCS have been used widely in the evaluation of probabilistic durability of concrete structures. For example, Val and Trapper (2008) employed the MCS to evaluate the initiation time of chloride-induced corrosion. Ma *et al.* (2020) used the MCS to evaluate the probabilistic life prediction of RC structures subjected to seasonal corrosion-fatigue damage. However, the use of MCS can be very time consuming when dealing with complex numerical models, especially in the iterative calculation to predict the time-dependent chloride transport in the repair of RC structures with two different material layers. For this reason, MCS does not seem to be suitable for probabilistic maintenance modeling in this study.

2. Latin Hypercube Sampling (LHS): Latin Hypercube Sampling is a variant of MCS that improves sampling efficiency by reducing the number of trials required. This sampling strategy disperses the sample points more uniformly among all possible values, thus reducing the number of samplings generated with satisfactory result (Olsson, Sandberg and Dahlblom, 2003). Compared to the crude MCS, LHS adequately covers of the domain of variations of the random variables using a stratified sampling strategy. In a wide range of scenarios, LHS has been demonstrated to be more efficient than basic random sampling. Because of its popularity and stability, LHS has been frequently employed in the design of experiments and probabilistic analysis for evaluating the probability of failure (Bastidas-Arteaga *et al.*, 2009; Dastmalchi and Burton, 2021). In this study, LHS is used to perform a probabilistic analysis of maintenance modeling related to one-dimensional chloride penetration due to the ease of use in implementation as well as the affordable time cost of the method. The detail of its implementation is presented in Chapter 3.

1.5.2.2. Approximation-based methods

Approximation-based methods, such as the first-order reliability method (FORM) and the second-order reliability method (SORM), estimate the probability of failure using mathematical approximations, which are suitable for systems with analytical models and require less computational costs than simulation-based methods (Cizelj, Mavko and Riesch-Oppermann, 1994; Choi and Yoo, 2012). FORM and SORM approximate the limit state function using first- and second-order Taylor expansions, respectively. Besides MCS, probabilistic assessments commonly use second moment methods such as FORM /SORM. For instant, Yu, Ning and Li (2017) prove the efficiency of the FORM and SORM methods in predicting the corrosion probability of a RC structure by comparing it with MCS. Notably, most studies using these methods have been performed on simplified models of chloride ingress using analytical solutions based on Fick's diffusion laws (Nogueira, Leonel and Coda, 2012; Saassouh and Lounis, 2012; Yu, Ning and Li, 2017). However, it is well known that FORM and SORM can be inaccurate for probabilistic problems with nonlinear limit state functions. Modeling of

chloride transport in repaired concrete usually involves nonlinear finite element analysis, which generates the performance function in implicit form. Therefore, the FORM and SORM methods are not suitable for probabilistic maintenance modeling in this study.

1.5.2.3. *Surrogate-based methods*

Despite the use of the LHS method, the probabilistic assessment of maintenance models for chloride transport remains a challenge, particularly for high-dimensional problems (2D, 3D), due to significant computational costs. Therefore, surrogate-based methods provide a suitable alternative that is commonly used in engineering and scientific applications to mimic the behavior of the response. These surrogate models are constructed based on limited simulations or experimental data and are computationally efficient, making them ideal for sensitivity analysis, optimization, and uncertainty quantification. In addition, these metamodels are particularly well-suited for reliability analysis, which involves multiple calculations of the performance function (Sudret, 2014; Hariri-Ardebili and Sudret, 2020).

Several techniques have been proposed, including the response surface method (Alibrandi, 2014), artificial neural networks (ANNs) (Izquierdo, Crespo Márquez and Uribebarria, 2019), Kriging (Kaymaz, 2005; Ni *et al.*, 2020), and the polynomial chaos expansion method (Sudret, 2008; Ni *et al.*, 2019; Bastidas-Arteaga *et al.*, 2020). These techniques have been applied in various fields, such as reliability analysis of aircraft structures, stability analysis of ground slopes, optimization design of composite structures, and stochastic dynamic analysis of bridge structures. Although the response surface method is efficient, it has lower accuracy than other techniques. On the other hand, artificial neural networks require a large number of data samples to train the surrogate model. Although the Kriging method is applicable to general time-dependent reliability problems, it shares the drawbacks of the Kriging technique that is limited to low or moderate dimensional problems (Fu *et al.*, 2020; Zhou and Lu, 2020). However in complex engineering applications, time dependency may be characterized through multiple stochastic processes increasing the dimensionality of the problem. Polynomial chaos expansion (PCE) is another popular surrogate modeling technique that can predict the output of the physical model with high accuracy.

The Polynomial Chaos Expansion (PCE) method is preferred to other surrogate modeling techniques because of its ability to capture nonlinear relationships between input variables and output responses without requiring excessive model evaluations, especially for complex systems of maintenance models (Sudret, 2014). In addition, PCE is easy to implement and interpret, making it a popular choice for surrogate modeling. It involves fitting a polynomial function to a set of input-output data, with the degree of the polynomial determining the complexity of the model. PCE has also proved useful in time-invariant structural reliability analysis (Bastidas-

Arteaga *et al.*, 2020; Hariri-Ardebili and Sudret, 2020). One of the advantages of PCE is the ability to quantify the uncertainty associated with the predictions. By passing the uncertainty in the input variables through the surrogate model, PCE can estimate the uncertainty in the output response. By this way, the Parseval-Plancherel theorem can be used to calculate Sobol indices, which allows variance decomposition without running PCE as a surrogate model (Sudret, 2008). Future work will compare the PCE approach with alternative metamodeling methods and the Bayesian approach.

In summary, there are several reliability methods for structural analysis, including simulation-based (MCS, LHS), approximation-based (FORM, SORM), and surrogate-based (PCE, Kriging, ANNs). Each method has its own advantages and disadvantages, and the choice of method depends on the complexity of the problem, the availability of data, and the desired accuracy. Simulation-based methods are computationally expensive but can provide accurate results, while approximation-based methods are less computationally expensive but may have limitations in handling complex problems. Surrogate-based methods offer a balance between accuracy and computational efficiency, but require high-quality training data. Chapter 3 of the study presents detailed applications of the LHS method and the PCE method for the probabilistic assessment of a 1D maintenance model and a 2D maintenance model, respectively, in the context of chloride-induced corrosion in repaired concrete. The selection of the most appropriate method for each model aims to provide a comprehensive and accurate analysis of the maintenance system under study.

1.6. Framework for the proposed maintenance management

The focus of this study is on a management approach that targets specific maritime infrastructure that is vulnerable to chloride intrusion. This approach relies on a time-dependent reliability analysis to determine the maintenance strategy. The main objective is to find the most sustainable solution that takes into account factors such as structural safety, optimization of life cycle costs, and reduction of environmental and social impacts. This section explains various aspects of this strategy, including safety requirements, maintenance schedule and cost considerations, and sustainability.

1.6.1. Safety requirements

To ensure the durability of structures affected by corrosion, it is necessary to establish repair criteria and understand the dynamics of the deterioration process. In this study, decision making is based on the simulation results of chloride concentration in the depth of cover in RC structures repaired by cover replacement. To calculate the probability of failure, a probabilistic model is used based on the coupling of finite element and finite difference methods representing different phenomena affecting chloride penetration into the repaired concrete. Section 1.3

discussed the need for comprehensive maintenance modeling and explained the principles of chloride penetration into repaired concrete.

In maintenance and repair of RC structures, the selection of the repair criterion is a crucial issue that must take into account the agency's maintenance policy, which may include preventive or corrective actions. Studies such as that of Val and Stewart (2003) show that corrosion-induced severe cracking can reduce the capacity of a structure by 10 to 20%, necessitating repair measures such as replacement or addition of reinforcing bars (corrective maintenance) to ensure a minimum level of residual reliability. Therefore, this study focuses on preventive maintenance, in which maintenance and repair work is performed before or as soon as corrosion begins. The goal is to restore the structure to its original condition, minimize the risk of corrosion outbreak, reduce loss of residual strength, and avoid excessive repair costs. Based on that, a proactive repair criterion is derived from the probabilistic approach, where the repair schedule is set in advance based on an acceptable damage threshold or probability of corrosion initiation (Estes and Frangopol, 2001; C. Boschmann Käthler and Angst, 2022).

The choice of a particular repair criterion depends on several factors, including the importance and purpose of the structure or component, its expected service life, and others. This study focuses on the maintenance of transportation infrastructure (i.e., bridges, ports, etc.) particularly components located in harsh marine environments with a high risk of corrosion. Therefore, this study uses the preventive approach to define the repair criterion for maintenance management.

1.6.2. Cost considerations

As mentioned earlier, the management system aims to generate various optimal alternatives for a specific structural component to determine the most appropriate solution. The results of the maintenance model ensure a certain level of safety, which implicitly relates to the repair threshold. After performing the maintenance modeling, a corresponding maintenance plan must be created for each strategy. Thus, the concept of life cycle maintenance cost refers to the total cost of all interventions required to achieve the intended lifetime. The cost is based on a deterministic assessment that takes into account certain parameters such as the cost per unit of repair, the timing of the repair, and the number of repairs (C. Boschmann Käthler and Angst, 2022). Consequently, stakeholders can use this practical and user-friendly method to calculate life-cycle costs, support decision making, and account for the uncertainty associated with maintenance activities.

Besides, for maintenance management of chloride-polluted RC structures, a cost-based risk analysis could be used to calculate expected inspection and repair costs, which depend on repair methods, labor costs, material costs, site setup costs, and expected number of repairs. On the other hand, the costs associated with failure (i.e., damage, cost of living, injuries, delays in use,

etc.) can be calculated based on the probability of failure without repair. Optimized maintenance scheduling could be achieved by minimizing the combined costs of inspection, repair, and failure (Val and Stewart, 2003). Therefore, this approach can help improve the cost effectiveness of a single maintenance strategy and optimize the maintenance schedule based on it. Its detailed application can be found in the study by Bastidas-Arteaga and Schoefs (2012) and is not pursued further in this study.

1.6.3. Sustainability

In recent years, the issue of sustainability has become increasingly important in the construction industry. The environmental impact of the industry is significant, accounting for approximately 40% of global carbon emissions and 50% of global resource consumption (World Green Building Council, 2019). This has led to the development of sustainable building practices that aim to reduce the industry's impact on the environment while improving the social and economic aspects of construction.

The development, maintenance, repair, and rehabilitation of construction projects rely heavily on feasibility, benefit, and cost analyzes. However, in today's world, environmental regulations require integrated planning processes that focus on optimizing resource management, reducing waste production, and minimizing environmental impacts. To achieve these goals, management strategies must prioritize sustainability over cost efficiency.

Despite the importance of sustainable management, there is little literature evaluating the sustainability of maintenance strategies for corroded RC structures. There have been several studies that have calculated the cost of multiple repairs over the life of the structure (Fritz Binder, 2013; Bastidas-Arteaga and Stewart, 2018) or estimated the environmental impact of repair and maintenance activities (Petcherdchoo, 2015). Research efforts have been made to evaluate the environmental impact of concrete with a lower content of cementitious material (Mehta, 2004) and to compare the sustainability of different engineering solutions used to address the same problem (Struble and Godfrey, 2004). However, there has not been a comprehensive study that focuses on defining different repair strategies, evaluating their effectiveness, and thoroughly comparing these solutions based on economic and environmental considerations, such as maintenance cost, CO₂ emission and waste generation.

Maintenance strategies can have complex, interrelated impacts on economic, environmental, and social factors, making it difficult to assess their sustainability. To address this challenge, researchers have developed multi-objective optimization methods to find solutions that minimize both costs and environmental impacts. This study presents a method for evaluating the sustainability of maintenance strategies that uses multi-criteria optimization to find solutions that meet multiple criteria, including economic and environmental factors. This approach

provides a more holistic perspective on maintenance strategies and enables decision makers to make informed choices that balance competing objectives and promote sustainability. Methodology of selecting a sustainable maintenance strategy and its application in some case studies will be presented in Chapter 4.

1.7. Conclusions

1. The first part of this chapter discusses the different phases of corrosion in RC structures that can significantly shorten their service life and compromise their structural integrity. Phase 1 is the corrosion initiation phase, characterized by the beginning of corrosion activity on the reinforcing steel. Phase 2 is the corrosion propagation phase that occurs after the initiation phase and can lead to cracking, delamination and spalling in the concrete cover, then compromise the structural capacity. Effective corrosion control and maintenance strategies must be designed to address both phases of the corrosion process. Especially, early detection and intervention are critical to prevent the propagation phase of corrosion.
2. The cover replacement method is a common and effective repair strategy for chloride-polluted RC structures. However, modeling chloride ion penetration through two concrete layers and their interface in the cover rebuilding technique is complex due to the differences in chloride concentrations and properties within each layer. Recent studies have focused on modeling chloride transport in concrete structures after repair, and on methods to predict repair times during the life of a structure, but still had some limitations. This study proposes a comprehensive repair model that considers the complex process of chloride transport in repaired concrete and simulates the performance of different repair strategies. This model considers factors such as the time-varying chloride diffusion coefficients of original and repair concrete, chloride binding capacity, concrete ageing, weather conditions, two-dimensional chloride penetration, and other parameters associated with the repair process.
3. Maintenance management of RC structures susceptible to corrosion can be approached using various probabilistic methods, such as time-dependent reliability analysis, risk analysis, Markov chain models, and Bayesian networks. Each of these methods has its own strengths and weaknesses, making them suitable for different problems. In this study, time-dependent reliability analysis is used to predict the lifetime of RC structures and make informed repair decisions. The proposed repair model enables the implementation of reliability analysis to manage maintenance activities by predicting the probability of corrosion initiation and determining repair time intervals based on repair criteria.

Among the reliability methods, LHS and a surrogate-based method are selected to perform probabilistic assessment of maintenance modeling for RC structures subjected to one- and two-dimensional chloride attack. The choice of method depends on the specific application and data characteristics. For the surrogate-based method, the PCE is chosen due to its ability to capture nonlinear relationships, ease of implementation, and ability to quantify uncertainty. PCE has also proved useful in time-invariant structural reliability analysis with its computational efficiency and high accuracy.

4. Finally, a suitable maintenance management strategy is discussed to formulate sustainable repair solutions, taking into account factors such as structural safety, life-cycle costs, and environmental and social impacts. To ensure the durability of RC structures exposed to chloride-induced corrosion, it is critical to establish criteria for preventive repair and to understand the kinematics of chloride penetration into the repaired concrete through the cover rebuilding technique. When considering cost, the concept of life-cycle cost should consider not only the initial construction cost, but also the cost of all repairs required to achieve the designed service life. Finally, sustainable criteria should be included in the maintenance management, such as environmental impacts of CO₂ emission and waste generation. There is a gap in the scientific literature regarding a comprehensive study that focuses on formulating different repair strategies, evaluating their effectiveness, and comparing these solutions based on economic and environmental considerations, including but not limited to maintenance costs, CO₂ emissions, and waste generation. To achieve this, a multi-objective optimization method is proposed to select repair solutions that meet multiple criteria to minimize costs and environmental impacts in repair and maintenance activities.

Chapter 2 *Modeling Chloride Ingress into Repaired Concrete*

2.1. Introduction

Generally, the service life of RC structures is assumed between 50 and 100 years. However, in chloride-contaminated environments, the deterioration of infrastructure may occur after 20 to 30 years (Zhou *et al.*, 2015). Hence, the inspection and maintenance are essential to ensure an optimal level of serviceability and safety during its lifespan. In the context that maintenance strategies often emphasize preventive rather than corrective measures, it is generally accepted that the optimal time for repairs is before the corrosion initiation phase is complete when the concentration of chlorides near the reinforcement reaches the critical threshold and the steel's protective oxide domain is disrupted (Bastidas-Arteaga and Schoefs, 2012). Rapid progression of corrosion can result in shortening the phase of corrosion propagation, leaving little time for safe repairs.

In this context, the repair method of “cover rebuilding” is widely used in practice, where the chloride-contaminated concrete is removed and the structure is rehabilitated by installing new concrete or repair mortar without damaging the reinforcing bars. The new material is usually placed as wet shotcrete, dry shotcrete, formed concrete, or by manual repair (BSEN 1504-10:2017). Some recent research (Song *et al.*, 2009; Soleimani *et al.*, 2010; Petcherdchoo, 2018) provide models for simulating chloride transport in new/repared materials. These studies have some limitations since they neglect the influence of temperature, humidity, chloride binding, and the decrease of chloride diffusivity with concrete age as well as the two dimensional flow of chlorides. Consequently, it is necessary to establish a comprehensive model that can bridge those gaps. In addition, maintaining safety requirements may increase costs, hence proper repair strategies can save a considerable amount of expenses for the owner. Therefore, this chapter aims to develop a comprehensive maintenance model that can simulate the chloride transport in

repaired RC structures and evaluate the efficiency of different repair strategies of cover rebuilding method for the long-term durability of RC structures.

The objectives of this chapter are:

1. to describe the mechanism of chloride transport in RC structures without/with repairs and formulate a maintenance model;
2. to propose and implement a numerical solution to the problem;
3. to investigate the influence of different repair strategies on the effectiveness of cover rebuilding; and
4. investigate the effects of different environmental conditions at a given location on the performance of the proposed maintenance model.

Section 2.2 presents the governing equations for the deterioration model used in this study, then perform the model validation. On that basis, section 2.3 develops the maintenance model in RC subjected to one- and two-dimensional chloride penetration. Finally, sections 2.4, 2.5 and 2.6 analyse three case studies to illustrate the proposed repair model and evaluate the influence of some main parameters on the repair effectiveness.

2.2. Deterioration model

2.2.1. Governing equations for chloride ingress model

Chloride penetration into marine RC structures is typically governed by chloride diffusion, convection, adsorption, permeation, and chloride binding, among other mechanisms. However, it has been proposed that Fick's second law of diffusion serves as the main driver to characterize the distribution of chloride content inside concrete (Tuutti, 1982). The simplification of the chloride transport mechanism to a diffusion-based model is reasonable and acceptable under the condition that RC structures operate in a marine environment that can generally be considered saturated in some exposure zones, e.g., atmospheric or submerged zones (Wang, Gong and Wu, 2019; Li *et al.*, 2022; Zacchei and Bastidas-Arteaga, 2022b). In fact, diffusion-based model allows us to develop a maintenance framework to simulate multiple cover rebuilding repairs with reasonable technical difficulty and calculation time which is main aspect for probabilistic modeling (Chapter 3). Under these assumptions, the main governing equation for chloride transport in concrete in one direction is presented below:

$$\frac{\partial C_{fc}}{\partial t} = D_c^*(x, t) \frac{\partial^2 C_{fc}}{\partial x^2} \quad (2.1)$$

For two-dimensional flow of chlorides in concrete, Eq. (2.1) becomes:

$$\frac{\partial C_{fc}}{\partial t} = D_c^*(x, y, t) \left(\frac{\partial^2 C_{fc}}{\partial x^2} + \frac{\partial^2 C_{fc}}{\partial y^2} \right) \quad (2.2)$$

where C_{fc} (kg/m^3 concrete) is the chloride concentration at point (x, y) in a concrete cross-section at exposure time t (s), D_c^* (m^2/s) represents the apparent chloride diffusion coefficient, x (m) and y (m) are the distances from the exposure surface in the X and Y directions, respectively. Eq. (2.2) could be solved by combining finite or semi-infinite boundary conditions, where the boundary conditions or the flux of chlorides crossing the concrete surface J_c^s ($\text{kg}/(\text{m}^2 \cdot \text{s})$), is expressed as:

$$J_c^s = B_c (C_{fc}^s - C_{evn}) \quad (2.3)$$

where B_c (m/s) is the surface chloride transfer coefficient, C_{fc}^s (kg/m^3) is the value of chloride content at the concrete exposed surface, and C_{evn} (kg/m^3) is the chloride concentration in the surrounding environment. D_c^* (m^2/s) in Eq. (2.2) is estimated as:

$$D_c^* = \frac{D_c}{1 + \left(\frac{1}{w_e}\right) \left(\frac{\partial C_{bc}}{\partial C_{fc}}\right)} \quad (2.4)$$

where w_e is the evaporable water content. The relationship between free chloride and bound chlorides, $\partial C_{bc}/\partial C_{fc}$ is defined as chloride binding capacity, which can be formulated by the Langmuir binding isotherm:

$$C_{bc} = \frac{\alpha C_{fc}}{1 + \beta C_{fc}} \quad (2.5)$$

where α and β are binding constants obtained experimentally. The time-dependent chloride diffusion coefficient D_c in Eq. (2.4) is defined as:

$$D_c = D_{c,ref} f_1(T) f_2(t) f_3(h) \quad (2.6)$$

where $D_{c,ref}$ is a diffusion coefficient determined at standard conditions. $f_1(T)$ is a factor to account for temperature effects:

$$f_1(T) = \exp \left[\frac{U_c}{R} \left(\frac{1}{T_{ref}} - \frac{1}{T} \right) \right] \quad (2.7)$$

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

$$f_2(t) = \left(\frac{t_{ref}}{t} \right)^{m_c} \quad (2.8)$$

where m_c is the aging factor and t_{ref} is the time at which $D_{c,ref}$ is measured. $f_3(h)$ accounts for variations in relative humidity:

$$f_3(h) = \left[1 + \frac{(1-h)^4}{(1-h_c)^4} \right]^{-1} \quad (2.9)$$

where h is the relative humidity inside the concrete that depends on external environmental conditions, and h_c is the relative humidity at which D_c drops halfway between its maximum and minimum values. Previous studies proposed deterministic data to present those parameters, such as B_c , w_e , α , β , D_{ref} , U_c , R , T_{ref} , m_c , t_{ref} , and h_c (Baant and Najjar, 1971; Page, Short and El Tarras, 1981; Anna V. Saelta, 1993; Luping and Nilsson, 1993; L. O. Nilsson, 1994; Xi, Bažant and Jennings, 1994; Glass and Buenfeld, 2000; Martín-Pérez, Pantazopoulou and Thomas, 2001; Val, 2006)

2.2.2. Numerical modeling of chloride ingress

In this section, numerical modeling of chloride transport in two-dimensional (2D) directions is presented. It is noted that the numerical solution for one-dimensional (1D) chloride ingress is performed in a similar but simpler manner, with only one concrete surface exposed to chloride attack.

As shown in section 2.2.1, Eqs. (2.1)-(2.9) are used to characterize the process of chloride ingress in RC structures. The partial differential equations are solved in space and time using a two-dimension (2D) finite element model and finite difference direct integration method developed in FreeFEM++ (Hecht, 2012). In this study, partial scale modeling is used to build a truncated model for a concrete cover region in symmetrical boundary conditions (Figure 2.1) (Martín-Pérez, Pantazopoulou and Thomas, 2001; Bastidas-Arteaga *et al.*, 2011).

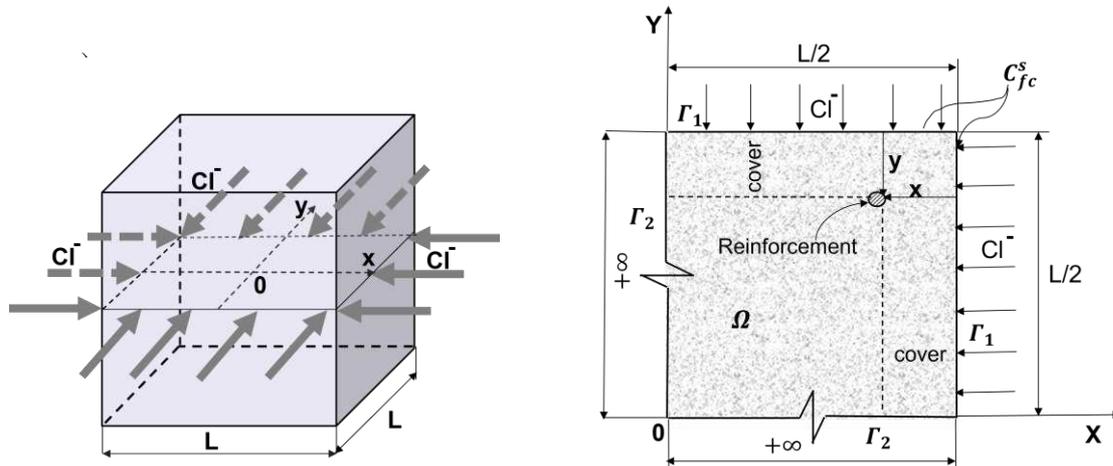


Figure 2.1: (a) 2D chloride ingress area of concrete to be modeled, (b) Boundary conditions of a truncated chloride ingress model

Figure 2.1 shows the 2D chloride ingress area of concrete to be modeled including the boundary conditions of the truncated chloride ingress model. In this study, to solve Eqs. (2.1)-(2.2) with non-constant chloride diffusion coefficient, boundary conditions and initial conditions for 2D chloride ingress model are defined respectively, as below:

$$\begin{cases} C_{fc}(x,y,t)=C_{fc}^s, & \text{for } (x,y) \in \Gamma_1, t>0 \\ C_{fc}(x,y,t)=C_0, & \text{for } (x,y) \in \Gamma_2, t>0 \end{cases} \quad (2.10)$$

$$C_{fc}(x,y,t) = C_0, \text{ for } (x,y) \in \Omega, t = 0 \quad (2.11)$$

where C_{fc}^s represents the chloride concentration of the exposed concrete, which rises fast in the early stages before stabilizing in the latter stages (Petcherdchoo and Chindaprasirt, 2019). Moreover, in this study, Eqs. (2.1)-(2.2) are used to model chloride transport in submerged or splash zone, where C_{fc}^s can be considered as a constant corresponding to an average chloride concentration during the structure lifetime or the chloride concentration of seawater (Amey, 1998). Γ_1 is the boundary of exposure surface, Γ_2 contains the remaining parts of the boundary within concrete. C_0 is the initial chloride concentration within concrete and Ω denotes the domain of 2D chloride diffusion (as shown in Figure 2.1b).

2.2.3. Model validation

For model validation, the values for the chloride ingress model parameters were selected (see Table 2.1). These values correspond to the concrete mix of ordinary Portland cement (OPC) with 8% C3A and a w/c ratio of 0.5.

Table 2.1: Values selected for the model parameters of the chloride penetration model

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

For the 1D diffusion model, a comparative study is performed between the results obtained with: (1) the code developed in FreeFem++ and (2) the analytical solution to the problem. The closed-form analytical solution based on the error function is formulated as follows:

$$C_{fc}(x,t) = C_{fc}^s \left[1 - \operatorname{erf} \left(\frac{x}{2\sqrt{D_{fc}t}} \right) \right] \quad (2.12)$$

where $C_{fc}(x, t)$ is the free chloride concentration at depth x after time t , $\text{erf}()$ is the error function. Figure 2.2 shows the configuration of the case studied.

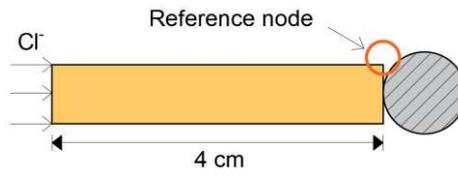


Figure 2.2: Configuration of the case studies

The results of this analysis focus on the reference node, which is located at a depth of 4 cm. Some assumptions are made as follows:

- the effective chloride diffusion coefficient is defined as constant in the analytical solution: $D_c = 2.10^{-12} \text{ m}^2/\text{s}$;
- the temperature and humidity are considered constant in the model, $T = 15 \text{ }^\circ\text{C}$, $h = 0.7$, the model also takes into account the effects of concrete aging and chloride binding; and
- the chloride concentration at the surface is constant: $C_{fc}^s = 4 \text{ kg/m}^3$

Figure 2.3 shows the chloride concentration at the studied depth for one dimensional chloride ingress. It can be observed that there is a difference between the numerical modeling and the analytical solution after 20 years of exposure, but it is not significant. The FreeFem++ program underestimates the chloride evolution in the later phase of a structure's service life. This is because the effects of temperature, moisture, concrete aging and chloride binding are neglected in the analytical formulation. In particular, the aging of concrete, in relation to chloride ingress, can result in a substantial decrease in the apparent diffusion coefficient over time. Therefore, this comparison could be considered for the validation of the model of one-dimensional chloride ingress.

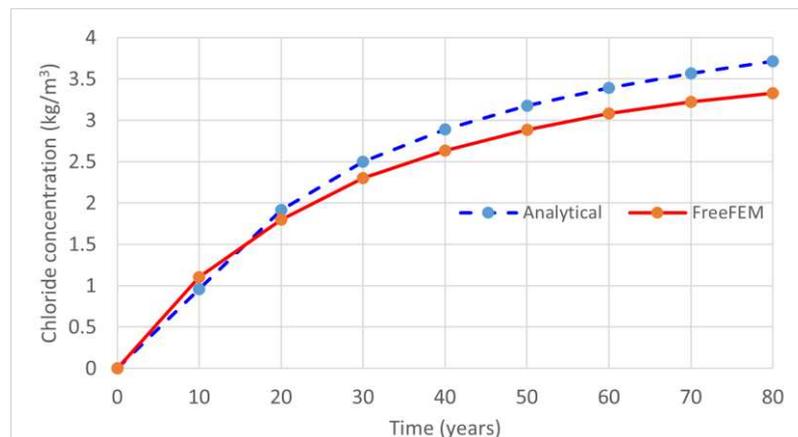


Figure 2.3: Model validation for one-dimensional chloride ingress

For 2D model, the numerical simulation presented in section 2.2.2 is validated by comparison to the 2D chloride diffusion experiments conducted by Yu *et al.* (2021). The values selected in the model apply to a concrete mix of OPC and a w/c ratio of 0.5 (Table 2.1). In their work, 100 mm x 100 mm x 400 mm concrete cubes were immersed in an artificially saturated marine environment. After a curing period of 28 days at the temperature of 20°C, four surfaces of the concrete were coated with epoxy resin, allowing chloride ions from the environment to penetrate onto the remaining two adjacent surfaces. The concrete samples were then placed in an automated simulation system and exposed to synthetic seawater. Experimental data on the distribution of chloride concentration were obtained by examining the chloride content from the corner and edge areas of the samples after 70 and 180 days of exposure to the artificial marine environment. In order to ascertain the conditions in artificially simulated marine environment, a review of relevant experiments (Wang *et al.*, 2018; Wang, Gong and Wu, 2019; Wu *et al.*, 2020) was conducted to determine ambient temperature and relative humidity during testing. Generally, the samples are placed in controlled environmental conditions with a temperature of approximately 20°C and a relative humidity averaging 70% (ranging from 60-80%). Since the exact value is unknown, to validate the model, three different ambient temperatures (18°C, 20°C, and 23°C) and an average humidity value of 0.7 were utilized. The simulated chloride concentration was compared to the tested chloride results after 70 days (designated as M70-T18, M70-T20, M70-T23) and 180 days (designated as M180-T18, M180-T20, M180-T23) as depicted in Figure 2.4. Even if these results are still based on some assumptions, the agreement between the modeled chloride profile curves and the experimental chloride measurements indicates the validity of the 2D chloride ingress model.

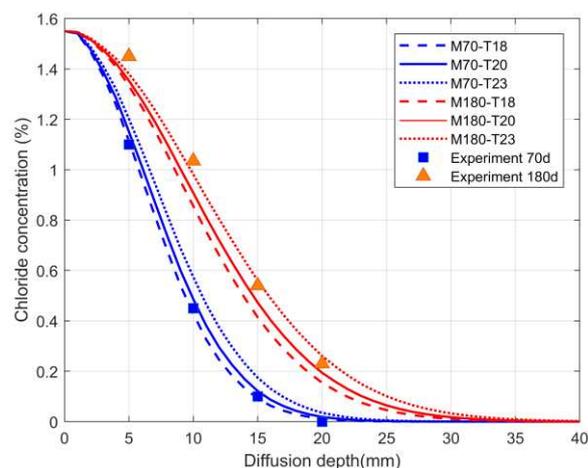


Figure 2.4: Chloride diffusion model validation with data taken from (Yu *et al.*, 2021)

The experimental work of Roy, Liam Kok Chye and Northwood (1993) is also examined to validate the results of the numerical model. The concrete specimens were rectangular prisms with dimensions 100 mm x 100 mm x 500 mm. Data were obtained from the chloride

penetration test of standard concrete after an 80-week exposure period in a submerged zone at a temperature of 20°C and a relative humidity of 0.75. Figure 2.5 shows the trend of free chloride distribution from FreeFEM++ modeling with the parameters used in the simulation. The comparison of results obtained with the numerical modeling with previous experiments confirms the prediction accuracy of the model for 2D chloride ingress modeling.

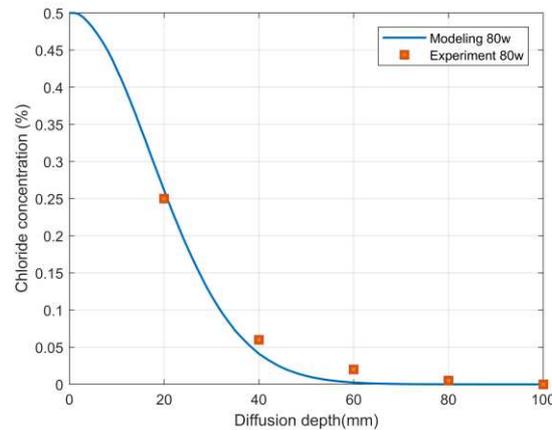


Figure 2.5: Chloride diffusion model validation with data taken from (Roy, Liam Kok Chye and Northwood, 1993)

This model verification is essential for the development of a maintenance model that incorporates the influence of concrete replacement repairs, where the diffusion of chloride ions remains the primary control mechanism within the concrete. The chloride ingress model considering the repair will be presented in the next section.

2.3. Chloride ingress model considering the repair/maintenance actions

Modeling chloride penetration in concrete after repair is challenging because of the interaction of the two main sources of chloride ions: chloride concentration in the surrounding environment and residual chloride ions in the remaining concrete. Previous studies concluded that both macrocell corrosion (between two materials) and microcell corrosion (within a homogeneous material) may occur at the same time in repaired concrete (M. Raupach, 2006; Soleimani *et al.*, 2010). This macrocell corrosion is caused by the differences in existing and new concrete causing: 1) an electrochemical potential difference, and 2) inhomogeneity of chloride concentration in the materials and then 3) the redistribution of chloride ions from the aged concrete into the new concrete after repair. These consequences were studied in existing experimental and in situ observations (M. Raupach, 2006; Shi *et al.*, 2020). Also, both macrocell and microcell corruptions were observed in existing numerical models for the simulation of chloride transport in repaired structures (Song *et al.*, 2009; Rahimi *et al.*, 2014, Petcherdchoo, 2018).

2.3.1. One-dimensional maintenance model

In this section, a detailed description of the chloride transport in repaired RC structures as well as the numerical approach considering a spatially domain-dependent and time-dependent diffusion coefficient in two material domains are illustrated. The one dimensional maintenance model is considered first. Figure 2.6a illustrates the chloride penetration immediately before repair. At this stage, the chloride concentration abutting the concrete surface is the main source of chloride ions. The initial chloride content is assumed equal to 0, $(C_{fc})_0 = 0$ for all volume of concrete and $D_c^*(t)$ is defined as the chloride diffusion coefficient of the original concrete.

Figure 2.6b shows the chloride profile immediately after cover rebuilding repair. At repair time t_i , the chloride-contaminated concrete cover is removed to the depth X_p . X_p is defined as repaired depth or thickness of replaced concrete, which is not necessarily the same depth as concrete cover, X_c . In this case $X_p > X_c$. Consequently, the amount of chloride ions between the concrete surface and the repair depth is removed, and a chloride-free, cement-based repair material is applied as a repair domain. The repaired concrete and the unrepaired remaining concrete domains are represented by Ω_1 and Ω_2 , respectively. At this stage, only Ω_1 is exposed to the surface chloride concentration in the environment and will be subjected to chloride ingress first before reaching the original concrete. The main diffusion mechanism of chloride penetration in the repaired material is also modeled by Eqs. (2.1)-(2.9), where the boundary condition is still the flux of chlorides crossing the concrete surface, but the initial conditions in Ω_1 and Ω_2 are different. While the initial chloride content in the repaired material domain is zero, $(C_{fc})_{01} = 0$, this concentration in the old concrete corresponds to the residual chloride ions, which were not removed after repair. Additionally, the diffusion coefficient of the repair concrete, D_{rep}^* , and model parameters related to material properties are updated to reflect the new repair material and the aging of the new concrete based on Eqs. (2.4)-(2.9), while the diffusion process in the old concrete is governed by the original model parameters. It should be noted that in this case no new boundary conditions are imposed in the interface and that the domain decomposition technique in FreeFem++ can help to solve numerical problems of a concrete with two domains and in the interface (more details in Section 2.3.3).

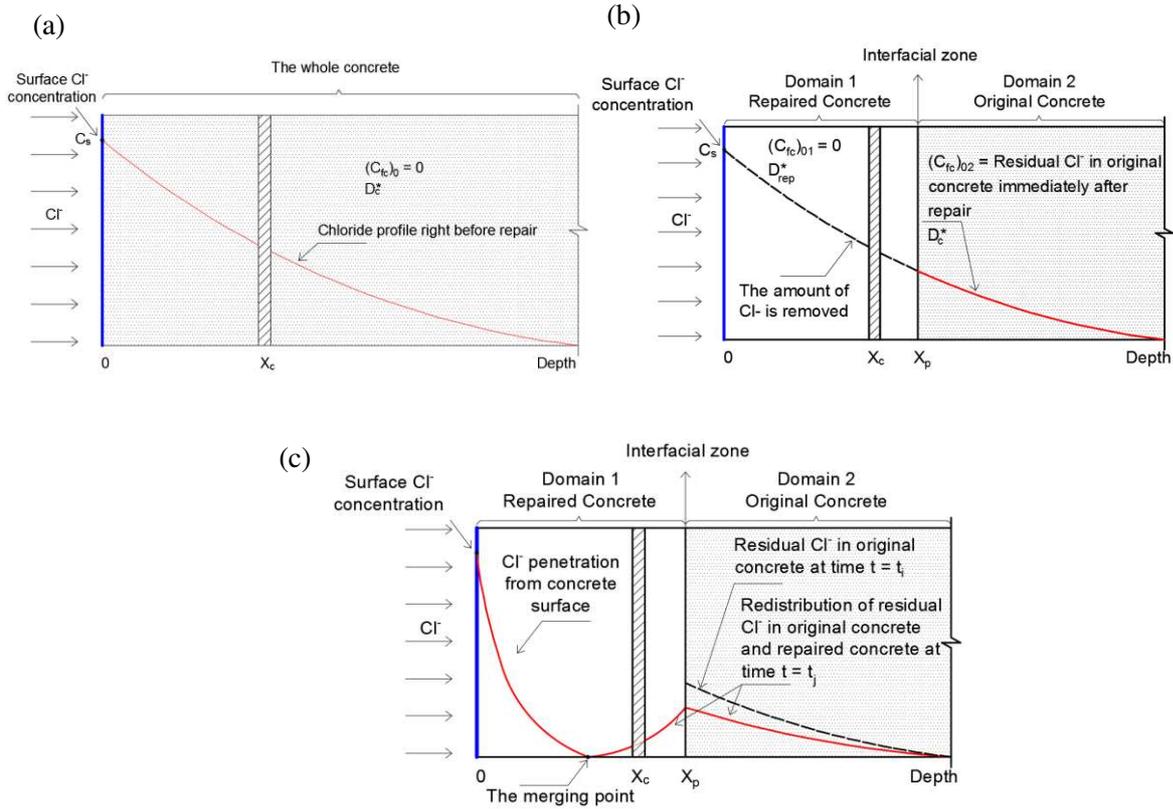


Figure 2.6: (a) Chloride profile immediately before repair, (b) Chloride profile immediately after repair at $t = t_i$, (c) Chloride profile after repair at $t = t_j$ ($t_j > t_i$)

After repair, there are three principal stages of chloride distribution as shown in Figure 2.6c. Firstly, the chloride ions remaining in Ω_2 will be redistributed in the aged concrete domain itself. Secondly, these residual chlorides will also diffuse across the transition zone and into the new, chloride free concrete domain which provides low chloride diffusion resistance immediately following repair. Thirdly, at time t_j (with $t_j > t_i$) the resulting diffusion of chloride ions in the original and repaired concrete is a combination of chloride ingress from both the exposed concrete surface and the chloride redistribution inside the concrete. Due to the difference of the diffusion coefficients in two domains Ω_1 and Ω_2 , the problem will involve a spatially domain-dependent diffusion coefficient after repair $D_{ar}^*(x, t)$:

$$D_{ar}^*(x, t) = \begin{cases} D_{rep}^*(t_{rp}) & \text{if } 0 < x < X_p \text{ (Domain 1)} \\ D_0^*(t) & \text{if } x \geq X_p \text{ (Domain 2)} \end{cases} \quad (2.13)$$

where t_{rp} is the exposure time for the new repair concrete domain that is calculated from the moment the new material is installed to replace the chloride-contaminated concrete part ($t > t_{rp}$). Consequently, the partial differential equation (PDE) used in this stage based on the Fick's diffusion law can be written as:

$$\frac{\partial C_{fc}}{\partial x} = \frac{\partial}{\partial x} D_{ar}^*(x, t) \frac{\partial C_{fc}}{\partial x} \quad (2.14)$$

The issues for solving the PDE occur again and can be further complicated by multiple repairs during the RC structure service life.

2.3.2. Two-dimensional maintenance model

The numerical approach to develop the 2D maintenance model is similar to the development of the 1D model presented above, which considers a time and space-domain dependent diffusion coefficient in two domains of material corresponding to the old and the repaired concrete. Figure 2.7 illustrates the evolution of chloride transport in repaired RC structures in detail, with color iso-contour lines. In this case, the entire concrete cover is replaced.

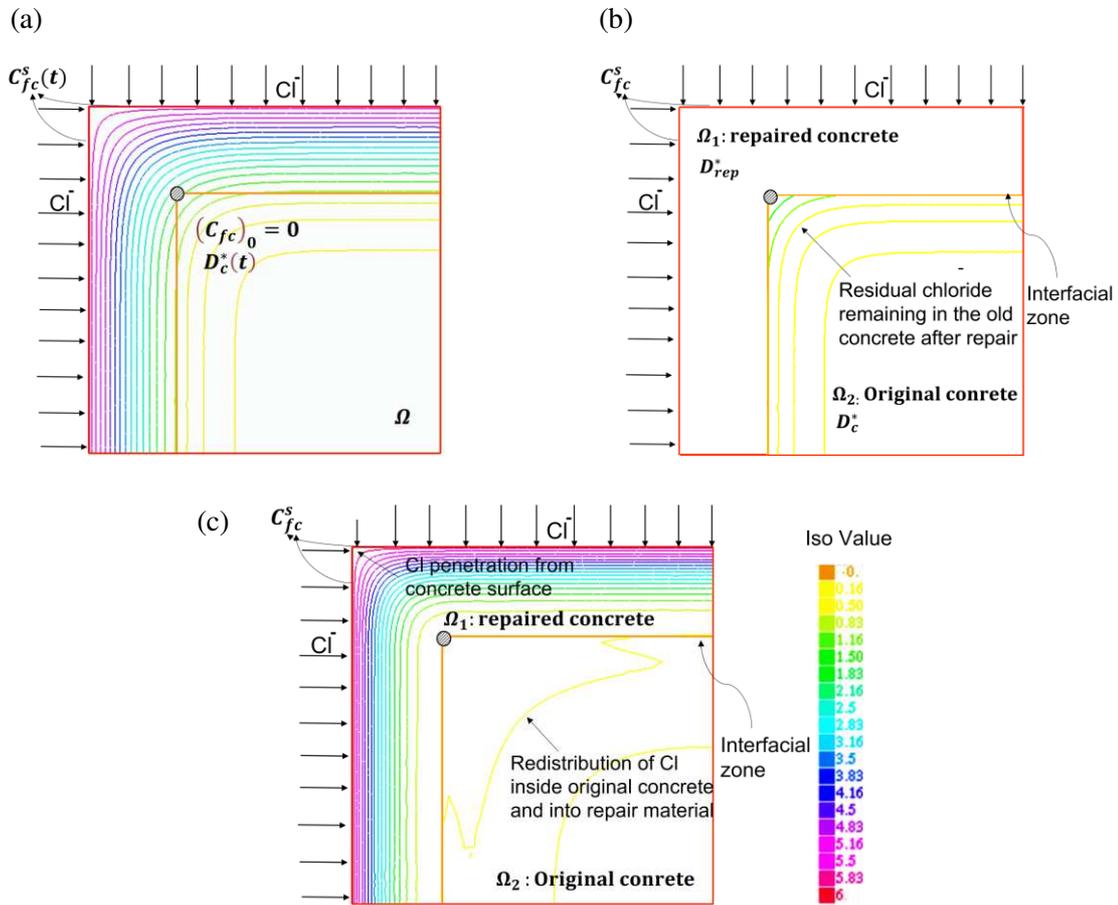


Figure 2.7: (a) Chloride profile immediately before repair (b) Chloride profile immediately after repair (c) Chloride profile after repair

At this point, a spatial domain-dependent diffusion coefficient $D_{ar}^*(x, y, t)$ and the PDE are defined as follows, respectively:

$$D_{ar}^*(x, y, t) = \begin{cases} D_{rep}^*(t_{rp}) & \text{if } (x, y) \in \Omega_1 \\ D_c^*(t) & \text{if } (x, y) \in \Omega_2 \end{cases} \quad (2.15)$$

$$\frac{\partial C_{fc}}{\partial t} = \left(\frac{\partial}{\partial x} D_{ar}^*(x, y, t) \frac{\partial C_{fc}}{\partial x} + \frac{\partial}{\partial y} D_{ar}^*(x, y, t) \frac{\partial C_{fc}}{\partial y} \right) \quad (2.16)$$

2.3.3. Numerical solution and modeling process in repaired concrete in FreeFem++

The FreeFEM++ software was utilized to address the issue of chloride diffusion modeling in an existing and repaired RC structure (Hecht, 2012). FreeFEM++ is a free and open-source software that solves partial differential equations (PDEs) in 1D, 2D, and 3D boundary domains for nonlinear multiphysics systems. The software features a rapid interpolation procedure and a language for data manipulation across multiple meshes, reducing the number of source code lines required. The mesh is automatically generated in FreeFEM++, and the domain geometry is described through parametric equations of boundary segments.

The FreeFEM++ tool is suitable for predicting chloride penetration in concrete with regular repairs due to its use of independently measurable data as input, such as material properties and exposure environment, and its solution of the time evolution problem of chloride diffusion through the finite element method in space and finite differences in time. The tool also allows to consider the nonlinear binding capacity between bound and free chlorides through a fast iterative procedure. Additionally, the tool can solve the problem of a two-domain concrete repair system using a domain decomposition technique, including iterative sub-structuring and connecting subdomains through interfacial conditions.

The modeling process of chloride diffusion in repaired concrete is shown in Figure 2.8 as a flowchart of the numerical program developed in the FreeFEM++ software. The numerical structure must first be established through the formation of two subdomains, depending on the chosen repair strategy. The repair material and remaining concrete domains constitute the entirety of the concrete element, and the chloride concentration at critical locations is monitored. When the concentration reaches a repair criterion, which could be a threshold value for corrosion initiation or a pre-determined repair time, the repair is defined, and the repair material is considered with updated parameters. The calculation of a new service life for the structure is performed, taking into account the spatially domain-dependent and time-dependent diffusion coefficients of the two concrete domains. This process is repeated until the lifespan of the structure is reached, and the maintenance schedule corresponding to the repair strategy is determined through the timing of repairs.

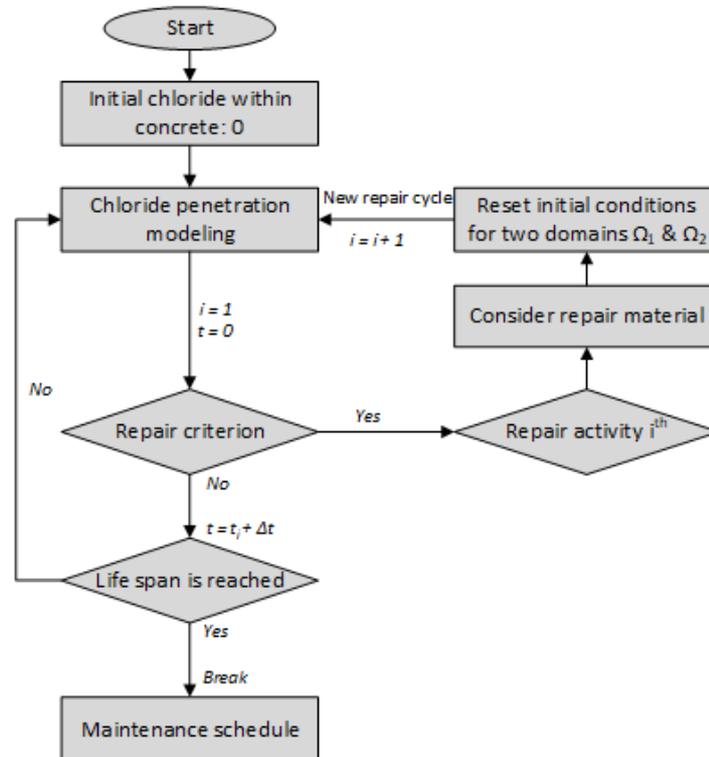


Figure 2.8: Modeling process of chloride ingress in repaired concrete

2.4. Case study 1: Deterministic 1D model of chloride ingress in repaired concrete

2.4.1. Problem description

The main objective of this case study is to illustrate a 1D maintenance model, that allows the application of different repair strategies to ensure the durability of the structure. A deterministic analysis of chloride ingress in repaired concrete structures in extreme marine environments by considering time-dependent variation of diffusion coefficient, chloride binding, average temperature and humidity, and constant surface chloride concentration, is illustrated in this example. The deterministic analysis implements the approach to model the stages of chloride diffusion in repaired concrete described in section 2.3.1 and provides a baseline comparison of model responses to different repair strategies. The following assumptions are made:

- Chloride flow is a one-dimensional process (e.g., concrete slab).
- The average temperature $T = 15^{\circ}\text{C}$; the average humidity $h = 0.7$.
- The structure is exposed to an extreme marine environment. The chloride surface concentration is time-invariant at the value of $C_{fc}^S = 6 \text{ kg/m}^3$.
- The concrete contains 400 kg/m^3 of Ordinary Portland Cement (OPC) with 8% C3A and a water/cement ratio of 0.5.
- The depth of concrete cover X_c is 50 mm.

- The service life of the structure is 70 years.
- Time of corrosion initiation is determined when chloride content reaches the critical chloride value $C_{crit} = 2 \text{ kg/m}^3$ (0.5% of cement mass) at the concrete cover depth.
- The quality and properties of repair material are the same as original concrete. And the bond strength between repair material and old concrete is in good condition.
- Values chosen for the model parameters in this example are listed in Table 2.1 in section 2.2.3. These model parameters were selected to ensure that the structure will be repaired several times during its lifespan for illustrative purposes.

2.4.2. Maintenance model and results

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

2.4.2.1. Corrective repair strategy

The corrective repair strategy supposes that a chloride-contaminated concrete domain is removed each time the chloride concentration at the cover thickness reaches a critical value (C_{crit}). After repair the domain is supposed be replaced with a similar material as the original one. In the following example, this maintenance model is used to calculate these time intervals between repairs following the corrective repair strategy. The calculation further demonstrates how the depth of each repair affects the calculated repair intervals. Two concrete repair depths (X_{p1}, X_{p2}) are chosen; the first repair depth, X_{p1} , is equal to the existing cover depth ($X_{p1} = X_c = 50 \text{ mm}$), and the second repair depth, X_{p2} , is 60 mm ($X_{p2} = 60 \text{ mm}$), which is deeper than the existing concrete cover domain. The replacement material is assumed to have an initial chloride concentration of 0 kg/m^3 at the time of each repair. Although the repaired concrete is assumed to get the same quality as the old concrete, the diffusion coefficient in each concrete domain is different due to the aging factor $f_2(t)$ and the binding capacity. The diffusion coefficient in the original concrete, D_c^* , decreases with initial aging, while D_{rep}^* assumes its initial value when the material is put into service. Figures 2.9a, b, c illustrate the chloride diffusion versus depth in a concrete structure before and after repair with X_{p1} and X_{p2} .

Figure 2.9a shows the progress of chloride penetration until C_{crit} is reached in year 24. Concrete repair is assumed to occur in that year. In Figures 2.9b and 2.9c, the residual chloride concentration after repair is shown in year 24 (after repair) curve and the progress of concrete

penetration after repair is shown from year 25-40. In year 25, chloride penetration in the new concrete has two sources: the concrete surface exposed to the environment and, after repair, the residual chloride in the aged concrete domain. After repair, residual chloride in the aged concrete redistributes between the abutting domains until reaching an equilibrium between years 30-35. In the following years, only environmental chloride concentration contributes to chloride penetration

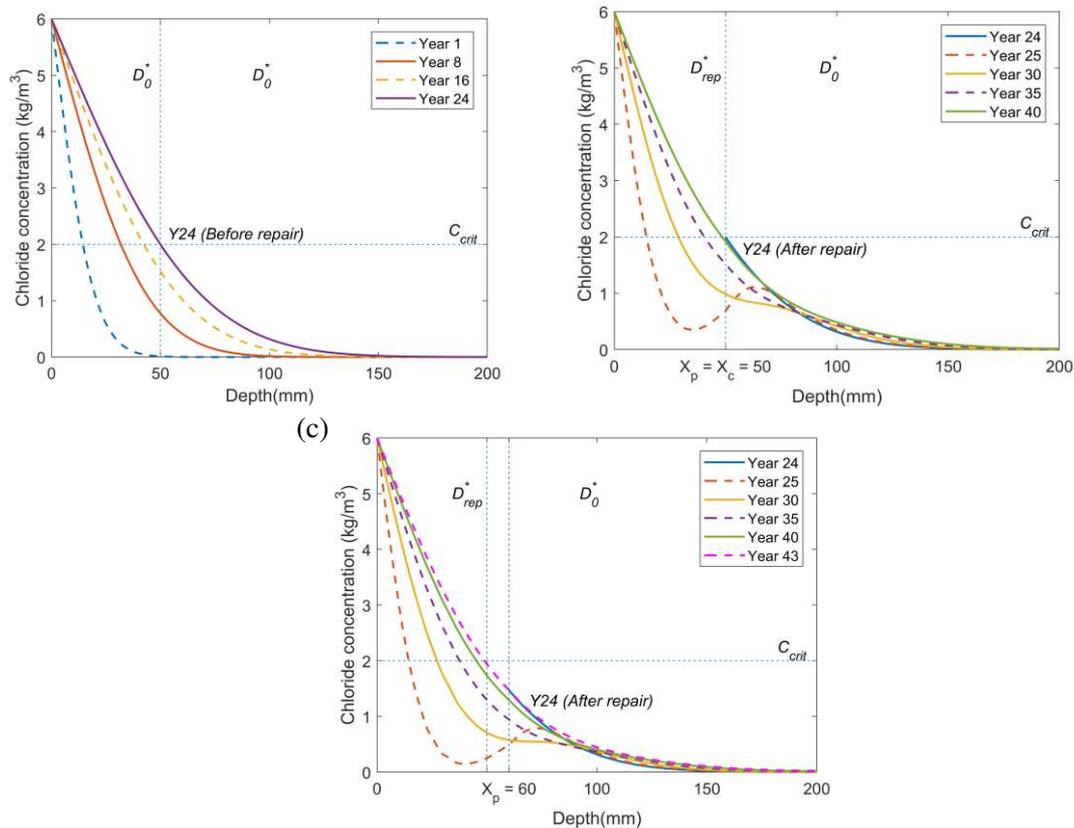


Figure 2.9: Chloride profiles: (a) before repair, (b) after repair with X_{p1} , (c) after repair with X_{p2}

These simulation results are physically compatible with the mechanism of chloride transport in repaired concrete structures analyzed in section 2.3.1, including chloride diffusion from two sources in the after-repair condition. The example demonstrates that the application of cover rebuilding reduces the critical chloride concentration at the reinforcement depth and thereby delay corrosion initiation from concentration above C_{crit} .

In Figure 2.10, it is observed that the two repair depths have similar chloride progressions after repair, but chloride concentrations after the deeper concrete repair, ($X_{p2} = 60$ mm), are consistently lower than those with the shallower concrete repair, ($X_{p1} = 50$ mm). That is because the deeper replacement depth slows the chloride redistribution from the aged concrete to the critical depth due to lesser chloride ions remaining in the old concrete and longer diffusion distance. Based on the results, a second repair would be recommended around year 40

for the shallower repair and year 43 for a deeper repair. These second repairs and additional repairs each time C_{crit} is reached were also simulated to observe the repair schedule under the corrective repair strategy with different repair depths. The results for 70 years are shown in Figure 2.10.

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

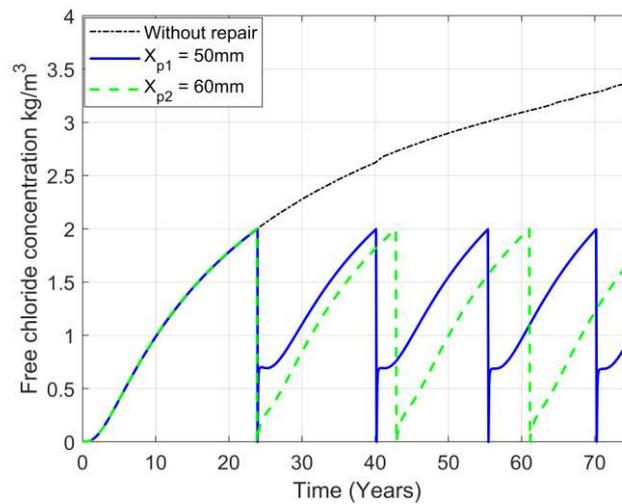


Figure 2.10: Time-dependent evolution chloride concentration at the cover depth without and with repair

The repair application time and the number of repairs of X_{p1} and X_{p2} for the considered structural lifetime are presented in Table 2.2. While the repair strategy of X_{p1} is triggered four times in 70 years to mitigate the risk of corrosion initiation, the concrete structure with X_{p2} requires only three repairs in its lifespan. It is observed that a deeper repair depth leads to a fewer number of repairs and so that a 60 mm repair depth may be preferred. Conversely, the cost and the amount of new concrete placed for a single repair at a deeper depth is higher. Economic and environmental comparison of the repair strategies will be addressed in Chapter 4.

Table 2.2. Number of repairs for two different repair depths

Repair depth	Repair application time (year)					Number of repairs
	First	Second	Third	Fourth	Fifth	
50mm	23.9	40	55.2	69.8	>70	4
60mm	23.9	42.8	60.8	>70		3

2.4.2.2. Scheduled repair interval strategy

The second approach for a maintenance model schedules structure repairs at regular intervals. Repairs are therefore carried out at pre-scheduled times. It is necessary to evaluate by modeling, testing, or expert advice the risk of corrosion occurring at those times. The first repair strategy considered in this section is the removal and replacement of $X_p = 50$ mm of surface concrete at 15- and 18-year intervals, as shown in Figures 2.11a, b.

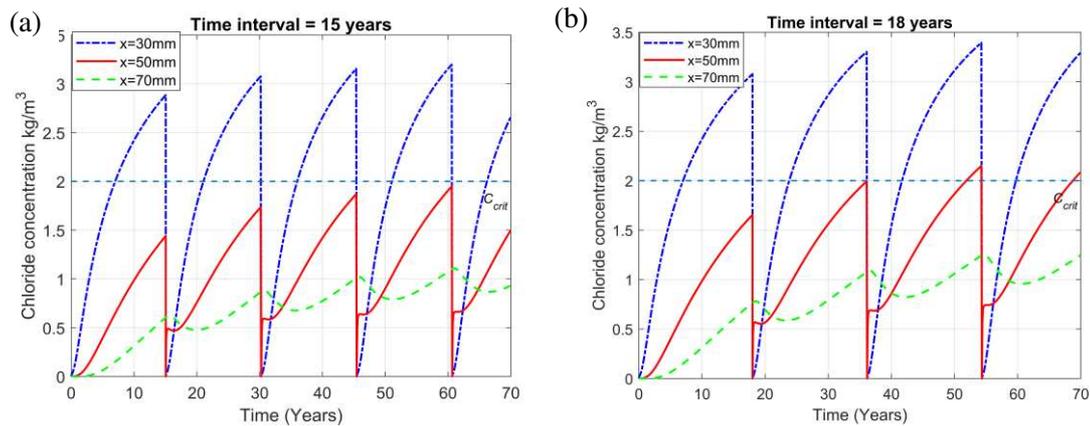


Figure 2.11. Time-dependent evolution of the chloride concentration at three depths ($x = 30, 50$ and 70 mm) with $X_p = 50$ mm and (a) 15-year repair interval, or (b) 18-year repair interval

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

To assess the usability of the repair strategy, the chloride concentration is assessed at the cover depth of 50 mm and compared with a threshold chloride content of 2 kg/m³ (as seen on the profile $x = 50$ mm in Figures 2.11a, b). It is observed that the chloride concentration by the repair strategy of 50 mm replacement depth and 15-year maintenance interval satisfies the requirement of reducing corrosion initiation risks for multiple repairs (Figure 2.11a). Meanwhile, with $X_p = 50$ mm but the 18-year maintenance interval, this repair plan is not suitable in preventing the risk of corrosion initiation because the chloride concentration exceeds

the critical value before the third repair action is implemented (see Figure 2.11b), which may affect the durability issue in RC structures.

Next, a deeper repair depth, $X_p = 60$ mm, is considered, with repair intervals of 15 and 18 years. Figures 2.12a, b show that when considering the critical chloride concentration of 2 kg/m^3 at the cover depth (as seen on the profile $x = 50$ mm), both repair intervals are appropriate to mitigate corrosion initiation risks for RC structures. In which case the less frequent repair interval of 18 years may be preferred from an economic perspective and environmental point of view as it has less expected number of repairs. Hence, a comprehensive maintenance strategy should evaluate both repair intervals and replacement depths, balancing (a) costs, environmental impacts, and other (b) risks, to achieve an optimal protection from corrosion. The deterministic example highlights how different repair depths affect the maintenance schedule, however the impact of concrete cover should not be underestimated because of its contribution in extending the time of corrosion initiation (Bastidas-Arteaga and Stewart, 2016).

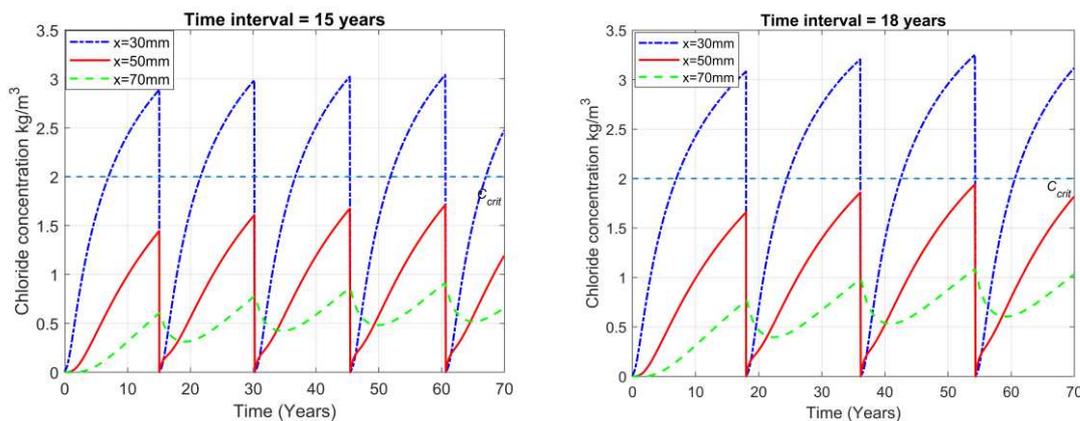


Figure 2.12: Time-dependent evolution of the chloride concentration at three depths ($x = 30, 50$ and 70 mm) with $X_p = 60$ mm and (a) 15-year repair interval, or (b) 18-year repair interval

2.5. Case study 2: Deterministic 2D model of chloride ingress in repaired concrete

2.5.1. Problem description

The objective of this example is to illustrate a 2D maintenance model presented in section 2.3.2 that simulates the process of chloride ingress in repaired concrete structures exposed to a marine environment. In addition, this model assesses the ability of cover replacement measures to extend the long-term durability of the structure, therefore providing a method to compare model responses to different repair and maintenance strategies. The maintenance model assumes that the structure has a service life of 75 years. The necessary assumptions about concrete properties, the average ambient temperature and relative humidity, the constant surface chloride concentration, and the critical chloride value for corrosion initiation are similar with the 1D

deterministic example in section 2.4. Similarly, the remainder of the parameters used in the model are listed in Table 2.1.

A square RC column with cross-sectional dimensions 0.4 m x 0.4 m is examined. It is subjected to chloride penetration in the two orthogonal directions x and y . The concrete cover depth, X_c , is 50 mm (as shown in Figure 2.13)

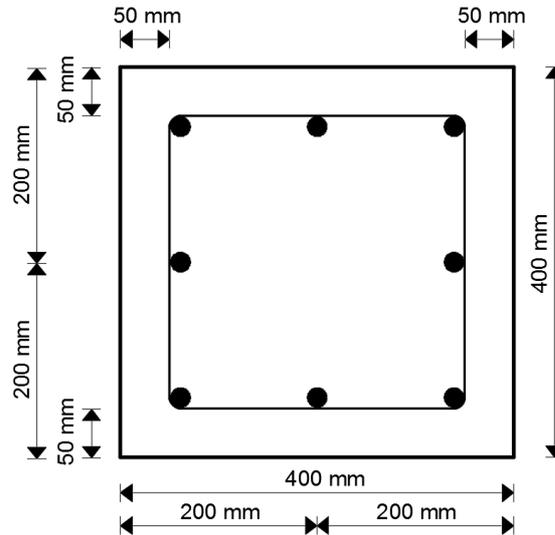


Figure 2.13: Cross-sectional of square column

Exploiting the model symmetry, a 0.2 m x 0.2 m truncated model is studied, assuming that the reinforcing bars are not included in the model. Three concrete replacement strategies are simulated for the same structural element, including:

1. ORC1 strategy: Orthogonal concrete replacement with $X_p = X_c = 50$ mm, assuming that the repair thickness (X_p) is equal to the existing cover (Figure 2.14a).
2. ORC2 strategy: Orthogonal concrete replacement with $X_p = 60$ mm $>$ $X_c = 50$ mm, where the repair thickness is assumed to be thicker than the existing cover (Figure 2.14b)
3. CCR3 strategy: Circular concrete replacement where the boundary line between two materials is a circular curve (Fig. 2.14c)

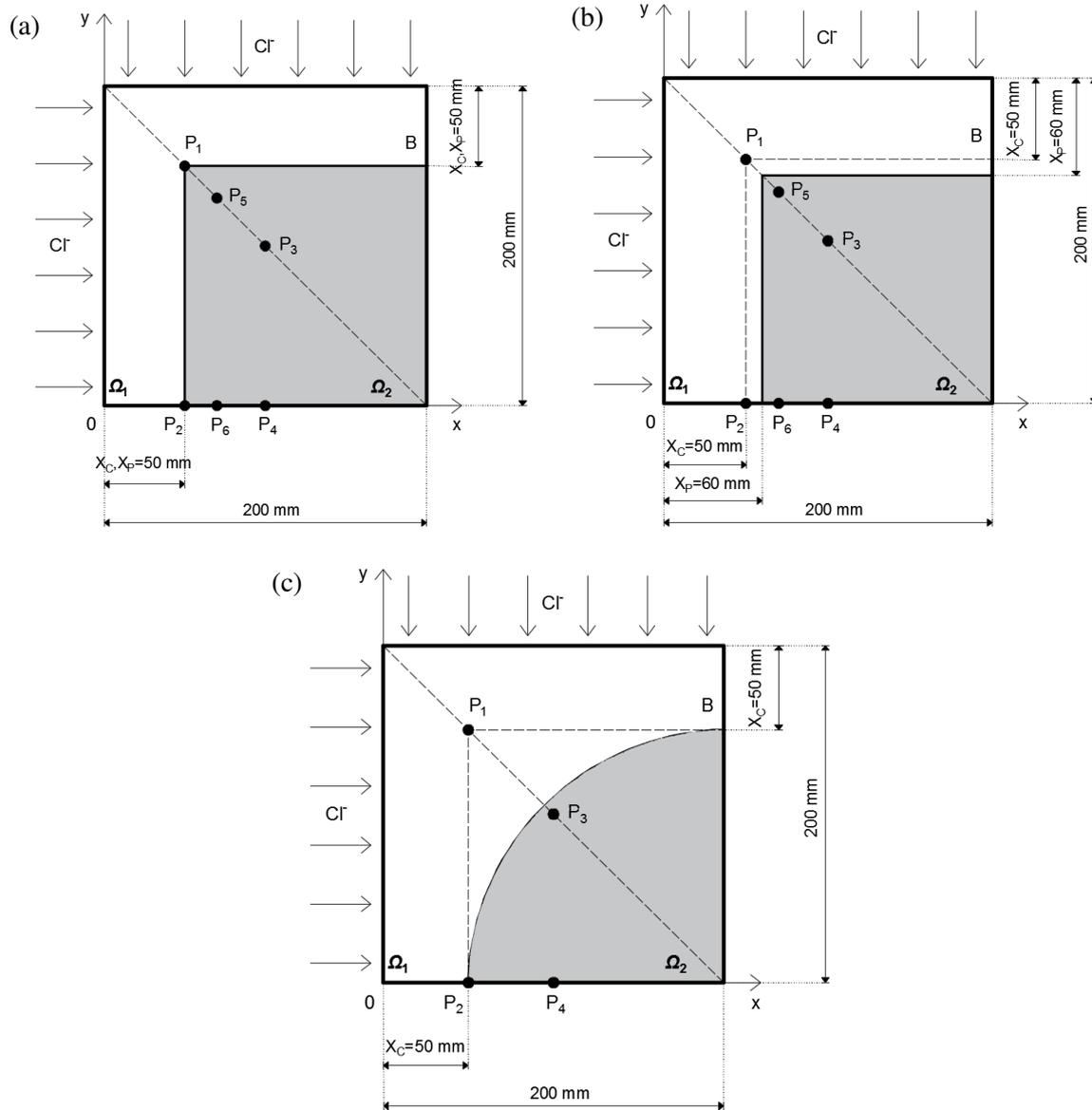


Figure 2.14: (a) OCR1 with $X_p = X_c$, (b) OCR2 with $X_p > X_c$ (c) CCR3

2.5.2. Analysis of the results

Under the reactive repair strategy, the corroded RC column is repaired by concrete replacement when the chloride content at the reinforcement cover exceeds a threshold that triggers corrosion. The maintenance model calculates the time intervals between repairs for the three strategies described above. Performing this analysis is necessary to: a) understand the evolution of chloride concentration at different locations within the RC column elements, including the repaired material and original concrete, b) investigate how the shape and thickness of the concrete replacement affect the reduction of chloride concentration at critical locations, and c) define the maintenance schedule.

2.5.2.1. Evolution of the chloride concentration within the repaired material

For each of the strategies OCR1, OCR2, and CCR3, the evolution of chloride concentration at two points located at the depth of the concrete cover was studied (as shown in Figure 2.15). These points, designated P_1 and P_2 , are depicted in Figure 2.14 and have coordinates (0.05, 0.15) and (0.05, 0.0), respectively. Point P_1 corresponds to the chloride distribution at the corner reinforcement, while point P_2 represents the chloride distribution at a middle reinforcing bar.

It is worth noting that if the repair thickness is equal to the existing cover (OCR1), the residual chloride in the old concrete quickly redistributes to points in the adjacent zone or to points at the critical depth, whereupon the redistribution is gradual (see Figure 2.15a). A similar trend is observed for the chloride profile of point P_2 in CCR3 after repair (see Figure 2.15c) because P_2 lies at the interface between Ω_1 and Ω_2 , causing the chloride concentration at P_2 to predominate over P_1 in the initial years after repair. However, the chloride concentration at P_1 ultimately reaches C_{crit} before P_2 due to the increased uptake of environmental chloride at the corners of the element.

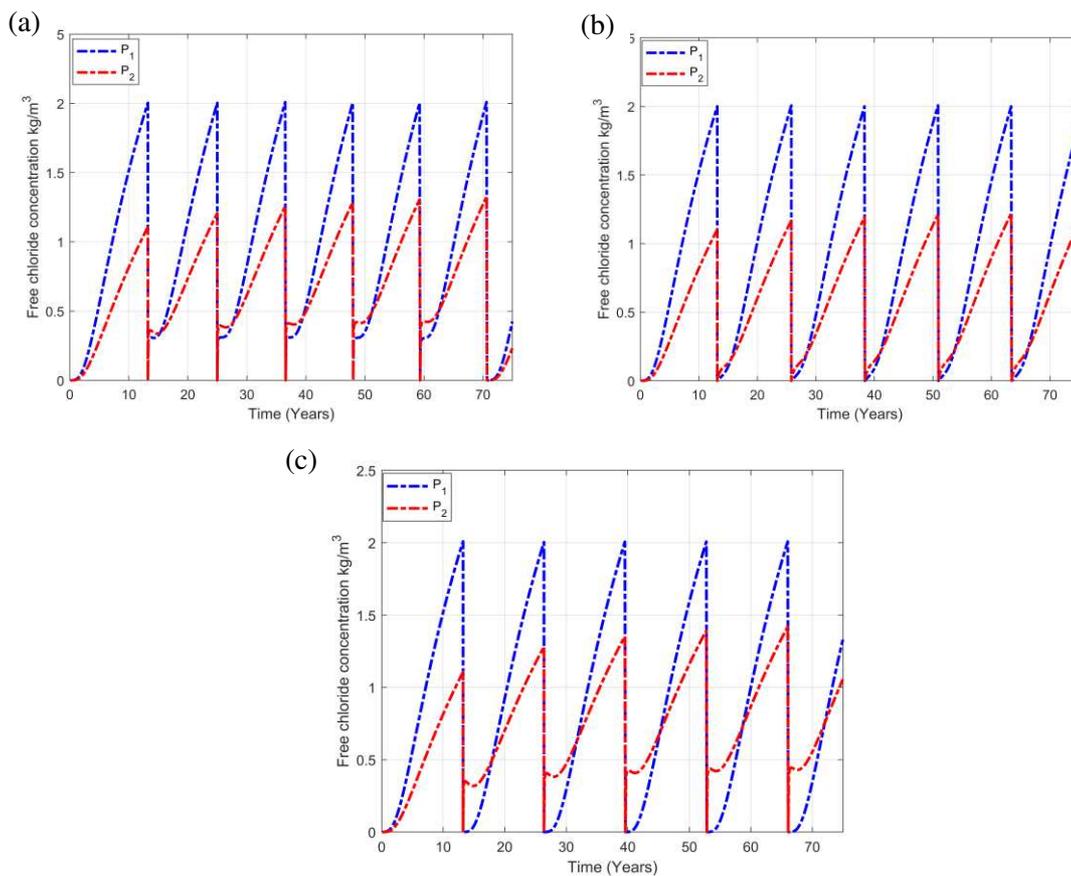


Figure 2.15: The evolution of chloride concentration in different locations of repair concrete (a) OCR1, (b) OCR2 (c) CCR3

Thus, it can be seen that the chloride concentration accumulates to the critical value for the onset of corrosion at the corners (P_1) before it does at P_2 , for all strategies (see the blue lines P_1 in Figure 2.15a, b, c). This outcome is consistent with previous studies in RC structures subjected to two-dimensional chloride attack, which emphasize the corners of the concrete as the most crucial positions in predicting the time to corrosion initiation (Jin *et al.*, 2015; Wu *et al.*, 2020). These results suggest that repair should occur when the chloride concentration reaches the critical threshold at point P_1 rather than at point P_2 , as a delay in repair at this point may negatively impact the durability and serviceability of RC structures. To determine the time intervals between repairs for each strategy, Figure 2.16 displays the progression of the chloride concentration at the corner of the square concrete column (P_1). It can be shown that the initial onset of corrosion occurs at approximately 13 years after the structure is put into service for all three strategies, as depicted by the blue line P_1 in Figure 2.16. If the material properties of the structures prior to repair are kept constant, the determination of the first repair time is solely dependent on the thickness of the cover. On the other hand, the duration of subsequent repairs depends on the repair characteristics, particularly the shape and area of concrete removed. When the replacement concrete domain is thicker than the concrete cover (strategies OCR2 and CCR3), the time to corrosion initiation is longer than in strategy OCR1 (as shown in Figure 2.16).

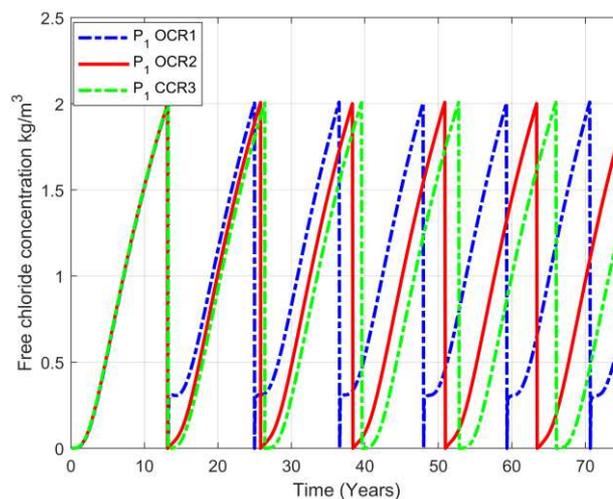


Figure 2.16: The evolution of chloride concentration at the corner area in three strategies

Based on the results in Figure 2.16, Table 2.3 shows the time intervals between corrosion occurrences during a 75-year service life, where the time interval between subsequent corrosion initiation times for OCR1 is about 11 years, while these time intervals for OCR2 and CCR3 are about 12.4 and 13 years, respectively. This can be explained by the facts that: (1) there is a smaller amount of remaining concrete and thus fewer residual chloride ions in the old concrete domain, and (2) the position of P_1 in OCR1 is at the interface between Ω_1 (new concrete) and Ω_2 (old concrete with residual chloride ions) while in OCR2 and CCR3, P_1 belongs to the new

concrete and is not in close proximity to the interface. If the onset of corrosion is considered as a repair criterion, the time intervals between corrosion occurrences in Table 2.3 indicate the times for repair based on a reactive approach. It is observed that CCR3 requires less repairs than OCR1 and OCR2.

Table 2.3: Time intervals between corrosion occurrences for the three repair strategies

Strategy	Repair time (year)						
	1 st	2 nd	3 rd	4 th	5 th	6 th	7 th
OCR1	13.15	11.67	11.42	11.26	11.17	11.1	$\Sigma T_i > 75$
OCR2	13.15	12.57	12.41	12.41	12.32	12.32	$\Sigma T_i > 75$
CCR3	13.15	13.06	13.06	13.06	13.06	$\Sigma T_i > 75$	

To further support the aforementioned points, the profile of chloride concentration along the line P_1P_2 (as depicted in Figure 2.14) at a depth corresponding to the concrete cover is plotted at three different times, as shown in Figure 2.17. Examining the blue curves in all three strategies (representing the chloride profile prior to repair at $t = 13$ years), it can be observed that the chloride concentration at the critical point P_1 is approximately equal to the threshold for the onset of corrosion, indicating the imminent need for repair. For the chloride profiles 10 years after the first repair (represented by the dashed yellow curves in Figure 2.17a, b, c), the chloride concentration is highest at the corner and decreases towards the center of the element. At this time, the mechanism of chloride transfer is no longer characterized by the redistribution of residual chloride in the old concrete, but rather by the penetration of chloride from the external environment. Among the three repair strategies, the chloride concentration at point P_1 is highest under OCR1 (about 1.7 kg/m^3), followed by OCR2 (about 1.6 kg/m^3), and lowest under CCR3 (about 1.5 kg/m^3). This confirms that, after the first repair, structures under the OCR1 strategy will require repair at the earliest point in time.

The chloride profile one year after repair in all three strategies (shown as dotted red curves in Figure 2.17a, b, c) exhibits lower values because the concrete containing chloride was removed and replaced with a new domain of concrete. As the redistribution of residual chloride does not favor the corner location in the same manner as environmental chloride, the corner area does not display the highest chloride concentration one year after repair. Specifically, for the CCR3 strategy (involving a circular repair section, as depicted in Figure 2.14c), the chloride concentration at point P_1 one year after repair is quite low and is largely unaffected by the redistribution of chloride ions from the old concrete. The observation that the chloride concentration at P_1 is lower than at P_2 in the early years following repair under the CCR3 strategy aligns with the findings presented in Figure 2.15c.

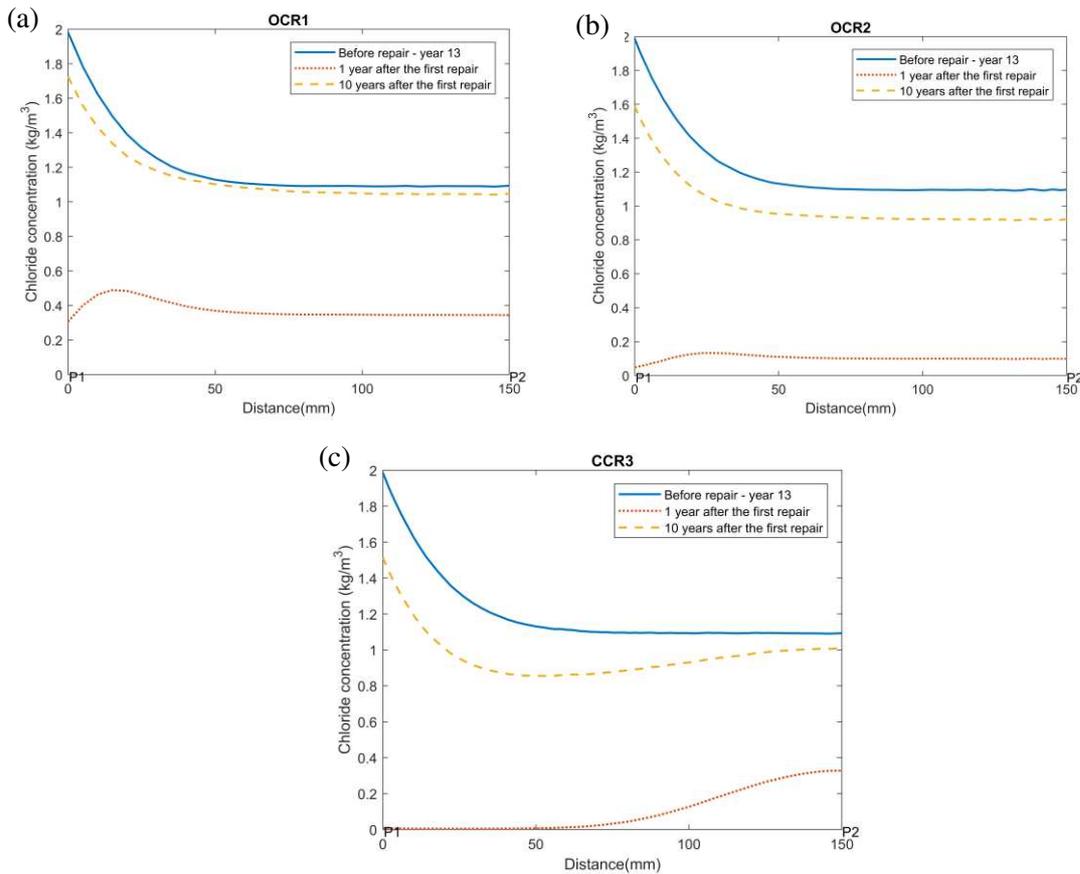


Figure 2.17. The chloride concentration at the depth of concrete cover (a) OCR1, (b) OCR2 (c) CCR3

Figure 2.18a, b, c depict iso-concentration contours of chloride ions (kg/m³) one year following the initial repair, showcasing the redistribution of chloride ions at the boundary between aged and repaired concrete under the three strategies. In the orthogonal strategies (OCR1 and OCR2), residual chloride tends to accumulate in the corner regions after repair (Figure 2.18a, b); while in the circular strategy (CCR3), where a larger amount of corner concrete has been replaced, the redistribution of chloride occurs initially near the center of each face of the element (Figure 2.18c). This characteristic of CCR3 is advantageous in comparison to OCR2, which also replaces more concrete than the cover concrete domain, as it can help mitigate the buildup of chloride stresses in the corner regions and therefore extend the time until the onset of corrosion in these critical areas.

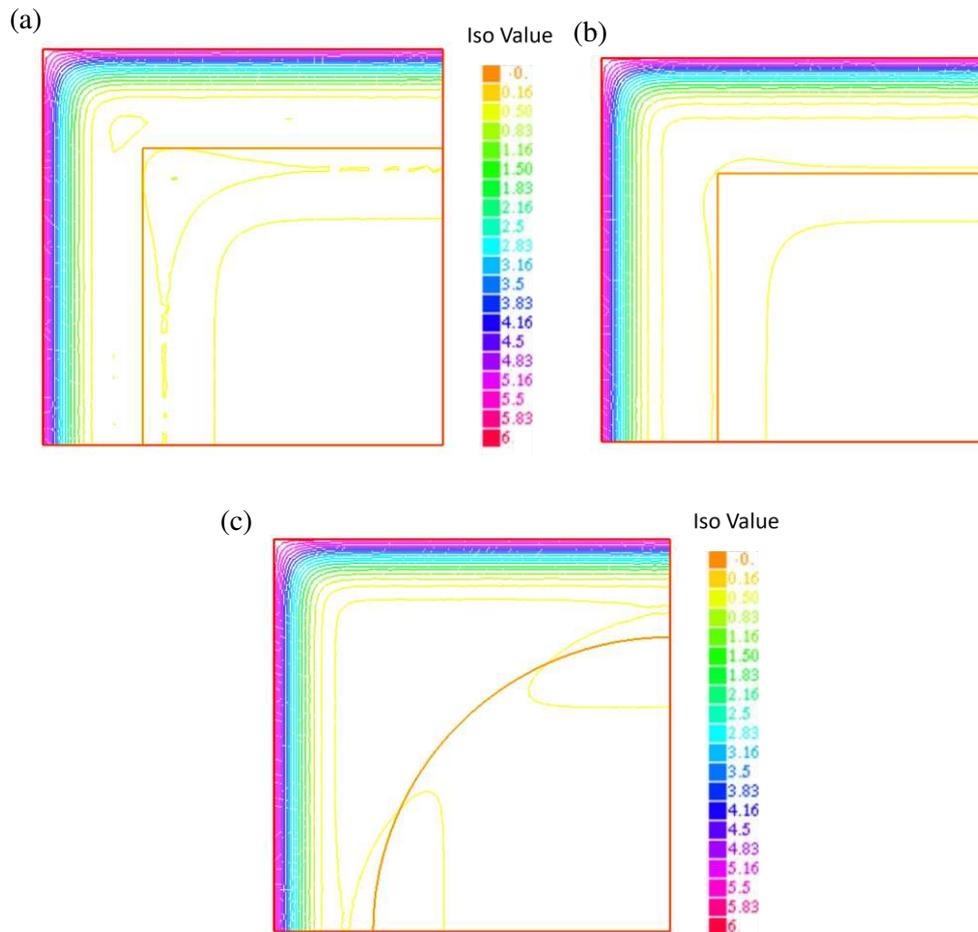


Figure 2.18: Chloride ion iso-concentration contours at one year after the first repair (a) OCR1, (b) OCR2 (c) CCR3

2.5.2.2. Evolution of chloride concentration in locations of unrepaired, aged concrete

Four points, designated P_3 , P_4 , P_5 , and P_6 and located at coordinates (0.1, 0.1), (0.1, 0.0), (0.07, 0.13), and (0.07, 0.0), respectively (as depicted in Figure 2.14), were studied for strategies OCR1 and OCR2. For strategy CCR3, only points P_3 and P_4 were examined. Point P_3 and P_5 are situated on the diagonal of the square cross-section of the concrete column, while points P_4 and P_6 are situated along the centerline of a face of the element and located within the old concrete, serving as representative points for areas that are distant from the corner area. The evolution of chloride concentration at these locations in the unrepaired, aged concrete is shown in Figure 2.19a, b, c. These figures demonstrate a redistribution of the chloride ions remaining in the aged concrete after repair. For each repair strategy, the chloride concentration decreases immediately after repair due to the chloride diffusion of the new concrete domain, and then gradually increases after a few years when redistribution is complete, and the environment is re-established as the primary source of chloride. This continues throughout the 75-year lifespan of the concrete structure, resulting in a curve that oscillates with the repair cycle, as shown in Figure 2.19a, b, c. This oscillation is more pronounced at points that are closer to the repair

boundary, such as P₅ and P₆ in strategies OCR1 and OCR2. At points far from the repair boundary, such as P₃ and P₄, the fluctuation is less pronounced because 1) there is less chloride at areas deeper beneath the element surface, and 2) there is no adjacent area of low chloride concentration to redistribute chloride ions to (see Figure 2.19a, b). In strategy CCR3, the oscillation of chloride redistribution is also observed at point P₃ near the repair boundary, but it is not as pronounced as in strategies OCR1 and OCR2 (see line P₃ in Figure 2.18c and P₅ in Figure 2.19a, b). At point P₄, the chloride concentration gradually increases over time, surpassing the concentration at point P₃ after the first repair and then continuing to increase in subsequent years (see Figure 2.19c). This trend is the opposite of that observed in strategies OCR1 and OCR2, where point P₃, near the corner of the element, consistently has a higher chloride concentration than the point near the middle of the column, point P₄ (see Figure 2.19a, b).

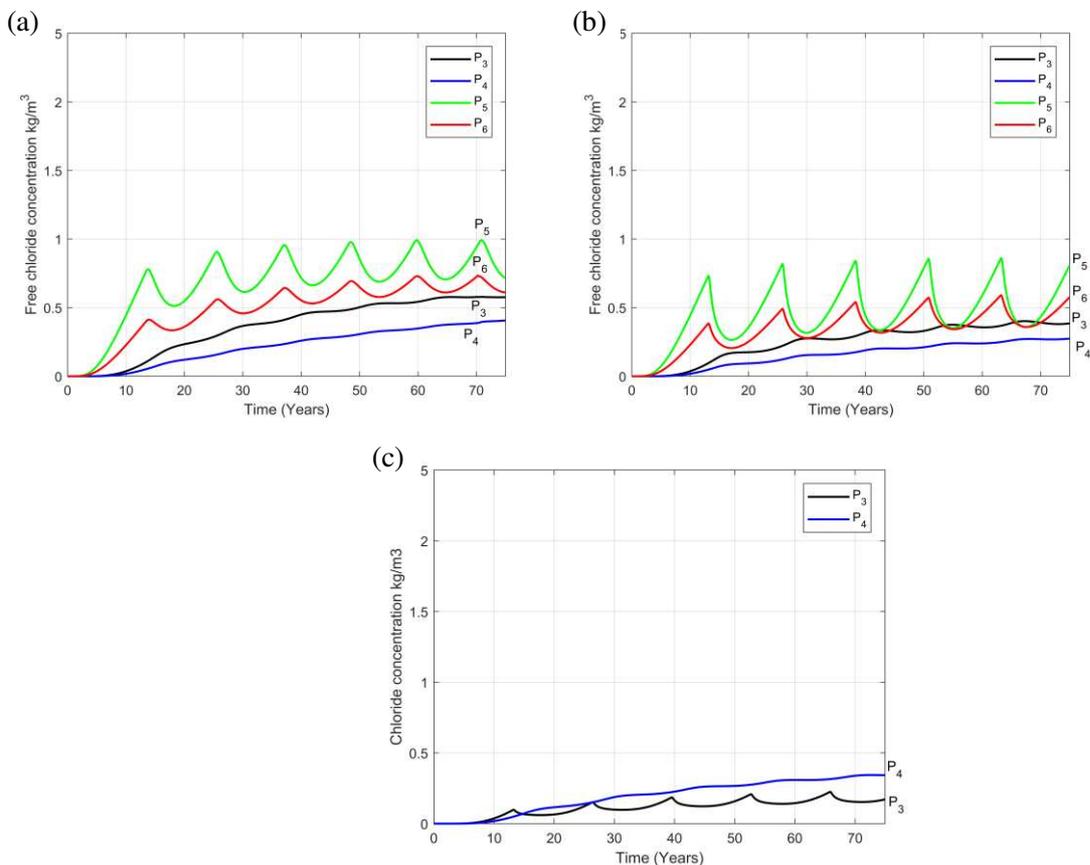


Figure 2.19: The evolution of chloride concentration in different locations of unrepaired, aged concrete (a) OCR1, (b) OCR2, (c) CCR3

The findings from this study indicate that the CCR3 repair strategy, which employs a circular curve boundary rather than a vertical boundary, significantly impacts the distribution of residual chloride, resulting in a higher concentration in the central region compared to the corner region. Given that the corner of the concrete is typically identified as the most crucial location for predicting the onset of corrosion, the longer repair intervals of the CCR3 strategy,

which translate to fewer repairs over the lifespan of the structure, can be attributed to this characteristic. Additionally, the replacement of new material leads to nearly zero chloride concentration at the location near the corner area immediately following repair, and the redistribution of chloride from the old concrete to this location is minimal. As a result, the CCR3 repair strategy has the potential to restore damaged structural elements to their original condition, with the time interval between corrosion initiations approaching the time to initial corrosion initiation. This is not observed in the repair of structures subjected to one-dimensional chloride attack (Petcherdchoo, 2018; Truong *et al.*, 2022) or in the CCR1 and CCR2 strategies, given that the quality and properties of the new concrete are assumed to be nominally identical to those of the original concrete.

For the purpose of simplicity, this study chooses to utilize simple orthogonal and curved repair sections. However, it is important to note that in practice, the repair characteristics should be optimized in order to achieve an optimal balance between the effectiveness of the repair and the volume of concrete replaced, which impacts cost, carbon dioxide (CO₂) emissions, and waste generation. A comparison of the three strategies in terms of optimizing maintenance scheduling, cost, and environmental impact will be presented in Chapter 4.

2.6. Case study 3: Influence of temperature and humidity on corrosion process and maintenance plan

2.6.1. Problem description

The aim of this example is to investigate the effect of varying environmental conditions in Vietnam on chloride ingress into concrete which leads to service life reduction for RC structures. Three coastal cities in the Northern area (Hai Phong), Central area (Danang) and Southern area (Ho Chi Minh) of Vietnam are selected. Based on the results, a proper maintenance strategy for RC structures in each coastal area is recommended. Other analysis assumptions are also considered in this study:

- The structure is exposed to a marine atmospheric zone and located 0.1 km from the shoreline. Chlorides are carried by salt spray from seawater and mixed in the humid air.
- Chloride flow is a one-dimensional process (e.g., concrete slab).
- The surface chloride content is assumed to be constant during the lifetime of structures, $C_s = 2.95 \text{ kg/m}^3$ concrete [15].
- The concrete is modeled as 8% C₃A Ordinary Portland Cement (OPC), its cement content is 400 kg/m^3 and water/cement ratio is 0.5, which meets Vietnamese standards for RC structures in a marine atmospheric zone [17].

- Concrete cover is 50 mm ($X_c = 50$ mm). The concrete used for repair is the same quality as the original concrete.
- Service life is 50 years, and the corrosion initiation starts when the chloride content at the position of rebar exceeds a critical value $C_{crit} = 0.5\%$ by weight of cement (about 2.0 kg/m^3).
- The model parameters and their values used for chloride ingress model and maintenance model are presented in Table 2.1.

Climatic conditions are based on three coastal cities in Vietnam with varying temperature and humidity data: Haiphong (Northern Vietnam where the climate is in the subtropical zone, high temperature and humidity), Danang (Central Vietnam, located in the tropical monsoon climate, high temperature with less fluctuation), Ho Chi Minh City (Southern Vietnam, sub-equatorial climate, high and quite stable temperature during the year). The data of annual temperature and humidity in the three mentioned-above cities is presented in Table 2.5.

Table 2.5: Annual temperature and humidity in three cities in Vietnam

Locations	Annual Temperature ($^{\circ}\text{C}$)	Annual humidity (%)
Haiphong	23.3	85
Danang	25.8	81.6
Ho Chi Minh City	27.4	78

2.6.2. Results and discussions

2.6.2.1. Service life prediction

Chloride concentration at rebar depth ($X_c = 50$ mm) in unrepaired concrete structures over a 50-year service life was modeled and results are shown in Figure 2.20 for the three coastal cities. The surface chloride concentration for all study locations was assumed to be the same and constant. It is observed that the relative humidity and temperature have an effect on the progression of free chloride concentration over 50 years. Considering a critical chloride content of 2 kg/m^3 , the estimated time to corrosion initiation in Haiphong and Danang are 27.6 years and 29.2 years respectively, while it is 34 years for RC structures in Ho Chi Minh City. Although the average annual temperature in Ho Chi Minh City is recorded as the highest, the time to initiate corrosion on RC infrastructure in this area is the longest. This may be attributed to the lower annual humidity in Ho Chi Minh City compared to the two other cities.

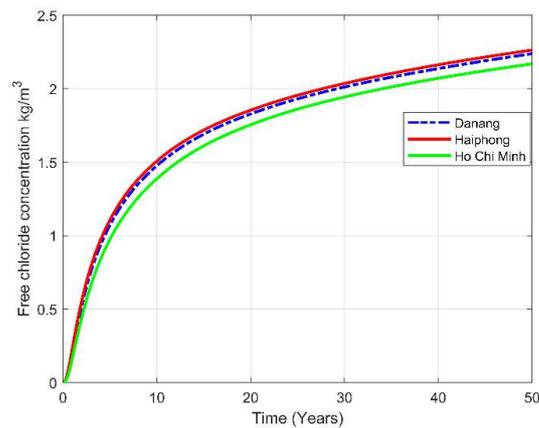


Figure 2.20: Chloride profiles at the cover depth of 50 mm in three coastal cities

2.6.2.2. Service life prediction of repaired concrete structures

Chloride ingress is now modeled considering that concrete repair occurs during the structure service life. Chloride concentration is again assessed at a depth of 50 mm. A maintenance plan may consider several repair factors including the number of repair cycles, the replacement concrete depth and repair interval (time between two repairs). The first repair strategy considered is the removal and replacement of 50 mm of surface concrete at 15 and 18-year intervals. Figures 2.21a, b present 50-year chloride concentration with repair intervals of 15 years and 18 years and a 50-mm repair depth. By taking a threshold chloride content of 2 kg/m^3 at a cover thickness of 50 mm, it is observed that the chloride profiles of Ho Chi Minh satisfy the requirement of reducing the risk of corrosion initiation for multiple repairs. Hence the 50-mm replacement depth and the 18-year maintenance interval are well-adapted for marine structures in Ho Chi Minh City. Meanwhile, in Haiphong, those above repair strategies are not suitable in preventing the risk of corrosion initiation; the repair interval for Haiphong should be shorter, approximately 10 to 13 years. For Danang, it is necessary to address the maintenance issue in the last years of service life when the chloride concentration rate is faster, and the repair should be conducted at an earlier age than planned.

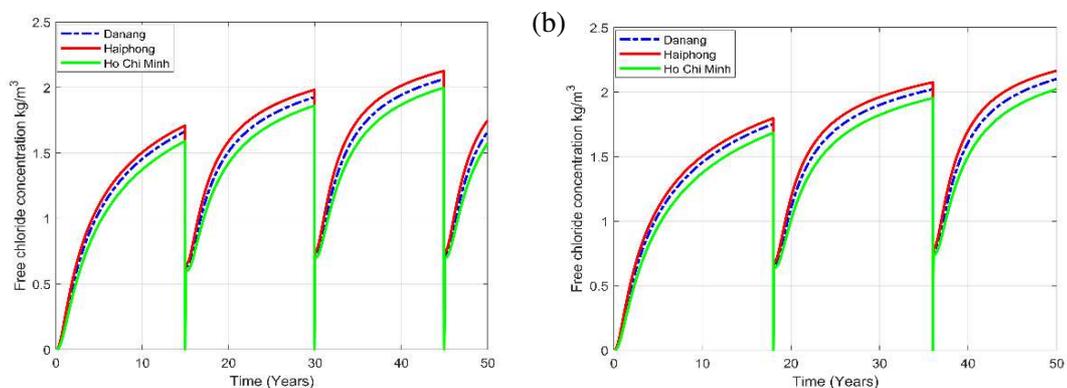


Figure 2.21: Chloride profiles with repair interval of (a) 15 years, (b) 18 years (50 mm concrete replacement depth $X_p = 50 \text{ mm}$)

According to previous studies [12, 19], replacement depth is one of the main factors influencing the effectiveness of maintenance and repairing with a deeper depth is proved by previous studies to be more effective. Hence, a 60-mm deeper repair depth is considered in this section, with repair intervals of 15- and 18-years. Figure 2.22a, b show that when considering the threshold chloride level of 2 kg/m^3 at the rebar depth, both repair strategies are appropriate for RC structures in Danang and Ho Chi Minh City. In which case the less frequent repair interval of 18 years may be preferred from both economic and environmental aspects as it has less expected number of repairs. For Haiphong, the 18-year interval appears to be a better solution compared to the 50-mm replacement depth. However, a comprehensive maintenance plan should evaluate additional repair intervals and replacement depths to optimize the protection of structures from corrosion and the maintenance cost.

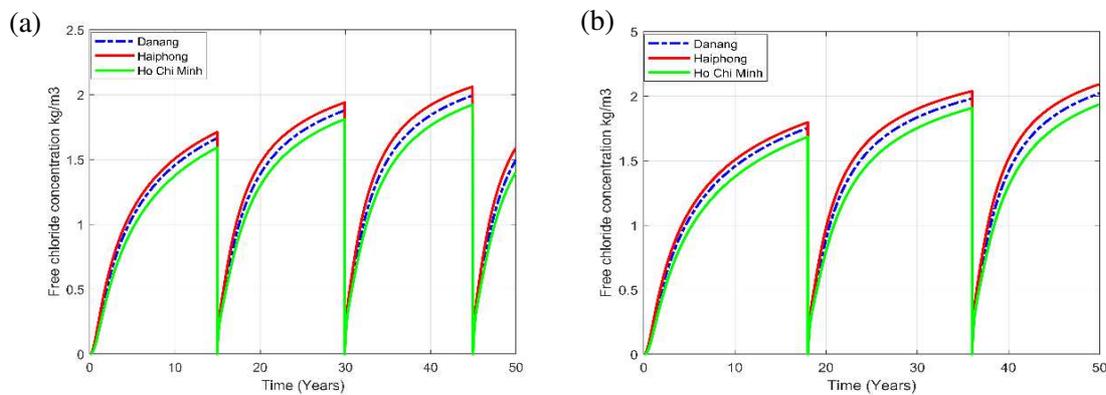


Figure 2.22: Chloride profiles with repair interval of (a) 15 years, (b) 18 years (60 mm concrete replacement depth $X_p = 60 \text{ mm}$)

It can be shown that temperature and humidity have a significant effect on the service life of RC structures exposed to chloride-heavy environments. The conditions of high temperature and high humidity in most of cities in Vietnam would accelerate the chloride penetration into concrete and shorten the service life of structures before and after repair. Particularly in coastal areas of Northern and Central Vietnam, where the average annual humidity and temperature are quite high compared to other locations, maintenance policy and repair strategy should be well planned to ensure the serviceability and durability of chloride contaminated RC structures. Besides cover rebuilding method, it is recommended to include additional corrosion protection methods such as water concrete coating, anti-corrosion coating for reinforcement, or application of supplementary cementitious materials to optimize the effectiveness of maintenance and enhance the durability of structures. Moreover, the current standards for concrete quality and concrete cover depth to prolong the operational life of RC structures subjected to chloride ingress must be complied with. It is noted that these predictions of the service life of structures, as well as the effectiveness of maintenance measures, can be improved when a more comprehensive model that incorporates heat and moisture transport is considered.

2.7. Summary and conclusions

Modeling chloride penetration into concrete after repair is challenging due to the interaction of the two main sources of chloride ions: ambient chloride concentration and residual chloride ions in the remaining concrete. In this chapter, a maintenance model is developed to simulate different strategies of cover rebuilding method. A numerical solution implemented in FreeFem++ using a domain decomposition technique is presented. Finally, several deterministic examples are used to evaluate the influence of some main parameters on the repair effectiveness and the influence of local weather conditions on the chloride ingress process. The summary and conclusions of this chapter are:

1. The first part of this chapter presents the basic considerations and the governing equations for the chloride ingress model. An efficient model for chloride intrusion is proposed, considering the diffusion process as the main controlling mechanism. The constitutive nonlinear partial differential equations are solved by coupling finite element and finite difference methods. The chloride ingress model was validated by comparing its simulation results with those of analytical and experimental works. In general, the computer code shows good agreement with the analytical and empirical results, which contributes to validate the chloride penetration model for further development of the maintenance model. However, further research needs to develop a more comprehensive model that considers moisture and heat transport to improve the service life prediction of corroded RC structures.
2. The mechanism of chloride transport in repaired concrete elements is proposed, and the numerical approach to develop 1D and 2D maintenance models, that consider a spatially domain-dependent and time-dependent diffusion coefficient in two material domains is illustrated. Compared to previous models, the maintenance model in this study takes into account the influence of general weather conditions, chloride binding capacity, decrease in chloride diffusivity with age of concrete and two-dimensional chloride flux in a two-domain system. It has been applied to maritime structures exposed to atmospheric or submerged zones with constant chloride surface concentration. However, the proposed model still has some limitations: It does not account for (1) chloride penetration by convection and capillary sorption, and (2) the change in surface concentration of concrete with time.
3. A deterministic analysis of a 1D maintenance model that predicts the service life of chloride-polluted concrete structures with multiple repairs is performed. Three factors are considered that affect the effectiveness of the repair method: (1) number of expected repairs, (2) depth of repair concrete, and (3) repair interval. It is found that a greater depth of repair results in a smaller number of repairs, while a shorter repair interval

reduces the likelihood of corrosion initiation but increases maintenance costs. Therefore, a comprehensive maintenance policy should consider these main parameters to find an optimal strategy that minimizes both material costs and environmental impact and prevents the onset of corrosion. Other control factors, such as repair costs, should also be considered as they play an important role for decision makers when comparing different repair strategies.

4. For the 2D maintenance model, the results show that for RC structures subjected to two-dimensional chloride attack, the corners of a column are the most critical areas to evaluate chloride concentration, predict the time of corrosion onset, and estimate repair times during the design lifetime. In addition, the circular concrete replacement strategy is recommended over orthogonal concrete replacement for repairing RC columns because replacing a larger amount of corner concrete that follows a curved shape can help reduce the chloride concentration that accumulates in the corner region over time, extending the time for corrosion to begin.
5. Finally, it is shown that the chloride-induced corrosion process is strongly influenced by the local weather conditions. Particularly, the high temperature and humidity in many cities in Vietnam can accelerate chloride penetration into concrete and shorten the service life of RC structures before and after repair. Therefore, proper maintenance policies and repair strategies that include additional corrosion protection methods should be well planned and implemented to ensure the durability of corroded RC structures, along with compliance with current standards for concrete quality and depth of cover.

Chapter 3 *Probability-based Maintenance Modeling and Planning for Chloride-Contaminated Concrete Structures*

3.1. Introduction

The durability of RC structures is a crucial aspect of infrastructure maintenance, as it directly affects the safety and serviceability of these structures. Therefore, it is of utmost importance to understand the degradation processes and develop appropriate repair strategies. Traditionally, deterministic models have been used for maintenance planning, but they do not fully account for the inherent uncertainties in material properties, environmental conditions, and other relevant factors. To overcome this limitation, probabilistic modeling has proven to be a powerful tool for concrete repair and maintenance analysis and planning.

However, probabilistic modeling of degradation processes in repaired concrete structures is still at an early stage, especially in the context of chloride-induced corrosion. To address this issue, a comprehensive probabilistic approach is needed to accurately estimate the extent of chloride penetration in repaired concrete structures and predict their service life. This requires consideration of several random parameters, including material properties, environmental conditions, depth of cover, and depth of concrete replacement.

In this chapter, a comprehensive probabilistic model is proposed for predicting the service life of corroded RC structures with cover rebuilding repair. The developed model facilitates the determination of appropriate repair strategies and the optimization of maintenance schedules considering uncertainties. The results of this section have important implications for infrastructure maintenance and planning by providing a more accurate and reliable basis for decision making. The objectives of this chapter are:

1. to present the reliability problem of RC structures subjected to chloride ingress, considering the repair;
2. to define the nature of uncertainty related to chloride penetration in repaired concrete and assign probabilistic values to the uncertain parameters;

3. to present the probabilistic methodology used for the maintenance model and perform the probabilistic assessment for maintenance modeling and planning;
4. to introduce the use of surrogate models to facilitate probabilistic assessment and sensitivity analysis;
5. to propose a maintenance schedule optimization approach for different repair strategies.

The first section of this chapter, Section 3.2, discusses the basic elements of reliability theory along with the limit state functions used in this research. Then, the methodology of the probabilistic maintenance model is presented and the random variables are explained in detail. In the following section (Section 3.3), a numerical example is used to demonstrate the probabilistic lifetime assessment of repaired RC structures using the Latin Hypercube Sampling method. Section 3.4 presents the mathematical setting of Polynomial Chaos Expansions (PCE) method to construct a surrogate model and perform a global sensitivity analysis for maintenance modeling. Finally, two case studies are presented in Sections 3.5 and 3.6 to illustrate the effectiveness of the proposed surrogate models in controlling the probability of corrosion initiation and optimizing the maintenance schedule for repaired RC structures.

3.2. Probabilistic assessment for reliability analysis

As presented in section 1.5.1, Chapter 1, two main types of limit states are considered in the design and analysis of structures: serviceability limit state (SLS) and ultimate limit state (ULS). For the problem treated in this study, the SLS is related to *the probability of corrosion initiation*, and the ULS is related to *the probability of failure*.

The safety and durability of RC structures under chloride-induced corrosion is highly dependent on understanding and predicting their SLS and ULS. For repairing corroded concrete, it is generally accepted that the optimum time to carry out repairs is before the completion of the corrosion phase (Bastidas-Arteaga and Schoefs, 2012). As a result, maintenance strategies often emphasize preventive rather than corrective measures, hence the probability of failure used in this study is the *probability of corrosion initiation*, which is calculated based on the serviceability limit state.

3.2.1. Probability of corrosion initiation

The time of corrosion initiation, T_{ini} , is defined as the instant at which the chloride concentration at the cover thickness reaches a critical level, C_{crit} . The chloride concentration at which the passive layer on steel bars is broken down, leading to the start of steel's corrosion, is represented by this critical value. C_{crit} in this paper is assumed to be a random variable because of the uncertainty of concrete material properties. For RC structures repaired by cover

rebuilding technique, the corrosion initiation time may occur on several occasions depending on the exposure conditions, lifetime assessment and number of repairs. It is determined by evaluating the time-dependent chloride concentration at the cover depth using the solutions of nonlinear partial differential equations presented in Chapter 2. The cumulative distribution function (CDF) of time to corrosion initiation, $F_{T_{ini}}(t)$, is defined as:

$$F_{T_{ini}}(t) = \text{Prob} \{T_{ini} \leq t\} = \int_{T_{ini} \leq t} f(\mathbf{x})d\mathbf{x} \quad (3.1)$$

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

$$g(\mathbf{X}, t) = C_{fc}(\mathbf{X}, t, X_c) - C_{crit} \quad (3.2)$$

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

$$p_f = P[g(\mathbf{X}) \leq 0] = \int_{g(\mathbf{x}) \leq 0} f(\mathbf{x})d\mathbf{x} \quad (3.3)$$

3.2.2. Methodology of probabilistic analysis

Recently, Monte-Carlo Simulation (MCS) and the first-order reliability method (FORM) have been applied in probabilistic assessment of the durability of RC structures (Ma *et al.*, 2020; Lehner, Hornáková and Hrabová, 2021). Notably, most of the studies using reliability-based analysis such as MCS, FORM, or SORM (second-order reliability method) were performed on simplified models of chloride ingress using analytical solutions based on Fick's laws of diffusion (Nogueira, Leonel and Coda, 2012; Yu, Ning and Li, 2017). However, it is widely shown that the FORM and SORM may be inaccurate for probabilistic problems with nonlinear limit state functions, while the computational cost of MCS can be prohibitively high, especially when involved with time-consuming finite element (FE) analysis. Particularly, the use of finite element models incorporating elements in the interfacial zone in the modeling of chloride transport in repaired concrete is computationally demanding (Beaurepaire *et al.*, 2012). This is due to several factors such as the simultaneous presence of two chloride diffusion mechanisms, the need to model the temporal progression of chloride concentration across multiple repairs, the consideration of different repair methods, and the presence of uncertainties in several random variables. Hence, another sampling method called Latin Hypercube Sampling (LHS) was adopted to overcome those constraints. The LHS technique facilitates an even distribution of sample points across the entire spectrum of possible values, resulting in a reduction in the number of samplings required to obtain satisfactory results. By covering the full spectrum of each random variable, LHS guarantees comprehensive coverage (Bastidas-Arteaga *et al.*, 2011).

LHS has been shown to be a more efficient alternative to basic random sampling in a variety of contexts, making it a popular and robust approach for designing experiments and probabilistic analyses to assess the probability of failure (Olsson, Sandberg and Dahlblom, 2003; Dastmalchi and Burton, 2021). Due to its ease of implementation and reasonable time, LHS is used in this study to perform a probabilistic analysis of maintenance modeling related to one-dimensional chloride penetration. More information on this method can be found in Appendix A.

However, as far as the two-dimensional model of chloride ingress is concerned, the application of simulation-based methods such as LHS to perform probabilistic maintenance modeling remains a challenge due to the significant computational cost. In this case, surrogate-based methods provide a suitable alternative that is commonly used in engineering problems to mimic the behavior of the response metamodels. These methods, such as the response surface method, artificial neural networks (ANNs), Kriging, and the polynomial chaos expansion, can be used to approximate the results of the time-intensive finite element simulations with reduced computational requirements. Moreover, these metamodels are particularly well-suited for reliability analysis, which demands numerous calculations of the performance function (Sudret, 2014; Hariri-Ardebili and Sudret, 2020).

The choice of method depends on the complexity of the system to be modeled, the available data, and the desired level of accuracy and computational efficiency. Among these methods, PCE is chosen because it can capture nonlinear relationships between input variables and output responses without requiring excessive model evaluations (Sudret, 2014), especially for a complicated system of maintenance models subjected to two-dimensional chloride ingress. Additionally, the PCE can be used to perform sensitivity analyses to quantify the effects of input variables on output. The Sobol indices, for example, can be calculated from the PCE coefficients to determine the importance of each input variable in shaping the response. This method is particularly useful for high-dimensional problems where conventional sensitivity analysis methods, such as Monte Carlo simulation, are not feasible due to the high computational cost (Deman *et al.*, 2016).

Therefore, this chapter uses the LHS method to perform a probabilistic assessment of a 1D maintenance model and the PCE method to evaluate a probabilistic 2D maintenance model for chloride-induced corrosion in repaired concrete. Their detailed applications are presented in the next sections.

3.2.3. Framework of probabilistic maintenance modeling

This section presents the framework for probabilistic maintenance modeling. Figure 3.1 illustrates the proposed approaches with a flowchart. It is composed of two main loops. The

external loop illustrates the probabilistic strategy of using LHS to generate the required number of random variables, or constructing a PCE-based surrogate model based on a limited number of simulations; while the internal loop represents the numerical code in FreeFEM++ used to model chloride transport in both existing and repaired concrete. In the internal loop, the “repair criterion” is set command based on two main approaches. The first approach is that repair is carried out when the chloride content at the rebar cover reaches a specific critical chloride value. Meanwhile, the second approach considers the application of pre-scheduled repair intervals for structures as the condition to perform a repair. These approaches allow us to compute the probability of corrosion initiation, which is one of the main criteria in reliability assessment for corroding RC structures. The calculation flowchart for chloride ingress into concrete after repair is also presented, as already mentioned in Section 2.3.3, Chapter 2. The results of two repair approaches could be the corrosion initiation times or the chloride profiles at given repair time. On this basis, the expected number of repairs, n , is determined.

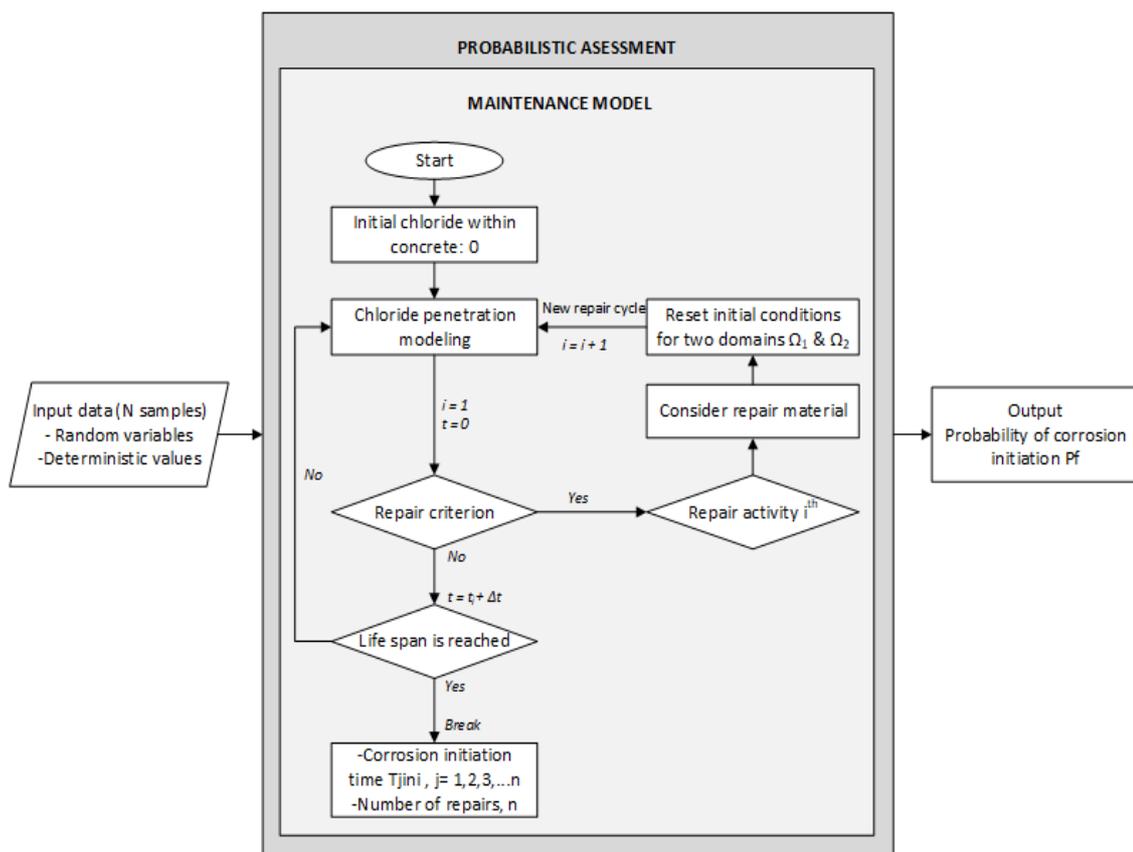


Figure 3.1. Flow chart of probabilistic maintenance modeling

3.2.4. Probabilistic data

In the probabilistic assessment of this problem, there are two main sources of uncertainty to be considered, one related to the chloride diffusivity in concrete and the other related to assessing criteria. In the first group are considered the surface chloride concentration and the concrete

diffusivity parameters ($D_{c,ref}$, U_c , m_c) for original and repair materials (Anna V. Sietta, 1993; Val and Trapper, 2008; Page, Short and El Tarras, 1981; Val, 2006). The surface chloride, C_{fc}^s , which represents the degree of chloride attack, is uncertain due to the randomness of chloride content in surrounding environment (McGee, 2000). The uncertainty in concrete diffusivity parameters, which represent the chloride resistance of concrete material, exists due to different or random properties of concrete material properties (the w/c ratio, the concrete mix proportions, the type of cement, the type of curing). Although concrete structures have the same mix design, their material properties may differ due to different manufacturing processes, quality control and environments. For the second source of uncertainties, cover depth, replacement depth and critical chloride value are the three random variables considered herein. The variations in the cover depth X_c and replacement depth X_p are attributed to construction/repair methods and labor during execution (Vu and Stewart, 2000; Duracrete, 2000). The uncertainty in the critical chloride value, C_{crit} is due to its sensitivity to chemical properties of concrete components such as sand, gravel, and cement. There are many proposals to the probabilistic modeling of C_{crit} ; however this work adopts the values presented in (Duracrete, 2000) to account for the randomness of the critical chloride value.

3.3. Case study 1: LHS -based probabilistic assessment of 1D maintenance model

3.3.1. Problem description

In Chapter 2, the chosen data about material properties, environmental conditions, cover depth, replacement depth and other model parameters used in the deterministic maintenance model allows us to conveniently optimize the inspection and repair plan during the structure lifespan. However, this is not sufficient from an engineering point of view because some of the aforementioned parameters are random. Therefore, a probabilistic analysis should be performed to obtain a comprehensive estimation of chloride penetration in repairing concrete structures

The main goal for this case study is to assess the probability of corrosion initiation for repaired concrete structures by considering the uncertainties described in previous section. The problem description and necessary assumptions are similar with the deterministic example presented in Section 2.4, Chapter 2. Two main approaches are adopted to determine repair conditions and application times, which are named “corrective” and “preventive” maintenance models. The intention of this section is to explore the results obtained from the probabilistic analysis so that a reasonable maintenance plan can be achieved. The probabilistic models of all considered random variables are shown in Table 3, assuming statistical independence between these variables. These random variables have been defined based on previous experimental and

probabilistic studies (Anna V. Saetta, 1993; Val and Trapper, 2008; Page, Short and El Tarras, 1981; Val, 2006; McGee, 2000; Vu and Stewart, 2000; Duracrete, 2000). Since the maintenance model in this case study is related to one-dimensional chloride ingress, the LHS method is chosen to generate the required samples for each random variable.

In addition, previous studies (Rahimi *et al.*, 2014; Petcherdchoo, 2018) have shown that cover rebuilding technique benefits from a replacement depth below cover depth. This is also implied according to the deterministic analysis in section 2.4, chapter 2. Therefore, in this case study, the replacement depth, X_p , which is considered as a random variable, is established to be deeper than the cover thickness. To solve this issue, X_r appears as the extra replacement depth behind the rebar cover. Depending on construction methods and quality control during execution, X_r would be variant and follows a normal distribution with a mean value of 10mm and a coefficient of variation $[COV]_{X_r} = 0.1$. The total replacement depth X_p could be presented as $X_p = X_c + X_r$. The remaining constant parameters used in the maintenance model are presented in Table 2.1, Section 2.2.3, Chapter 2. For the sake of simplicity, it is assumed that the same material is used for construction and repair in all cases.

Table 3.1: Probabilistic models of random variables

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

3.3.2. Results and discussion

3.3.2.1. Corrective maintenance model

The definition of the corrective repair approach was described in Section 3.2.3. The chosen repair strategy herein is to apply a replacement layer approximately 10 mm deeper than the cover thickness. And the probabilistic analysis considers the uncertainty of random variables.

The probabilistic times to repair (or the probabilistic time to corrosion initiation) can be represented in terms of the probability density function (PDF) of repair application time. From

the PDF in Figure 3.2a, the mean and the standard deviation of the first application time is calculated as equal to 15.82 and 15.03 years, respectively. The means of the three following repair times are 11.77, 11.22, and 10.95 years, respectively. To be specific, it takes an average 11.77 years from the first repair application time to trigger the second repair. Similarly, it takes an average 11.22 years from the second repair to trigger the third repair and so on. It is observed that the time for corrosion initiation at the early stage (before any repair measure is applied) is longer than that in the remaining operating period within a 70-year service life. Except the first repair application time of about 16 years, the repair intervals in following years are all approximately 11 years. The corrective repair model does not demonstrate significant variation of chloride diffusion in RC structures between repair periods. The results are in agreement with findings from a previous probabilistic study (Petcherdchoo, 2018). This observation could be useful in developing a maintenance plan for RC structures in marine environments at an early stage.

Considering the PDFs in Figure 3.2, the cumulative density function (CDF) of repair application time during a 70-year service life can be computed to provide the probability of maintenance activity at any year (Figure 3.3). For example, if the time to repair is defined at the year when the probability of corrosion initiation reaches 50% and it is required to carry out the cover replacement measure, the first repair will be performed at year 15.82, the subsequent repairs will be implemented at years 27.59, 38.82, 49.77, and 60.72. However, if 40% probability is chosen to trigger a repair, the first repair time is in year 10.7, and this period will be even shorter, year 8, with 30% probability for the first corrosion occur. Therefore, the outline of maintenance planning based on probabilistic repairing times should consider the cost constraints, environmental impacts as well as the expert judgment of the quality of structures. In addition, an early inspection on site to evaluate the degree of corrosion is highly recommended to support a decision-making process.

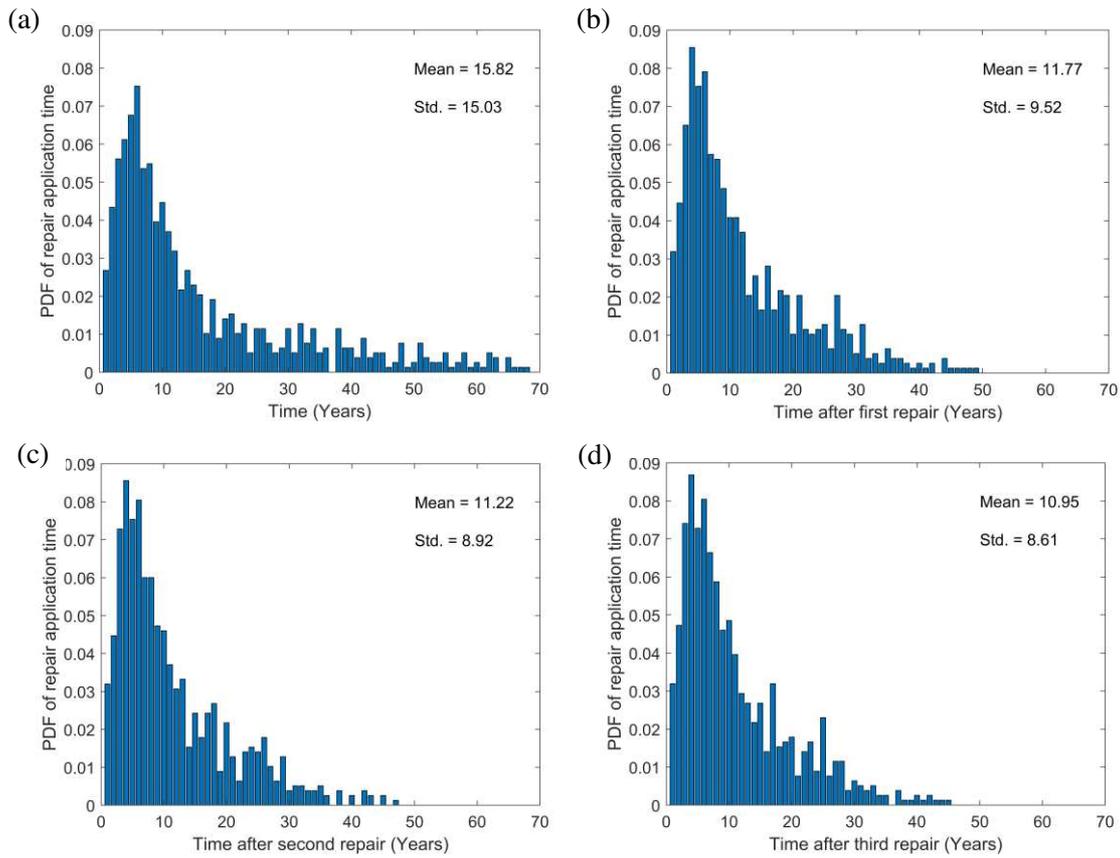


Figure 3.2: PDF of repair application time (a) First repair application (b) Second repair application (c) Third repair application (d) Fourth repair application

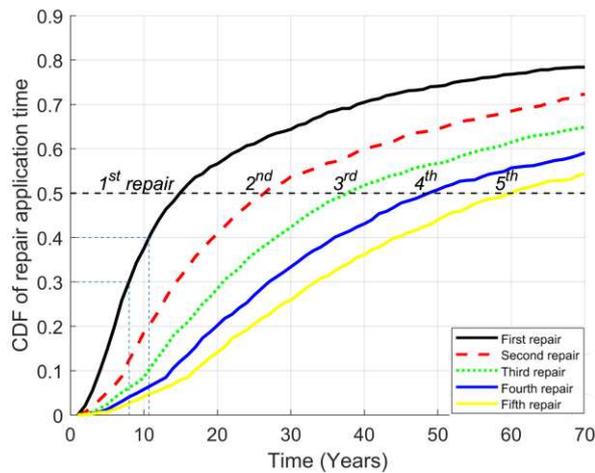


Figure 3.3: CDF of repair application time

Furthermore, the calculation of the total expected number of repairs is introduced. It is assumed that if the cumulative period of repairs exceeds the service life of 70 years, the maintenance could be stopped, hence the expected number of repairs would be determined. As shown in Table 3.2, with the repair strategy of $X_p = 60$ mm replacement depth, on average five repairs are estimated during the structure’s lifetime. Additional repair strategies could also have been

explored in this study such as applying different replacement depths or using better-quality materials. For instance, recent studies (Petcherdchoo, 2018; Lehner, Hornáková and Hrabová, 2021) proved that self-compacting concrete is more sensitive to the values of the diffusion coefficient, which can be used as an indicator of material quality.

Table 3.2: 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		

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

3.3.2.2. Preventive maintenance model

This probabilistic model concerns the maintenance policy of applying a scheduled repair interval at a given repair depth for RC structures as the condition to perform a repair. For instance, the repair application will be implemented immediately whenever the structure lifetime reaches a specific time of repair. Within a 70-year service life, the repair intervals (the time between two repairs) suggested in this example are 15, 10, 10, 10, 10, and 10 years, respectively (i.e. in years 15, 25, 35, 45, 55, and 65). Again, the chosen repair strategy is to apply a replacement layer approximately 10 mm deeper than the cover thickness. These chosen intervals are not randomly determined, but they are based on the results obtained from the corrective repair model, that the scheduled repair times are determined to be less than the mean values of repair application times. The mean repair time value could be approximated as the time to reach a probability of 0.5 in Figure 3.3. Since this approximation should reduce the risks of corrosion initiation, this probabilistic model is referred as a preventive repair model. Other threshold probabilities could be selected depending on the risks allowed for the end user. For instance, small probabilities are related to reinforced concrete assets with higher consequences in case of failure and vice-versa. The selection of the repair depth will mainly influence the probability of corrosion initiation after the first repair. The effects of considering different repair schedules and depths for preventive repair will be illustrated and discussed in this section.

Figure 3.4 presents the simulated time-dependent chloride concentration profiles at the critical depth. It is noted that the cover depth and the replacement concrete depth are considered

random. Thus, for each sample, the chloride content is captured at varying cover depths and is compared with the random value of threshold chloride concentration for corrosion initiation. Eqs. (3.1) to (3.3) are applied in this case to compute the probability of corrosion initiation as shown in Figure 3.5. The chloride profiles of all samples as well as the mean profile, the lower and upper bounds of chloride content are presented in Figure 3.4. It is observed that at the years of repair (i.e. year 15, 25, 35, 45, 55, and 65) the mean chloride content is higher than the mean value of the chloride limit to start corrosion (2 kg/m^3). However, Figure 3.5 shows that the probability of corrosion occurring would be 51.9% only at the first repair time (year 15), while the probabilities are below 50% for all remaining repair periods. Hence, the assessment of corrosion risk and the reliability estimation for a maintenance plan based on mean values can be misleading in this case.

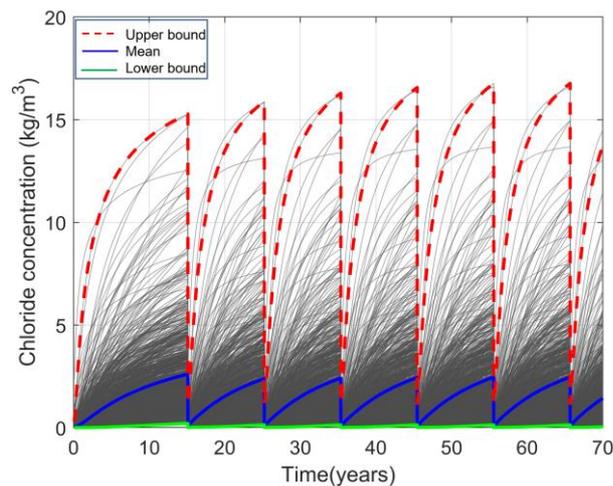


Figure 3.4: Time-dependent chloride concentration profiles at rebar cover

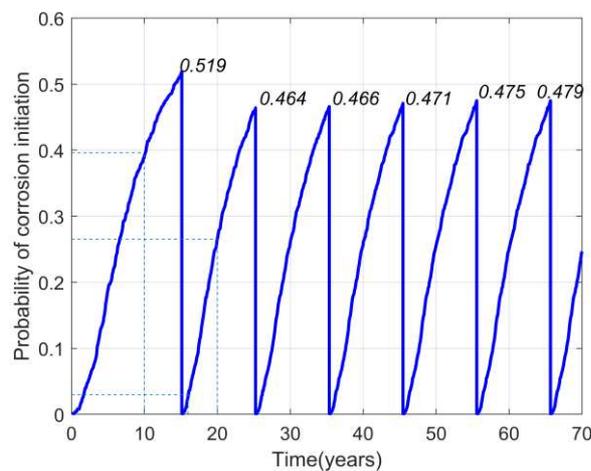


Figure 3.5: Probability of corrosion initiation

In Figure 3.5, the probability of corrosion initiation at any year can be recorded. For example, in year 10 (5 years before the first repair), there is a 39.6% possibility of corrosion occurring, while its probabilities in year 16 (1 year after the first repair), year 20 (5 year after the first

repair), and year 25 (10 year after the first repair) are 3%, 26.5% and 46.4% respectively. It can be concluded that the further away from the most recent repair, the higher the probability of corrosion.

Figure 3.6 shows the chloride profiles at (a) one year after the first repair, (b) 5 years after the first repair, and (c) 10 years after the first repair. In Figure 3.6a, the chloride content in year 16 (1 year after the first repair) is illustrated by curves that fluctuate in value around the interfacial position between two concrete layers, or at $x = X_p$. This is due to the redistribution of chloride ions existing in the original concrete to the new concrete layer after the first repair. Moreover, at year 16, the mean of chloride content at the position of rebar cover is much lower than the mean of critical value (2 kg/m^3), due to the effective chloride uptake of new concrete layer in the early years after repair. For the rest of survey years, the decrease of chloride concentration versus depth tend to remain (Figures 3.6b, c). It is noticed that in year 25 (10 years after the first repair), the mean of chloride content at the critical depth is over the average critical value, it is when the second repair should be scheduled.

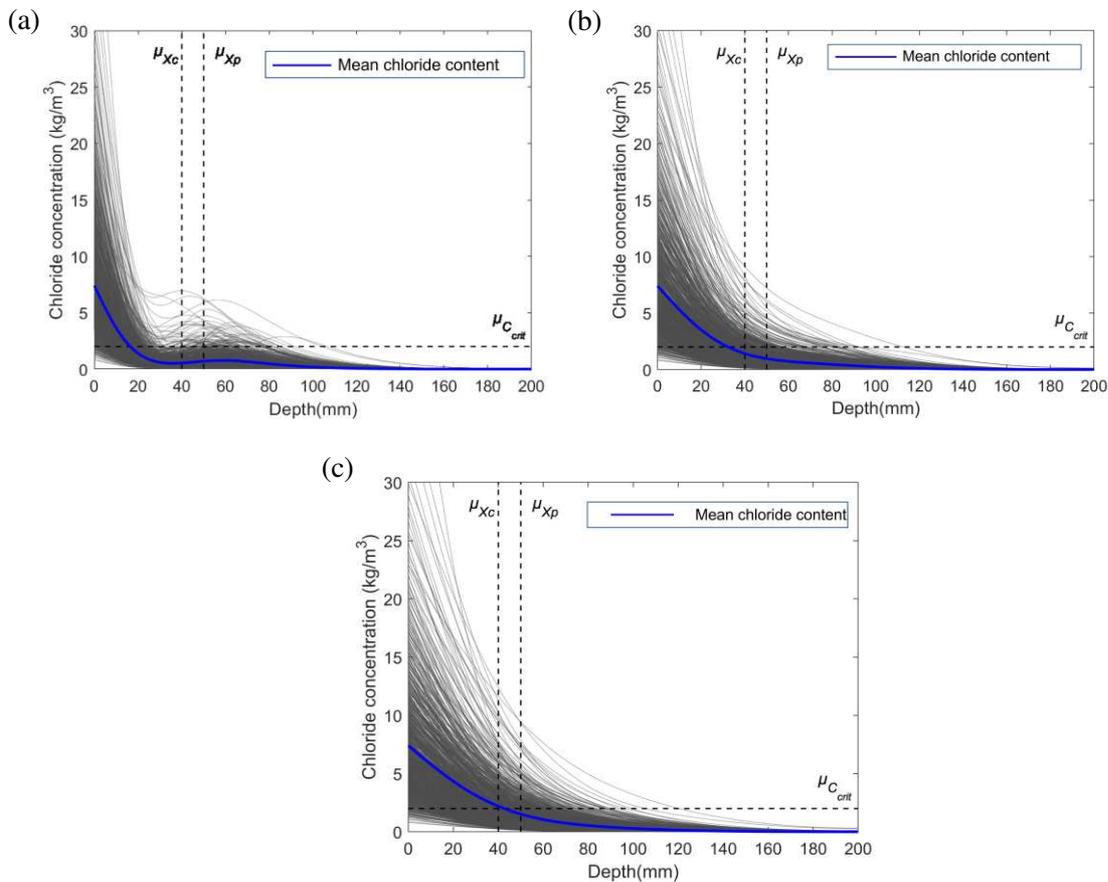


Figure 3.6: Chloride profiles (a) one year after the first repair (b) 5 years after the first repair (c) 10 years after the first repair

From Figure 3.6, the PDF of chloride content at the cover depth can be determined, as shown in Figure 3.7a, b and c. The mean line of critical chloride value (2kg/m^3) is considered as the landmark value for comparison. It is observed that the year with more PDF values being to the right of the threshold value will have a higher probability of corrosion initiation because it represents that the chloride content exceeds the limit state. Accordingly in a 10-year repair cycle from year 15 to year 25, the probability of starting corrosion at year 16 (1 year after the first repair) is lowest (Figure 3.7a), followed by year 20 (5 year after the first repair) (Figure 3.7b) and 25 (10 year after the first repair) (Figure 3.7c). This is consistent and compatible with Figure 3.5.

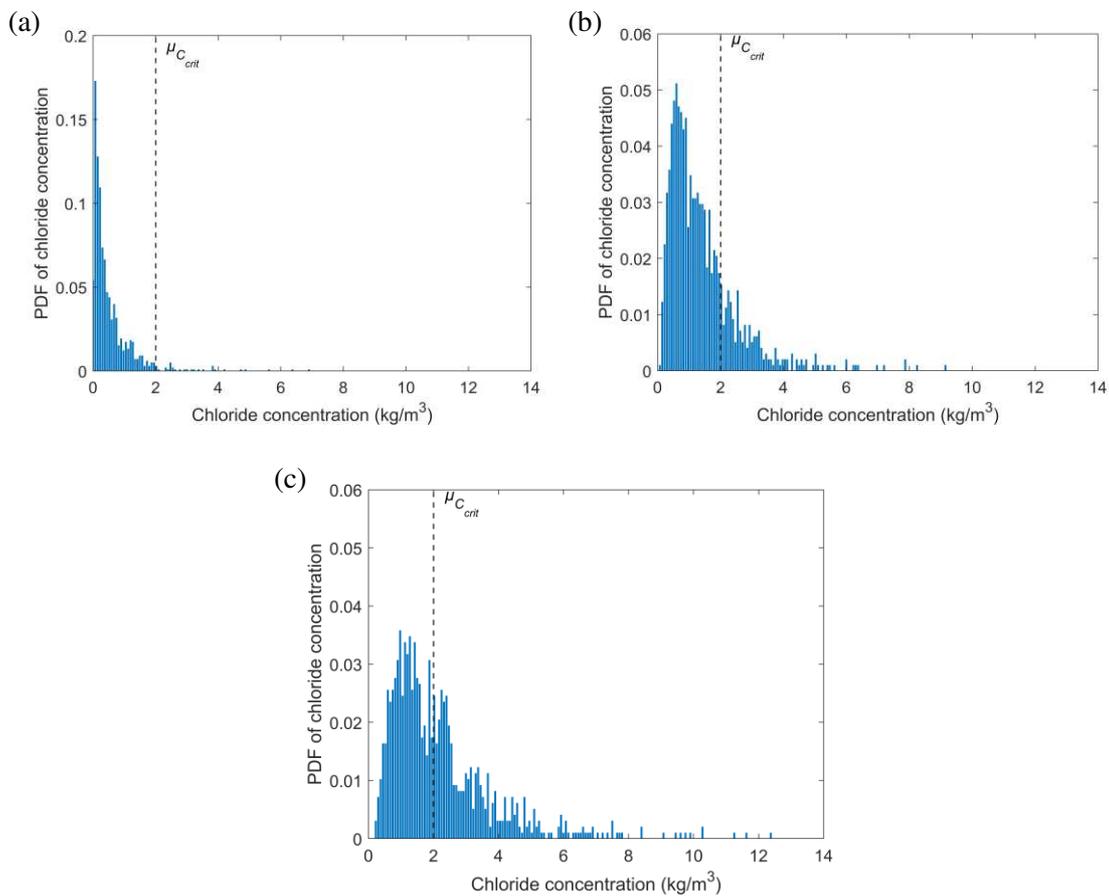


Figure 3.7: PDF of chloride content at cover depth (a) one year after the first repair (b) 5 years after the first repair (c) 10 years after the first repair

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 3.8 shows the probability of corrosion initiation when applying different preventive repair strategies, including (a) 10-year repair intervals, (b) 20 mm extra replacement depth, and (c) 10-year repair intervals and 20 mm extra replacement depth. In Figure 3.8a, 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 3.5. 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 3.8b with the original repair intervals but deeper extra concrete patch of 20mm ($\mu_{Xr} = 20$ mm, $[COV]_{Xr} = 0.1$), 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 $\mu_{Xr} = 10$ mm. Hence, if both strategies are applied for RC structures, the probability of corrosion initiation would be lower (Figure 3.8c). However, other factors, such as maintenance costs and environmental impacts, should also be involved in infrastructure life cycle assessment to formulate a cost-effective and environmentally friendly maintenance strategy.

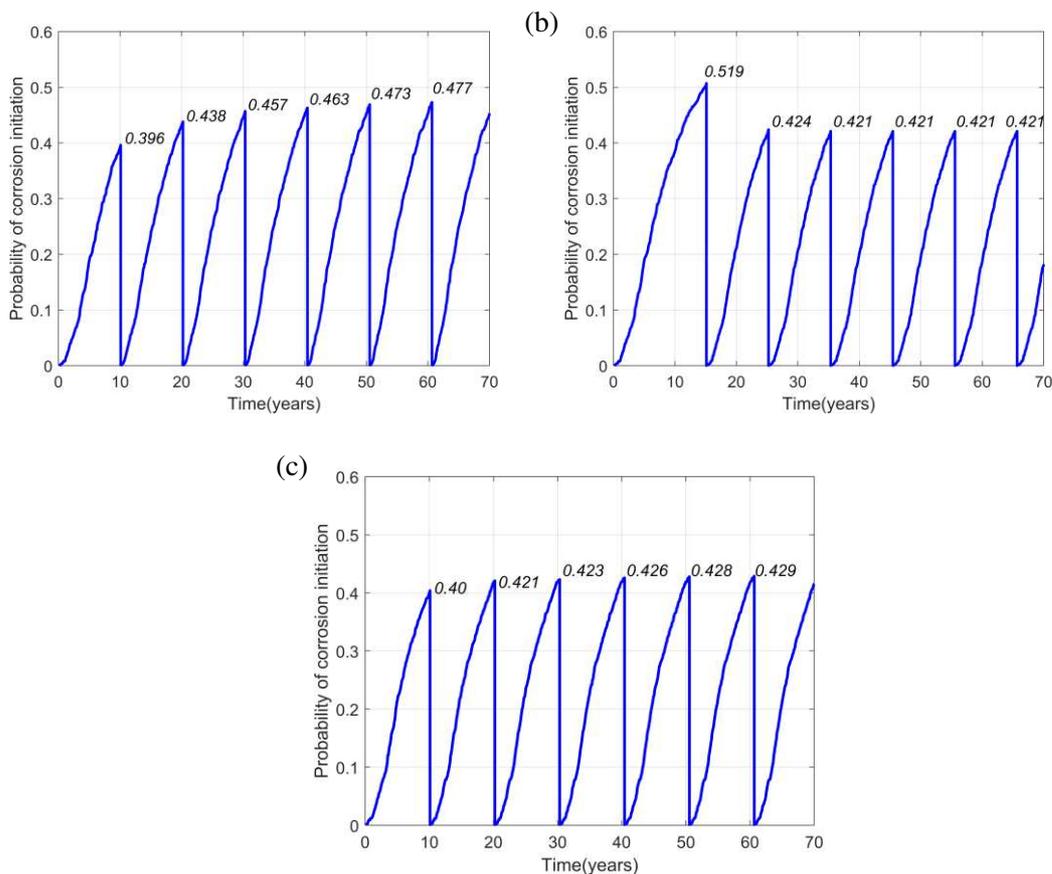


Figure 3.8: Probability of corrosion initiation of two cases (a) 10-year repair intervals (b) 20mm extra replacement depth (c) 10-year repair intervals and 20 mm extra replacement depth

This case study performed a comprehensive probabilistic assessment of the lifetime of repaired RC structures using corrective and proactive maintenance models, combining Latin hypercube sampling, finite element, and finite difference techniques. In particular, the preventive

maintenance model, which involves the implementation of predetermined repair schedules for corroded structures, appears to be an efficient approach to control the risk of corrosion initiation through improved techniques such as shortening the repair time or installing a deeper concrete replacement section. However, this case study only deals with the problem of one dimensional chloride penetration in repaired concrete. Further research is needed to investigate the modeling of two-dimensional chloride penetration, the use of different repair materials, and the optimization of maintenance schedule with a probabilistic approach. Although the LHS method was used to reduce the number of samples required, this probabilistic approach is still very time consuming and involves high computational costs. For example, it took 144 hours to run 1000 simulations of a 1D maintenance model with multiple repairs over a 70-year period. It is even more time-consuming with a high dimensional chloride ingress, or a 2D maintenance model. Therefore, improvement of the probabilistic method is necessary to save time and to perform a global sensitivity analysis of the parameters related to maintenance and repair.

3.4. Surrogate-based probabilistic assessment of maintenance model

In this section, a PCE approach is adopted to propagate uncertainties in maintenance modeling and estimate the probability of corrosion initiation at a given time, allowing for the development of an appropriate inspection and maintenance schedule. Additionally, the PCE can be used to perform sensitivity analyses to quantify the effects of input variables on output. The mathematical setting of this method is presented below.

3.4.1. Polynomial Chaos Expansions

The Polynomial Chaos Expansions (PCE) method aims to create a metamodel (or surrogate model) $\mathcal{M}^{\text{PCE}}(\mathbf{X})$ for the chloride ingress problem by representing the output in terms of probability space (Sudret, 2014). The input parameters are considered statistically independent and represented by a random vector $\mathbf{X} = \{X_1, X_2, \dots, X_M\}$. Due to the uncertainties in \mathbf{X} , the output such as the chloride concentration at the critical point, $C_{fc} = \mathcal{M}(\mathbf{X})$ is also random and can be approximated using a PCE formula:

$$\widehat{C}_{fc} = \mathcal{M}^{\text{PCE}}(\mathbf{X}) = \sum_{\alpha \in \mathcal{A}} y_{\alpha} \Psi_{\alpha}(\mathbf{X}) \quad (3.4)$$

This approximation is represented as the sum of multivariate polynomials, $\Psi_{\alpha}(\mathbf{X})$, multiplied by unknown expansion coefficients, $\{y_{\alpha} \in \mathbb{R}, \alpha \in \mathcal{A}\}$. The multi-indices, $\alpha \in \mathcal{A}$, are used to identify the components of the polynomials, and the set of multi-indices, \mathcal{A} , is defined by a truncation scheme that bounds the interactions between the input variables. The truncation scheme determines the degree of polynomials to be retained in the expansion, controlling the level of detail of the approximation.

$$\mathcal{A} = \{\alpha \in \mathbb{N}^M, \|\alpha\|_\beta\} = \left(\sum_{i=1}^M \alpha_i^\beta \right)^{1/\beta} \leq p, (0 < \beta \leq 1) \quad (3.5)$$

The coefficients, $\{y_\alpha \in \mathbb{R}, \alpha \in \mathcal{A}\}$, are computed using non-intrusive regression methods, in particular the least-square minimization. The set of unknown coefficients can be determined by minimizing the mean-square error of the polynomial expansion based on N samples of the input vector X , known as the experimental design (ED). The least angle regression (LAR) can also be used to obtain a sparse PCE, in which only polynomials with the largest impact on the model response are retained, while others are set to zero.

$$\widehat{y}_\alpha = \underset{y_\alpha \in \mathbb{R}^{\text{card}(\mathcal{A})}}{\text{argmin}} \frac{1}{N} \sum_{i=1}^N (\mathcal{M}(X_i) - \sum_{\alpha \in \mathcal{A}} y_\alpha \Psi_\alpha(X_i))^2 \quad (3.6)$$

The constructed Polynomial Chaos Expansion (PCE) can serve as a meta-model for the target output when it is not possible to evaluate a substantial number of model responses. To evaluate the accuracy of PCE, the Leave-One-Out (LOO) error is commonly used as a reliable measure of accuracy that provides a reasonable estimate of error at a manageable computational cost. The formula for the relative LOO error is presented as follows.

$$\varepsilon_{\text{LOO}} = \sum_{i=1}^N \left(\frac{\mathcal{M}(X_i) - \mathcal{M}^{\text{PC}}(X_i)}{1 - h_i} \right)^2 / \sum_{i=1}^N (\mathcal{M}(X_i) - \widehat{\mu}_Y)^2 \quad (3.7)$$

where h_i is the i^{th} diagonal term of matrix $\Psi(\Psi^T \Psi)^{-1} \Psi^T$, with $\Psi = \{\Psi_{ij} = \Psi_j(X_i)\}$

and $\widehat{\mu}_Y = \widehat{\mu}_{T_{\text{ini}}} = \frac{1}{N} \sum_{i=1}^N \mathcal{M}(X_i)$

In addition to providing a surrogate model, the PCE can also be used to obtain statistical moments of the response, specifically the mean and variance. Since the PC expansion is constructed in an orthonormal space, the statistical moments can be easily derived from the expansion coefficients. The mean of the response is equal to the first expansion coefficient y_0 and the variance is equal to the sum of the squares of the expansion coefficients, except for the mean.

$$\mu_{T_{\text{ini}}} = E(\mathcal{M}^{\text{PC}}(\mathbf{X})) = y_0 \quad (3.8)$$

$$\text{and} \quad \sigma_{T_{\text{ini}}}^2 = \text{Var}(\mathcal{M}^{\text{PC}}(\mathbf{X})) = \sum_{\alpha \in \mathcal{A}} y_\alpha^2 (\alpha \neq 0) \quad (3.9)$$

Moreover, the polynomial nature of $\widehat{C}_{\text{fc}} = \sum_{\alpha \in \mathcal{A}} y_\alpha \Psi_\alpha(\mathbf{X})$ allows the probability density function of C_{fc} and the probability of corrosion initiation as defined by Eq. (3.3) to be easily estimated by an extensive simulation of the metamodel \mathcal{M}^{PC} , such as Monte-Carlo simulations.

3.4.2. PCE-based Global Sensitivity Analysis

This section presents a PCE-based global sensitivity analysis that aims to determine how changes in each input variable X_i affect the uncertainty of the model output C_{fc} . A variance-based approach using Sobol's sensitivity indices is used to decompose the output variance into contributions from the input variables. The Sobol's indices can be challenging to compute for complex numerical models, but the PCE technique can simplify this process. The PCE approximates the model with a PC surrogate so that sensitivity analysis can be performed through post-processing the polynomial expansion coefficients (Sudret, 2014).

Sobol's decomposition of the PCE can be expressed as a series of polynomial terms describing the relationships between the input variables and the output. The first-order sensitivity index (S_i) evaluates the individual contribution of each input parameter X_i to the variance of the output, assuming that all higher-order interactions between input parameters are negligible. The Sobol total index (S_i^T) quantifies the cumulative effect of a given input parameter on the variance of output, which considers all interaction effects between input parameters, including higher order interactions, and serves as a measure of the importance of the input parameter in explaining the variability of the output (Sudret, 2014).

The Sobol's decomposition of the PCE can be written as:

$$\mathcal{M}^{PC}(X) = y_0 + \sum_{i=1}^M \sum_{a \in I_i} y_a \Psi_a(X_i) + \sum_{1 \leq i_1 \leq i_2 \leq M} \sum_{a \in I_{i_1, i_2}} y_a \Psi_a(X_{i_1}, X_{i_2}) + \dots + \sum_{1 \leq i_1 \dots \leq i_s \leq M} \sum_{a \in I_{i_1, \dots, i_s}} y_a \Psi_a(X_{i_1}, \dots, X_{i_s}) + \dots + \sum_{a \in I_{i_1, \dots, i_M}} y_a \Psi_a(X_{i_1}, \dots, X_{i_M}) \quad (3.10)$$

where I_{i_1, \dots, i_s} is the set of non-zero indices $(i_1, \dots, i_s) \in \mathcal{A}$.

$$\text{The first-order sensitivity index: } S_i = \frac{\sum_{a \in I_i} y_a^2}{\sigma_{Tini}^2} \quad (3.11)$$

$$\text{The total sensitivity index: } S_i^T = \frac{\sum_{a \in I_i^*} y_a^2}{\sigma_{Tini}^2} \quad (3.12)$$

where I_i^* denotes the set of all indices with nonzero i^{th} component.

3.5. Case study 2: PCE-based probabilistic assessment for 1D maintenance model

3.5.1. Problem description

The purpose of this case study is to perform a probabilistic assessment of maintenance modeling based on a surrogate model using PCE technique. The probabilistic assessment is again performed on the problem presented in Case study 1, Section 3.3, to investigate the efficiency of the newly proposed probabilistic method. The results obtained in the first example based on LHS could be a reference for the comparison and validation of the metamodels. Similarly, two maintenance models are studied, namely corrective and preventive maintenance models. The uncertain parameters are the surface chloride concentration C_{fc}^S , concrete diffusivity parameters

($D_{c,ref}$, U_c , m_c) for original and repair materials, depth of cover (X_c), replacement depth (X_p) and critical chloride value (C_{crit}).

3.5.2. Results and discussions

To prove the efficiency of the PCE metamodel, two crucial questions arise: 1) Can the PCE metamodel be used to quantify uncertainty in chloride ingress problems? If so, 2) how much computational time/cost is saved by using the PCE method?

To answer the first question, it is essential to compare the estimated responses of the problem based on PCE model with those of the finite element model and then evaluate the leave-one-out error (ϵ_{LOO}). On the other hand, the minimum number of simulations required by the PCE approach to construct a metamodel with the required accuracy is explicitly stated as an estimate of the computational cost. First, the number of initial simulations should be given, N_{sim} . Then, different PCE metamodels are created with different sizes of the experimental design, N_{ED} , a subset of the initial simulations. Although it stands to reason that more N_{ED} leads to a better metamodel, the optimal number of simulations for each problem should be explored. The efficiency of the PCE method could be calculated as N_{ED}/N_{sim} .

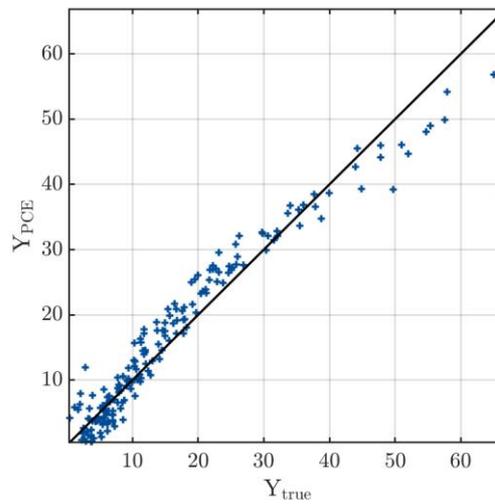
3.5.2.1. Metamodels for corrective maintenance strategy

A surrogate model is developed from the corrective maintenance model using the PCE method. It uses the corrective maintenance model outputs, which are the corrosion initiation time between multiple repairs (T_{ini}), to compute the polynomial chaos basis. The input random vector for PCE calculation is $X = \{D_{c,ref}, U_c, m_c, X_c, X_p, C_{fc}^S, C_{crit}\}$.

The number of initial simulations is set to $N_{sim} = 1000$, in which the samples are generated based on the LHS method (Case study 1). First, an initial set of experimental design points (ED) is generated with $N = 200$ samples corresponding to 200 assessments of the corrective maintenance model. Additional $N_{new} = 100$ samples (from $N_{ED} = 200$ to 400 samples) or 200 samples (from $N_{ED} = 400$ to 1000 samples) are then added sequentially to check the accuracy of the PCE model while increasing the size of the experimental design. Considering the first corrosion initiation time, T_{ini1} , the construction of the PCE model with different sample sizes is examined, as shown in Table 3.3. It is observed that as the ED size is increased from 400 to 1000 samples, the accuracy Q^2 converges at about 0.97. Since it is of interest to limit the number of costly evaluations of the exact corrective maintenance model with acceptable accuracy, an ED with 400 points is considered most appropriate. Figure 3.9 compares the exact values of T_{ini1} obtained by the maintenance model with the surrogate model with $N_{ED} = 400$ samples.

Table 3.3: Accuracy of the PCE model of first corrosion initiation time with different sample sizes

ED	Maximum degree	Mean	STD	LOO error
200	3	26.75	27.58	1.52×10^{-1}
300	3	33.6	46.9	4.4×10^{-2}
400	3	34.0	48.9	3.5×10^{-2}
600	3	34.68	49.1	3.4×10^{-2}
800	3	34.4	48.6	3.3×10^{-2}
1000	3	34.67	49.1	3.4×10^{-2}

**Figure 3.9:** Comparison 1st T_{ini} results obtained from the maintenance model and the PCE ($N_{ED} = 400$ samples)

Based on the surrogate model, a crude MCS with 1,000,000 simulations is performed to estimate the probabilistic times to repair (or the probabilistic time to corrosion initiation), defined here as the time to trigger the corrosion after the previous repair. The PDF and CDF of the first and second repair times are shown in Figure 3.10 and Figure 3.11, respectively. The probabilistic results of the following repairs are similar to those of the second repair, so they are not shown here. Looking at the CDF of repair times in both figures, it is found that the time of first repair is about 16 years (Figure 3.10) and subsequent repair intervals are about 12 years (Figure 3.11) with a 50% probability of corrosion initiation. It should be noted that the CDF of repair time resulting from the PCE-based probabilistic evaluation here and the LHS-based one in Case study 1 are quite consistent, demonstrating the efficiency of the PCE metamodel in saving computational time/cost.

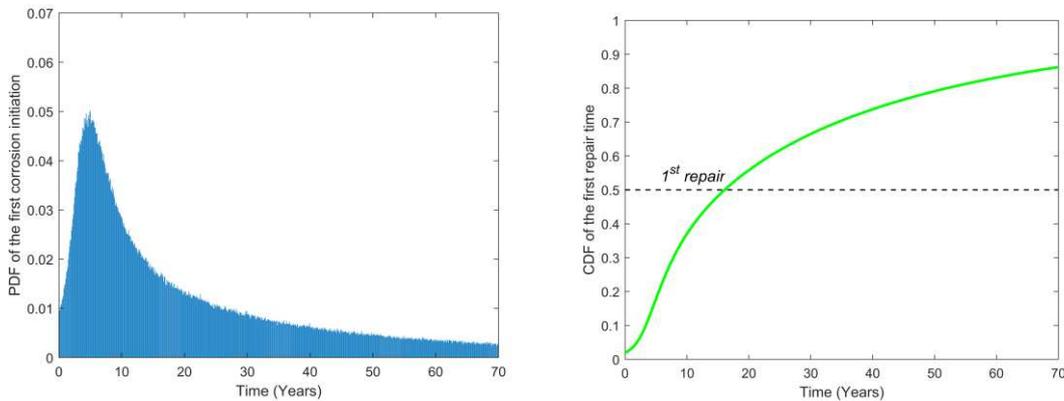


Figure 3.10: PDF and CDF of the first repair time

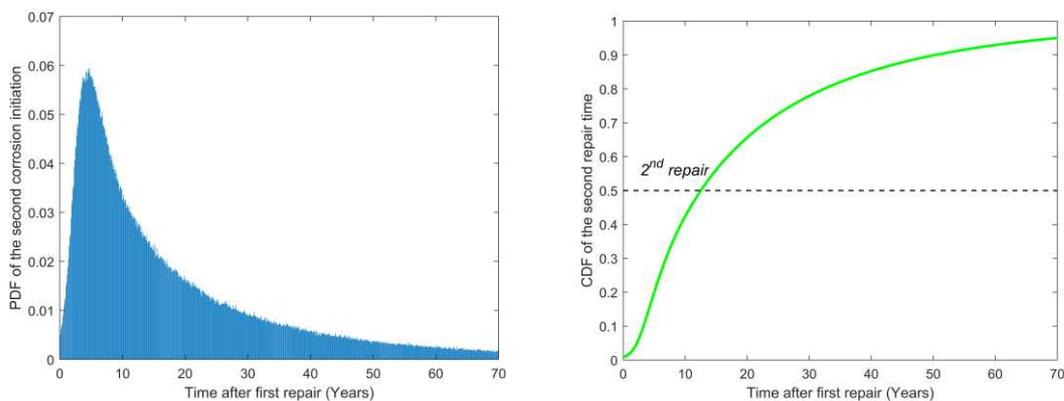


Figure 3.11: PDF and CDF of the second repair time

3.5.2.2. Metamodels for preventive maintenance strategy

A preventive maintenance model is developed with scheduled repairs intervals of 15, 10, 10, 10, and 10 years (called as repair strategy S1). These intervals correspond to repairs occurring in years with an approximate or less than 50% probability of corrosion initiation based on the corrective repair model discussed above. Note that the chosen threshold probability is indicative of risk tolerance and governs the repair schedule.

A surrogate model is developed from the preventive maintenance model using PCE. In this case, the model outputs, which are the chloride concentrations at the cover depth at planned repair times, are used to compute PC expansion. As a result, at each scheduled repair time, such as Year 15 (Y15), Y25, Y35, Y45, Y55, Y65, there is a corresponding PCE model that can estimate the chloride concentration at critical depth to compare with the threshold value for corrosion initiation. The input random vector for PCE calculation is $X = \{D_{c,ref}, U_c, m_c, X_c, X_p, C_{fc}^s\}$.

The procedure for determining the optimal number of experimental designs (ED) for the preventive model is similar to that of the corrective model presented in Section 3.5.2.1. As a

result, the final size of ED is chosen from 400 samples for the development of the surrogate model, achieving an accuracy of 0.999 ($\epsilon_{LOO} < 10^{-3}$). Looking at the chloride concentration at the critical depth at the first repair time Y15, Figure 3.12 compares the exact values of C_{fc} obtained by the maintenance model (Y_{true}) and the surrogate model (Y_{PCE}).

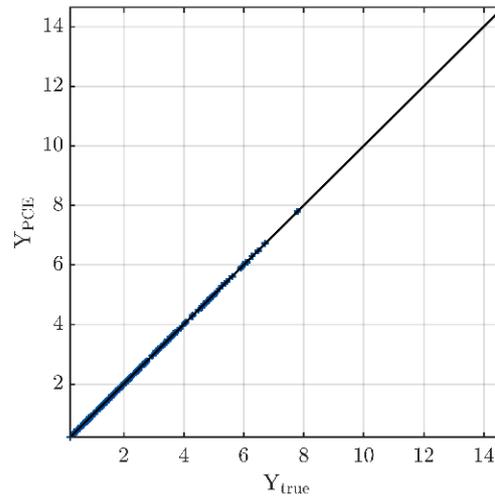


Figure 3.12: Comparison of maintenance model (Y_{true}) and PCE model (Y_{PCE}) for the chloride concentration at critical depth at Y15 (400 samples)

Then MC analysis is performed with 1,000,000 simulations, Table 3.4 shows the MCS results for surrogate models corresponding to different repair times, including the accuracy rate, mean, standard deviation, and median of C_{fc} at the cover depth, and probability of corrosion initiation. It is also observed that the accuracy of the surrogate model at each repair is about 0.999, which meets the target value.

Table 3.4: Accuracy of surrogate models and the probability of corrosion initiation at the repair time, P_{ini}

Repair	Q^2	Mean of C_{fc} (kg/m^3)	SD of C_{fc} (kg/m^3)	Median of C_{fc} (kg/m^3)	P_{ini}
1 st repair_Y15	>0.999	2.55	2.15	1.95	0.499
2 nd repair_Y25	>0.999	2.33	2.11	1.90	0.439
3 rd repair_Y35	>0.999	2.35	2.15	1.90	0.441
4 th repair_Y45	>0.999	2.37	2.18	1.91	0.446
5 th repair_Y55	>0.999	2.39	2.20	1.91	0.449
6 th repair_Y65	>0.999	2.41	2.22	1.92	0.450

Table 3.4 shows that the average chloride contents in the years of repair are higher than the average value of the chloride threshold for corrosion initiation ($2 \text{ kg}/\text{m}^3$), but the median chloride contents are lower than C_{crit} . In addition, at the first repair time (year 15) there is about 50% probability of corrosion initiation; for remaining repairs, is somewhat lower. Therefore,

evaluation of corrosion risk based on mean values is not recommended here. Note that the probabilities of corrosion initiation in Table 3.4 are consistent with the results shown in Figure 3.5 in Case Study 1. This demonstrates that the use of PCE is effective in developing a surrogate model for a preventive maintenance strategy and provides high accuracy at low computational cost.

The PCE-based metamodel could be applied for different maintenance strategies which consider other repair times and/or additional repair methods to reduce the risk of corrosion initiation, such as early repairs and/or deeper replacement depth. Two additional cases are then examined:

- Repair strategy 2 (S2), repeating 10-year repair intervals with similar repair depth as in repair strategy 1, and
- Repair strategy 3 (S3), repair 20 mm deeper than the cover depth and similar repair intervals as in repair strategy 1.

Only $N_{ED} = 400$ simulations of each maintenance strategy are needed to build the metamodels. After that, the procedure for performing the MCS analysis is carried out in a similar manner for repair strategies S2 and S3. Table 3.5 shows the probabilities of corrosion initiation at each repair time for the two cases. For S2, the application of a shorter initial interval reduces the corrosion risk at the time of the first repair and the maximum probability of corrosion occurrence is 45%. For S3, on the other hand, the greater repair depth reduces the probability of corrosion for the subsequent repairs, the maximum P_{ini} is observed at the first planned repair time with 50%.

Table 3.5: Probability of corrosion initiation at the repair time for three repair strategies

S2	Repair	1 st _Y10	2 nd _Y20	3 rd _Y30	4 th _Y40	5 th _Y50	6 th _Y60
	P_{ini}	0.37	0.41	0.43	0.44	0.44	0.45
S3	Repair	1 st _Y15	2 nd _Y25	3 rd _Y35	4 th _Y45	5 th _Y55	6 th _Y65
	P_{ini}	0.50	0.40	0.40	0.40	0.41	0.41

The above results confirm that the use of a metamodel (PCE) is computationally efficient to model maintenance actions such as repair depth and interval and to estimate the associated risks of corrosion initiation. In addition, the results of the PCE model can be used for maintenance schedule optimization, that will be presented in the next example.

3.6. Case study 3: Optimizing maintenance schedule through PCE-based probabilistic assessment

3.6.1. Repair criterion for maintenance planning

In maintenance and repair of RC structures, the selection of the repair criterion is a crucial issue that requires a comprehensive evaluation of various factors such as economic, environmental, practical, and others. In the present study, a proactive repair criterion is established where the repair schedule is pre-determined based on an allowable damage threshold, or acceptable probability of corrosion initiation. This approach can be considered conservative compared to other repair criteria found in the literature (Baji *et al.*, 2020; Ramani, Zhang and Kuang, 2021) that are based on the occurrence of concrete cracks or other damage indicators.

Determining the acceptable probability of corrosion initiation in RC exposed to chloride penetration is influenced by several factors, such as the type of structure, the intended use, and the durability requirements of the structure. It is important to note that a generally accepted criterion for this value has not yet been established. However, in many cases a low probability of corrosion initiation is generally considered acceptable, often less than 10%. For instance, the minimum reliability index recommended by the Eurocode for a serviceability limit state (corrosion initiation) is 1.5 (BS EN 1992; BS EN 206-1:2013), this corresponds to a probability of corrosion initiation of approximately 7%. According to the fib-Bulletin 34 (2006), a reliability index $\beta = 1.3$ (corresponding to a probability of corrosion initiation), could be accepted for the deterioration limit state (or initiation limit state). In view of this available information, a threshold probability of corrosion initiation of 10% is chosen in this study to evaluate and modify preventive maintenance strategies for corroded structures, in which the corrosion risks during planned repair periods are calculated based on Eqs. (3.1)-(3.3) presented in Section 3.2.1.

3.6.2. Problem description

The objective of this case study is to evaluate the potential of probabilistic modeling in maintenance schedule optimization of RC structures subjected to chloride-induced corrosion. Based on the PCE method presented in Section 3.4, a maintenance metamodel is developed that can estimate the probability of corrosion initiation at specific repair times. The probabilistic model proposed here is illustrated using a concrete column with a cross section of 0.4 m x 0.4 m. The column is located in seawater into which chlorides penetrate in two orthogonal directions. To meet the durability requirements of RC structures exposed to extreme marine environments (e.g., splash and tidal zone), a design cover X_c of 65 mm is proposed for a new structure according to Eurocode 2 (BS EN 1992) (see in Figure 3.13).

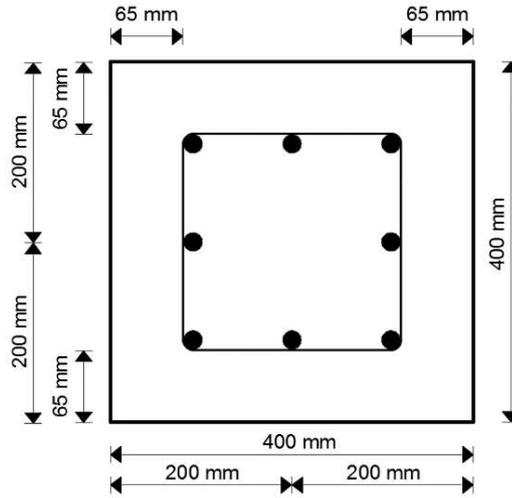


Figure 3.13 : Cross-sectional of square column

Due to the symmetrical boundary conditions of a square column working in seawater, partial modeling on the truncated surface of 0.2 m x 0.2 m is used in this study, assuming that no reinforcing bars are present (Figure 3.14). The chosen repair method is to replace the chloride-contaminated concrete cover with a new concrete layer, using the circular replacement method, in which a circular curve delineates the boundary line between the two materials. This repair method is recommended because it has the potential to restore damaged structural elements to their original condition (as shown in the deterministic example of 2D maintenance model in Section 2.5, Chapter 2). Figure 3.14 illustrates this repair method on the examined column, where the concrete cover X_c and concrete replacement area are treated randomly due to the variations in construction/repair methods and labor during execution.

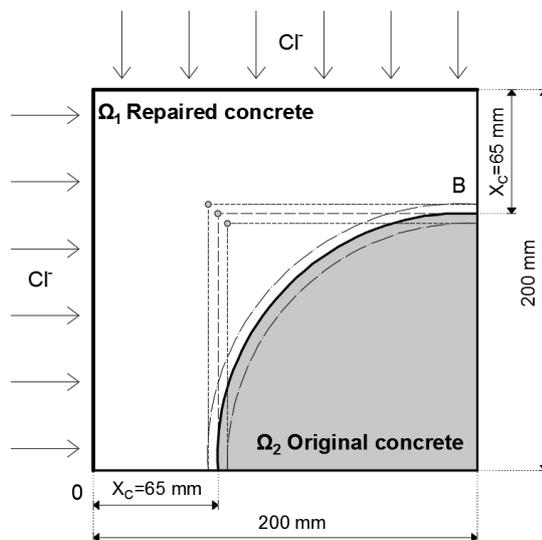


Figure 3.14: Circular concrete replacement method

To ensure the durability of the structure over its 75-year lifespan, a proactive maintenance plan with regular repairs is recommended. The components are located in an extreme marine environment characterized by higher ambient chloride concentrations. The average ambient temperature and relative humidity are 15°C and 0.70, respectively. The chloride concentration at the surface, C_{fc}^s , is considered as a random variable following a lognormal distribution with a mean value of 5.21 kg/m³ and a coefficient of variation (COV) of 0.25 (Zacchei and Bastidas-Arteaga, 2022b). The composition of the original concrete includes ordinary Portland cement (OPC) with 8% C₃A and 400 kg/m³ and a water-cement ratio of 0.5. The time at which corrosion begins is determined by the attainment of the critical chloride content at critical locations, $C_{crit} = 2$ kg/m³ (equivalent to 0.5% of the cement mass), although C_{crit} is assumed to be a random variable in this work because of the inherent uncertainties in the material properties of concrete (Duracrete, 2000). The probabilistic models of the selected random variables are presented in Table 3.6, assuming statistical independence between these variables. These random variables have been defined based on previous experimental and probabilistic studies.

Table 3.6: Probabilistic models of random variables.

Variables	Unit	Mean	COV	Distribution
$D_{c,ref}$	m ² /s	3×10^{-11}	0.20	Log-normal
$D_{c,rep1}$ (S1)	m ² /s	3×10^{-11}	0.20	Log-normal
$D_{c,rep2}$ (S2)	m ² /s	2.5×10^{-11}	0.15	Log-normal
U_c	kJ/mol	41.8	0.10	Beta on [32,44.6]
m_c		0.15	0.30	Beta on [0,1]
C_{crit}	kg/m ³	2	0.20	Normal
C_{fc}^s	kg/m ³	5.21	0.25	Log-normal
X_c	m	0.065	0.2	Normal (truncated at 10 mm)

Concrete diffusivity parameters, such as U_c , m_c and $D_{c,ref}$ are treated as uncertain (Anna V. Saetta, 1993; Martín-Pérez, Pantazopoulou and Thomas, 2001; Page, Short and El Tarras, 1981; Val, 2006). In this study, two main repair strategies S1 and S2 are proposed, which includes the chloride diffusion coefficients of two repair materials, $D_{c,rep1}$ and $D_{c,rep2}$, assuming that the first repair concrete is of similar quality and properties to the original material (S1 strategy) and the second repair concrete has a higher diffusion resistance due to the use of wet shotcrete (S2 strategy) (Bastidas-Arteaga and Schoefs, 2012). It should be emphasized that while these coefficients are used to study the effects of repair quality, the conclusions regarding repair techniques are not generalizable across the board. The random variable denoted X_c embodies the stochastic variations in the depth of cover and the amount of concrete replaced during the

performance of construction or repair work, as shown in Figure 3.14 (BS EN 1992; Bastidas-Arteaga and Stewart, 2015). Similarly with other maintenance models, the remaining parameters, which are considered constant, are listed in Table 2.1 in Chapter 2.

3.6.3. Results and discussions

The 2D maintenance model is implemented on a corroded RC column, where the concrete is replaced at regular intervals in a circular curve, using materials S1 and S2. This method guarantees timely repairs when the structure approaches a certain point in time. For RC structural members exposed to two-dimensional chloride ingress, such as columns and beams, the corners of the concrete are considered the most important locations to detect the onset of corrosion. Therefore, the chloride concentration accumulated at the column corner is investigated in all cases of the study.

For each of the repair strategies S1 and S2, several proactive repair plans are proposed. Their probability analysis is performed using the PCE method, with the aim of achieving a corrosion initiation probability close to the allowable damage threshold to obtain an optimal plan. To gain a better understanding of the construction of a maintenance metamodel, a given repair plan is first analyzed for the two repair cases S1 and S2. The following part focuses on the use of the PCE model to optimize the maintenance schedule. Finally, a sensitivity analysis based on Sobol indices are explained.

3.6.3.1. Construction of a maintenance metamodel

A 10-year repair interval is first proposed for S1 and S2 (referred to as the S1-10yr and S2-10yr strategies), during a 75-year life of the structures, leading to repairs in years 10, 20, 30, 40, 50, 60, and 70. It is noted that performing all necessary repairs for a 75-year period through a single deterministic run of the 2D maintenance model takes about 30 minutes, making conventional probabilistic techniques for reliability impractical because of the large number of deterministic model calls required. Therefore, our goal is to first construct the PCE model for the S1-10yr repair strategy and determine the optimal size of the experimental design, which can be used to create metamodels for other repair strategies. The PCE models generated from the chloride concentrations according to the random input vector $\mathbf{X} = \{D_{ref}, D_{c,rep1}, U_c, m_c, C_{fc}^s, X_c\}$ are developed using LAR method, setting β to 0.7 (Eq. 3.5) and choosing a target accuracy of 0.999, which corresponds to the LOO error of about 10^{-3} (Eq. 3.7).

The PCE model for chloride concentration at the first repair time, Y10, was built with an initial set of 50 experimental design samples, and another 50 samples were added sequentially until the specified accuracy was achieved. The accuracy and computational efficiency of the

PCE model are examined by increasing the initial sample size, as shown in Table 3.7 and Figure 3.15.

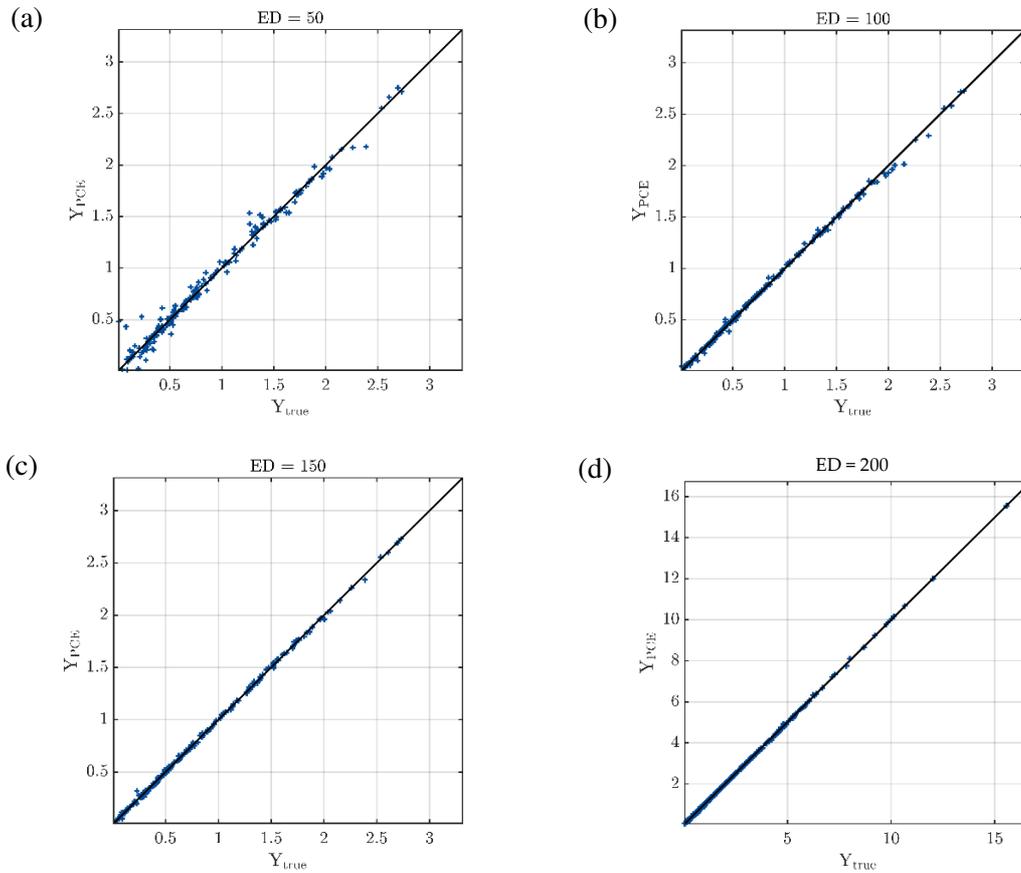


Figure 3.15: Sample prediction with $N_{ED} = 50, 100, 150, 200$ samples

Table 3.7: Accuracy of the PCE model with different sample sizes

ED	Maximum degree	Mean	STD	LOO error
50	3	0.72	0.66	8.6×10^{-3}
100	5	0.73	0.66	2.7×10^{-3}
150	6	0.72	0.67	1.5×10^{-3}
200	6	0.73	0.67	5×10^{-4}

From Table 3.7, it is noted that the mean and standard deviation (STD) are almost stable even after $ED = 50$, but when the number of samples increases, the error LOO decreases, which leads to the fact that the accuracy of the PCE model also increases. Therefore, based on this trade-off, the final experimental design size for the metamodel development is set at 200 samples to achieve a target accuracy of 0.999 ($\epsilon_{LOO} = 5 \times 10^{-4}$). The corresponding PC approximation has a polynomial order of $p = 6$ and contains 96 basis elements. It can be concluded that using PCE is

an effective approach for constructing metamodels for proactive maintenance plans that provides high accuracy at low computational cost.

Similarly, PCE models can be built using chloride concentration data at various times during the life of the structure. Monte Carlo simulations (MCS) can then be performed on these metamodels to determine the probability of corrosion initiation (p_f) by comparing the random output to the random threshold chloride concentration for corrosion initiation (see Figure 3.16). The MCS results show that there is no significant difference in p_{ini} between the first repair and subsequent repairs for S1-10yr, with all values being approximately 6.6%. This finding highlights the effectiveness of the circular replacement method in restoring damaged structural components to their original condition. In terms of the feasibility of the repair plan, this approach is considered safe because the overall probability of corrosion initiation is less than 10%, which is an acceptable threshold in this context.

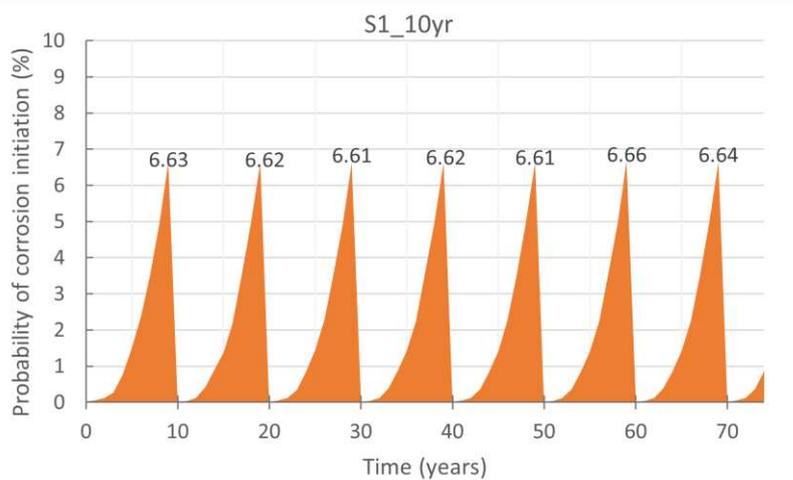


Figure 3.16: Probability of corrosion initiation for S1-10yr repair strategy

The entire process is repeated for the S2-10yr case. Only ED = 200 simulations of each maintenance strategy are needed to build the metamodel. Figure 3.17 illustrates the probability of corrosion initiation at any time during the 75-year service life. While p_f in the tenth year (Y10) is 6.6%, the same as for S1-10yr, subsequent repairs show a decrease in the probability of corrosion initiation due to the use of higher quality repair materials. As expected, this reduces the risk of corrosion initiation, with the probability dropping from 6.6% to 3.6%. This approach is also considered safe, as all p_f values remain below the 10% threshold. To optimize the repair plan with a focus on operational efficiency and cost effectiveness, it is essential to consider other proactive repair alternatives that can be evaluated in a comparable manner.

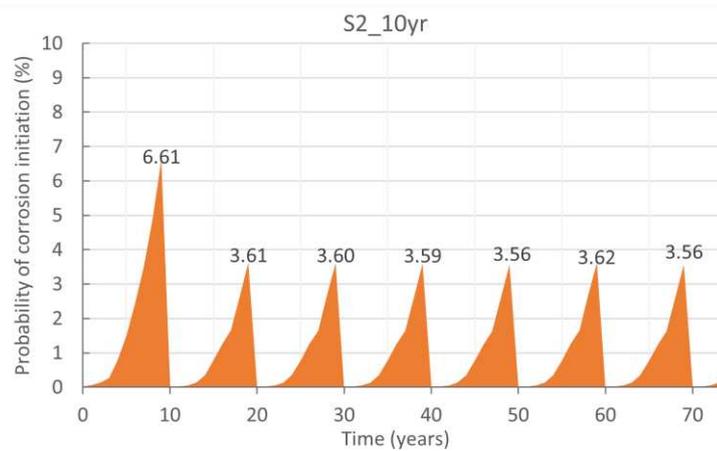


Figure 3.17: Probability of corrosion initiation for S2-10yr repair strategy

3.6.3.2. Optimizing maintenance schedule

The PCE-based metamodel could be applied to different maintenance strategies which consider different repair intervals. Two additional cases are then examined:

- S1-15yr, repair strategy of repeating 15-year repair intervals with a similar repair material.
- S2-15yr, repair strategy of repeating 15-year repair intervals with a higher quality repair material.

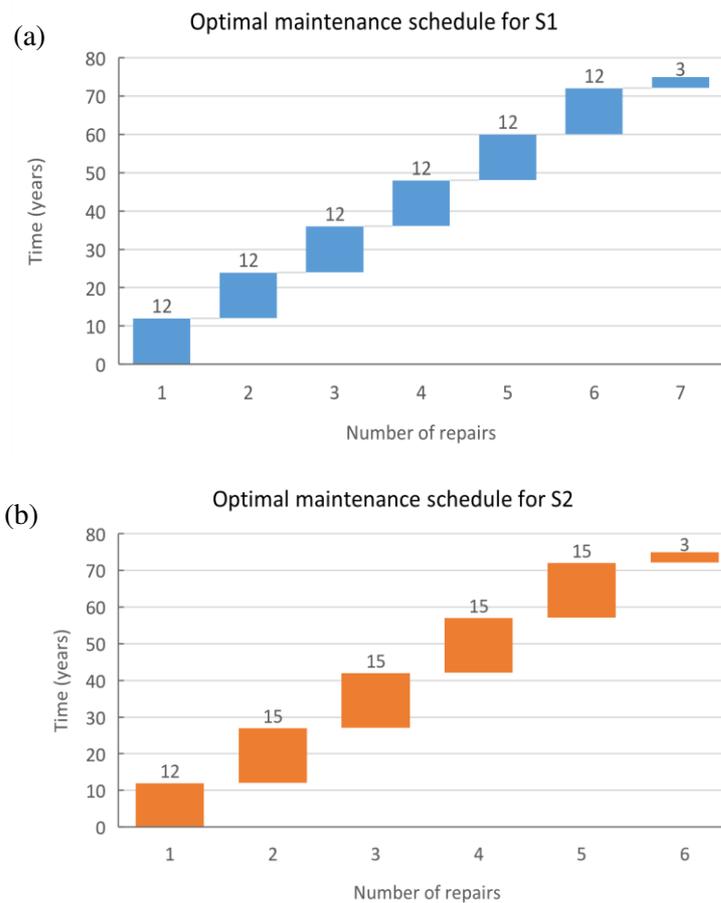
In these cases, the PCE-based probabilistic assessment requires 200 simulations for each maintenance strategy. However, instead of creating multiple PCE models for each year of the structure's life, the focus is on creating metamodels at specific repair times, e.g., Y15, 30, 45, 60, and 75. This approach ensures evaluation of the highest probability of corrosion occurring during critical periods, simultaneously optimizing computational time efficiency.

Table 3.8 presents the probabilities of corrosion initiation at the repair times for both S1-15yr and S2-15yr cases. In the case of S1-15yr, extending the repair interval results in a higher risk of corrosion, as evidenced by the probability of corrosion initiation increasing from 6.6% to 16.8% over the 5-year period, e.g., from Y10 to Y15. This suggests that the repair schedule needs to be revised to ensure the targeted structural safety requirement. For S2-15yr, the initial corrosion probability reaches a maximum of 16.5% at the first scheduled repair time, while subsequent repairs use new repair materials and have a reduced corrosion probability of about 10%. A minor adjustment of the repair schedule to reduce the first corrosion risk at Y15 would make this schedule more optimal in terms of durability and economic aspects.

Table 3.8: Probability of corrosion initiation p_f (%) at repair times for S1-15yr and S2-15yr

Repair times	Y15	Y30	Y45	Y60	Y75
S1-15yr	16.60	16.67	16.80	16.81	16.83
S2-15yr	16.60	10.26	10.25	10.29	10.22

In order to develop an optimal maintenance plan, the probability of corrosion initiation should be close to the allowable damage threshold of 10%, as mentioned earlier. Therefore, the repair schedule for both S1 and S2 is adjusted until this condition is met, following steps similar to those described previously. Finally, the optimal plans for S1 and S2 are determined and shown in Figure 3.18. For S1, regular repairs are recommended every 12 years, at years Y12, 24, 36, 48, 60, and 72. For S2, the first repair is performed at year 12, followed by subsequent 15-year repair intervals corresponding to years Y12, Y27, Y42, Y57, and Y72. It is noteworthy that the use of higher quality repair materials for S2 results in a smaller number of repairs (five repairs), over the same 75-year period compared to the six repairs required for S1.

**Figure 3.18:** Optimal maintenance schedule for (a) S1, (b) S2

3.6.3.3. Global sensitivity analysis of time-dependent chloride concentrations in repaired concrete

In this section, a PCE-based global sensitivity analysis is performed to identify the key random variables and their interaction effects in estimating the time-dependent chloride concentration both before and after repair (Tosin, Côrtes and Cunha, 2020). The S1-10yr scenario is selected as the case study for this investigation, and the analysis is performed using PCE models developed in Section 3.6.3.1. This analysis will provide valuable insight into the underlying mechanisms and factors affecting chloride concentration in repaired concrete and help improve the accuracy of the maintenance models.

The relationship between chloride concentration output at a certain time and six random variables used in the PCE model: $D_{c,ref}$, $D_{c,rep1}$, U_c , m_c , C_{fc}^S and X_c are examined. Figure 3.19 shows bar graphs of the first-order and total Sobol indices for the PCE models generated from the chloride concentration at the corner reinforcement position in year 10 (before repair) and in year 20 (after repair).

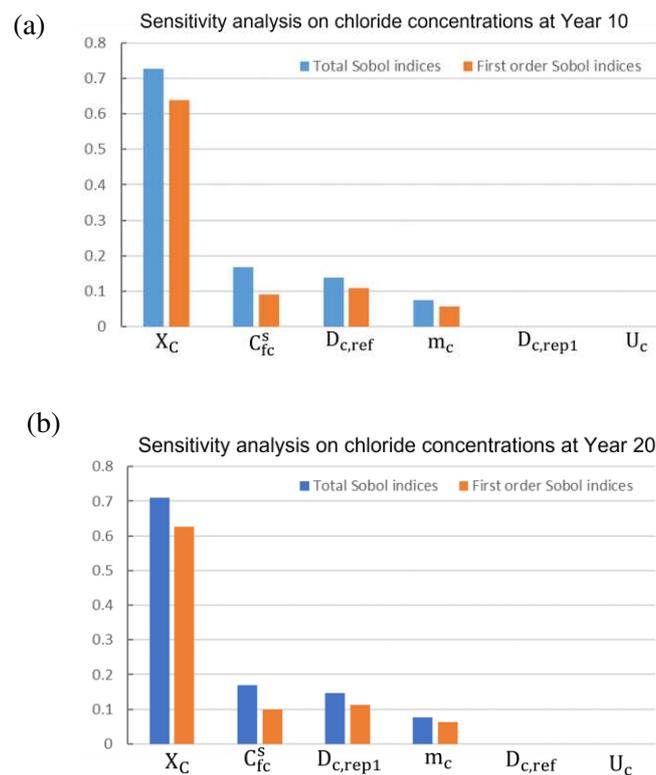


Figure 3.19: Influences of input parameters on chloride concentration at (a) year 10 (before repair) and year 20 (after repair)

In Figure 3.19a, the sensitivity analysis performed prior to the repair process shows that only four of the six random variables have significant sensitivity. These include concrete cover thickness X_c , surface chloride concentration C_{fc}^S , chloride diffusion coefficient $D_{c,ref}$, and aging factor m_c , presented in descending order of influence based on the total Sobol indices. On the

other hand, perturbation of the parameter U_c , which concerns the effect of temperature on the efficient chloride diffusion, has a negligible effect on chloride concentration. Furthermore, the Sobol indices of 0 for the parameter $D_{c,rep1}$ highlights the fact that the simulation model does not consider the diffusion coefficient of the repair material as a contributing factor in the accumulation of chloride within the concrete prior to the repair process. Figure 3.19a shows that the variances ($S_i^T - S_i$) for the highly influential parameters are less than 10%, which means that the correlation between the probabilistic parameters is not significant (Deman *et al.*, 2016).

The thickness of the concrete cover is in first place as the most important parameter with a S_i value of over 60%, whereby an increase in cover thickness leads to a decrease in chloride concentration. Therefore, it is important to control this parameter when designing new concrete structures or during construction/repair execution. This result is consistent with previous sensitivity analysis studies (Pradelle, Thiéry and Baroghel-Bouny, 2017; Senga Kiese *et al.*, 2020). However, note that these studies were mainly conducted with one-dimensional chloride models. In the present study with a two-dimensional chloride model, the high sensitivity of X_c is appropriate for the results when this parameter defines the position of corner reinforcement in a concrete column that is highly exposed to two-dimensional chloride attack.

Remarkably, the global sensitivity index and the first-order sensitivity index do not provide a consistent ranking of the two parameters C_{fc}^S and $D_{c,ref}$. However, considering the role of C_{fc}^S in the interaction between the input parameters, the surface chloride concentration is considered the second most important factor in this sensitivity analysis. This parameter is an environmental variable that is beyond our control due to the stochastic nature of chloride content in the marine environment.

Since chloride penetration into concrete structures is primarily controlled by diffusion, the chloride diffusion coefficient $D_{c,ref}$ and aging reduction factor m_c are expected to play important roles in probabilistic models for chloride penetration. This result has also been highlighted in previous studies, confirming that $D_{c,ref}$ and m_c (Van-Loc *et al.*, 2018; Bastidas-Arteaga *et al.*, 2020) are key parameters among concrete properties that can potentially be controlled by action levers.

In the sensitivity analysis at year 20 (ten years after first repair), shown in Figure 3.19b, the ranking of input parameters in terms of their influence on the model is similar to that in the sensitivity analysis at year 10, with X_c , C_{fc}^S , $D_{c,rep1}$ and m_c in decreasing order. However, it is worth noting some important considerations during this phase. After repair, the $D_{c,rep1}$ of the repair material replaces the $D_{c,ref}$ of the original concrete as the key parameter for characterizing the concrete properties. X_c remains the most sensitive factor, as it not only

determines the critical point for chloride evaluation, but is also responsible for variations in the replacement of repair concrete (as seen in Figure 3.14). In any case, X_c is a technological parameter that must be carefully controlled during the construction/repair process to minimize negative effects on the durability of structures subjected to chloride-induced corrosion.

The PCE models for proactive maintenance schedules are also suitable for performing a detailed sensitivity analysis for time-dependent systems, an aspect that has not been extensively studied in previous research. Figure 3.20 illustrates the impact of input parameters on the system over time, from the beginning to year 20, based on its first order sensitivity index (S_i). The results show that during a 10-year repair cycle, the ranking of the importance of input parameters remains constant, with X_c , $D_{c,ref}$ or $D_{c,rep1}$, C_{fc}^s and m_c in decreasing order. The influence of X_c decreases with time but still accounts for a significant portion of over 60%, while C_{fc}^s shows an increasing trend in its influence due to continued chloride attack by the external environment. Material characteristic parameters such as $D_{c,ref}$, $D_{c,rep1}$ and m_c also show a slight upward trend over the years, with the ageing exponent describing the time dependence of the apparent diffusion coefficient, leading to an increased effect with longer exposure time. These results once again emphasise the importance of X_c , the crucial parameter for evaluating the effects of chloride-induced corrosion on concrete. Controlling both the thickness of the concrete cover and the amount of new concrete used for replacement is critical to prevent the onset of corrosion and ensure effective repair. These results provide valuable guidance for future research and can help stakeholders identify important input parameters. As a result, efforts to quantify uncertainties can be focused to improve the evaluation of repair and maintenance activities for corroded concrete structures.

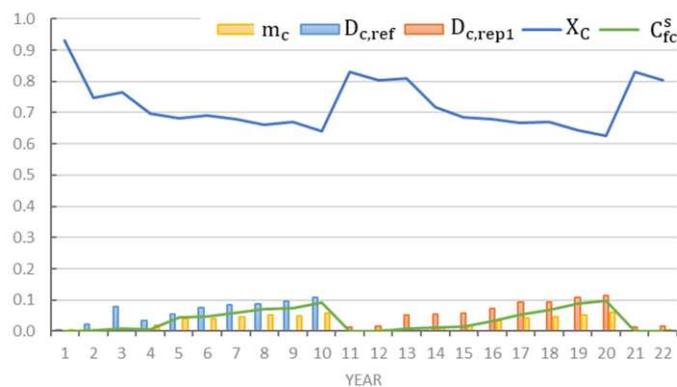


Figure 3.20: Sensitivity analysis for time-dependent systems based on first order Sobol indices

3.7. Summary and conclusions

This chapter performs probabilistic assessments and sensitivity analyses for maintenance modeling, estimate the probability of corrosion initiation and optimise maintenance plans.

Taking into account the complexity of the system, LHS is used first for the reliability analysis of 1D maintenance model. Then a surrogate-based probabilistic method is proposed to reduce computational time, which uses the PCE method to approximate the model of two-dimensional (2D) chloride ingress in repaired concrete. The summary and conclusions of this chapter are presented as follows:

1. Reliability analysis of corroded RC structures involves the consideration of two important limit states associated with serviceability and ultimate limit states, respectively. The selection of the appropriate limit state function depends on the objective of the analysis, which may be reliability assessment or maintenance/repair management. In this study, the focus is on the probability of corrosion initiation associated with the serviceability limit state, since the maintenance strategies used here are preventive rather than corrective measures.
2. The first example performed a comprehensive probabilistic lifetime assessment of repaired RC structures using the LHS technique. Both corrective and preventive repair strategies are performed in the probabilistic assessment. For the corrective repair strategy, repair is carried out when the chloride content at the rebar cover reaches a specific critical chloride value, and the total expected number of repairs can be used as a factor for comparing different repair strategies. It is observed that excepting the first longer repair time, the variation in chloride penetration behaviors in RC structures was not significant between repair periods. In the case of time constraint and expensive computational cost, the simulation of service life prediction for a few initial repair stages is sufficient as a basic result for formulating the maintenance schedule by the preventive approach. For the preventive repair model, this maintenance approach seems to be efficient for service life prediction and maintenance planning because the acceptable probability of corrosion initiation can be achieved by adopting some improved strategies. It is shown that the application of shorter repair intervals could contribute to the decrease of corrosion risk for the first repair, while the selection of the repair depth could mainly influence the probability of corrosion initiation after the first repair.
3. The following examples applied the PCE method to propagate uncertainties in maintenance modeling and optimize maintenance schedules. The PCE-based probabilistic assessment of proactive maintenance models is shown to be effective and accurate with low-cost evaluation patterns. Thanks to it, several proactive repairs are evaluated to obtain an optimal maintenance plan that meets the requirement of acceptable corrosion initiation probability.

4. As for the sensitivity analysis results, the influencing random variables are X_c (representing cover thickness and the amount of concrete replaced), surface chloride concentration C_{fc}^s , chloride diffusion coefficient $D_{c,ref}$, and aging factor m_c , where X_c is the most important parameter that should be controlled in the construction/repair process.
5. In addition, the results show that the use of higher quality repair material helps to reduce the risk of corrosion during the repair cycles, resulting in a longer time for the initiation of corrosion and thus a lower number of repairs.
6. Finally, real data (related to concrete chloride diffusivity and assessing criteria) is required to estimate correlations between the parameters used in the model and to improve the performance of maintenance model. Accordingly, the model-based approach for maintenance schedule could be compared with a practical engineering case and applied in practice. It is expected that this discovery will provide a basis for future theoretical and experimental efforts.

Chapter 4 *Selecting a Cost-effective and Sustainable Repair strategy for Chloride-contaminated Concrete Structures*

4.1. Introduction

Deterioration of concrete structures due to corrosion is a major problem in the modern infrastructure industry, which can lead to structural problems such as cracking, spalling and reduced load-bearing capacity. When corrosion occurs, repair of damaged structures is essential to maintain their safety and longevity. However, selecting the appropriate repair strategy can be a difficult task that requires consideration of several constraints. Specifically, cost is an important factor in maintenance management because repairs can be expensive and have a significant impact on the project budget. In addition, the repair strategy selected can have a significant impact on the sustainability of the structure. Factors such as environmental impact, durability, and life-cycle costs must be considered to ensure that the repair strategy is a durable and sustainable solution.

Integrating cost and sustainability considerations into the selection of repair strategies for corroded concrete structures requires a multidisciplinary approach that considers several factors. These factors include the type and extent of corrosion, location, accessibility, availability of materials and labor, and environmental impact. When these factors are considered in the selection process, engineers can find repair strategies that are both cost-effective and sustainable.

This chapter aims to provide insight about how environmental considerations can be integrated into the selection of repair methods, ensuring a balance between cost-effectiveness, durability, and sustainability. The objectives of this chapter are:

1. formulate the procedure for the repair and sustainable maintenance management of chloride-induced corroded RC structures;
2. formulate life-cycle cost analysis, which focuses on maintenance costs throughout the design life of a structure.

3. propose a method to consider the influence of cost and environmental impact in selecting repair methods that meet the required durability standards;
4. illustrate the proposed approach by applying it to some case studies.

The conceptual formulation for repair and sustainable maintenance management is described in Section 4.2. Section 4.3 presents the formulation for life-cycle cost analysis for repaired RC structures. Section 4.4 addresses the criteria considered in the sustainability analysis and introduces the multi-objective optimization method. Finally, two case studies are presented to illustrate the selection of the most cost-effective and sustainable repair strategies.

4.2. Repair and sustainable maintenance management for corroded structures

Classically, cost is the most important factor when selecting repair strategies. Analysis aimed at finding the most cost-effective solution applies a *cost-effective management strategy* to the selection of repairs. However, the consideration of cost and environmental issues in this process requires a systematic approach that applies a *sustainable management strategy* to determine the most sustainable repair solution. The most important factor that differentiates a "sustainable" solution from a "cost-effective" solution is the consideration of environmental constraints. It should be noted that the conceptual formulation for sustainable maintenance management in this chapter is not intended to improve the cost-effectiveness/sustainability of a single maintenance technique, but to compare different maintenance strategies. Therefore, in order to obtain a cost-effective/sustainable maintenance alternative, several aspects and stages should be considered in the formulation process:

1. The crux of the management process lies in *the formulation of the technique*, which is an essential step. Various aspects must be considered in the formulation, such as the process of degradation with/without repair, the methods of inspection, the maintenance philosophy - whether preventive or corrective, and the technical and financial feasibility of the approach. It is crucial that all stakeholders involved in the life cycle of the structure participate in this step in order to arrive at a mutually acceptable methodology. This study applies a repair technique that reconstructs a chloride-contaminated concrete cover. The approach adopted considers preventive maintenance, in which repairs are made either at the time of or before completion of the initiation phase, when the chloride concentration near the reinforcement reaches a critical level.
2. The use of a deterioration model that accurately reflects the studied repair practice. In this study, the objective is to *model the deterioration process and simulate the performance of maintenance actions*. Evaluating the effectiveness of maintenance actions is critical, and experimental tests are often considered the gold standard. However, such

tests can be prohibitively expensive and time consuming. In some cases, numerical modeling of degradation and repair processes can be a useful alternative to evaluate and improve the effectiveness of maintenance actions. To ensure the accuracy of such models, it is necessary to account for uncertainties associated with both the degradation process and maintenance procedures. It should be clear, however, that accurate predictions require empirical evidence combined with input from contractors and organizations. On this basis, several maintenance strategies and schedules could be developed that are both technically and economically viable and ensure an optimal level of safety and serviceability throughout the life of the structure.

3. Once the repair options are identified, the next step is to *evaluate the costs associated with each potential repair option*. This includes the cost of materials, labor, and necessary equipment. It is also important to consider any operational interruptions that may occur during the repair process, as these can affect the overall cost.
4. *Cost-effective maintenance management*: it requires estimating life-cycle costs associated with different repair alternatives. It is important to consider not only the initial cost, but also the life cycle cost of maintenance. These are calculated based on established repair schedules that meet the long-term durability and maintenance requirements of each option. Lower initial costs may seem attractive, but may result in higher life-cycle costs due to frequent maintenance or shorter service life. Therefore, the long-term impact of each repair option should be carefully evaluated to ensure optimal performance and cost efficiency.
5. *Environmental impact assessment*: one issue to consider is establishing criteria for evaluating environmental impacts. These could include emissions generated during the repair process, consumption of non-renewable resources, and disposal of waste materials. An environmentally friendly repair option may have higher initial costs, but may provide cost savings in the long run through reduced maintenance.
6. *Sustainable maintenance management*: it requires a method to support decision making taking into account multi-objective targets, i.e. minimizing costs and environmental impact. On this basis, the most sustainable maintenance solution can be selected.

4.3. Cost-effective maintenance management

The main objective of cost-effective maintenance management is to identify the most efficient and economical repair options. This includes various repair techniques, such as replacing polluted concrete with different shapes/thicknesses and repair materials. To achieve this goal, a comparison of life-cycle costs for each repair option is required. Figure 4.1 shows the proposed framework for comparing the costs of the various repair alternatives in this study. Due to different characteristics of repair strategies, each option has different repair schedules

that meet the long-term durability and maintenance requirements. On that basis, life-cycle costs of different repair alternatives are then calculated and compared to select a cost-effective maintenance solution.

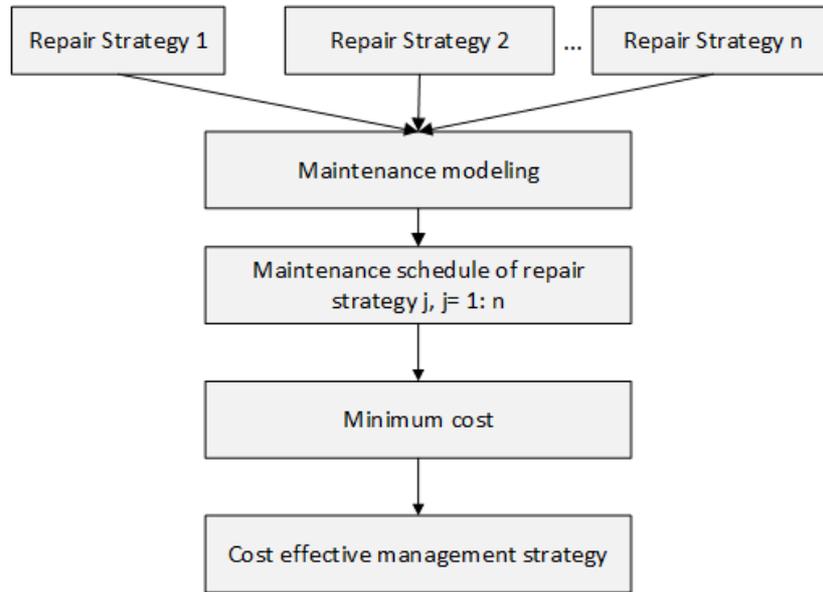


Figure 4.1: Framework for cost comparison of different repair strategies

4.3.1. Life-cycle cost estimation methodology

According to Boschmann Käthler and Angst (2022), the concept of life-cycle cost refers to the total cost associated with the new construction and, if necessary, the interventions required to achieve the designated lifespan. This can be represented mathematically by the following formula:

$$C_{LCA} = C_N + \left(\sum_{i=1}^n C_{RP,i} \times r_1^{-t} \times r_2^t \right) + C_R \quad (4.1)$$

where C_N is the initial construction cost, $C_{RP,i}$ is the cost of a single inspection or repair activity applied for the structure at time t , n is the number of repairs during the structure's lifespan. r_1 is the interest rate capitalization and r_2 is the inflation rate, C_R is the residual cost (i.e., demolition cost) at the end of service life. However, in this cost assessment, the initial construction and residual costs are expected to not significantly affect the comparison of the different alternatives because they are spent on the same identical RC components. For this reason, C_N and C_R may not need to be included in the above formula, and the focus of the cost analysis can be placed on the total cost of repairs aimed at ensuring structural durability throughout the life of the asset.

It should be noted that the repair and maintenance cost estimate presented in Eq. (4.1) is a deterministic evaluation based on some specific parameters such as C_{RP} , r_1 , r_2 , the repair time t , and the number of repairs n , where t and n of each repair strategy are taken from the potential

maintenance schedule. The maintenance plan could be determined by a deterministic analysis, as shown in Case Study 2, Section 2.3, Chapter 2, or by a probabilistic approach ensuring that a repair plan satisfies the certainty probability of corrosion initiation, as presented in Case Study 3, Section 3.6, Chapter 3. Consequently, it provides stakeholders with a practical and easy-to-use method for calculating life-cycle costs and supporting decision-making while considering the uncertainty associated with maintenance activities.

4.3.2. Life-cycle maintenance cost estimation

According to (Bastidas-Arteaga and Stewart, 2014), the cost of replacing concrete cover depth, consisting of inspection, demolition, repair and operating loss, is $C_{\text{cover}} = 323 \text{ €/m}^2$ (repair costs estimated for the port of Nantes-Saint-Nazaire in France in 2012). Excluding replacement of the corroded bars, the repair includes removal and reconstruction of chloride-polluted concrete cover between 4 cm and 8 cm for slabs and beams. The additional repair cost per mm of extra repair depth behind the cover is given in Table 4.1 (Bastidas-Arteaga and Stewart, 2014). As shown in Table 4.1, the repair cost is higher for a square column because it accounts for the damage occurring on all four sides of the column.

Table 4.1: Extra repair costs for various structural components

Structural element	Dimensions (mm)	C_{extra} (€/mm)
Slabs	100	0.85
Slabs	300	0.49
Beams	200 to 800	0.98
Square columns	300	1.56
Square columns	400	1.38
Square columns	600	1.01

Consequently, the construction cost of one concrete replacement activity, $C_{\text{RP},i}$, is defined as:

$$C_{\text{RP},i} = [C_{\text{cover}} + \text{Depth over cover (mm)} * C_{\text{extra}}] S_{\text{RP}} \quad (4.2)$$

where S_{RP} (m^2) is the surface area of the structure to be repaired, C_{cover} (€) are the costs for the replacement of the entire concrete cover, Depth over cover (mm) is the extra repair depth behind the cover. It should be noted that the cost information used in this study comes from the construction of French coastal infrastructure in 2012. Thus, the expected results cannot be adequately compared to other repair situations because repair costs can vary greatly depending on facility conditions and geographic location. Nevertheless, it is possible within the scope of this study to perform a cost-effective evaluation for different repair strategies.

4.4. Sustainable maintenance management

Sustainable development is defined by the World Commission on Environment and Development (1987) as development that meets present needs without compromising the ability of future generations to meet their own needs. Environment, economy and society are the three pillars of sustainability according to Struble & Godfrey (2004) (as shown in Figure 4.2). Sustainable development must balance these elements to achieve its goal.

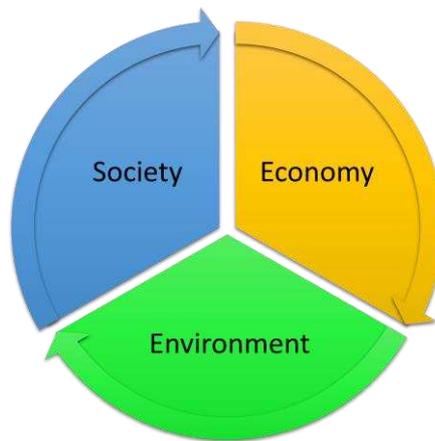


Figure 4.2: Three pillars of sustainability

The analysis of sustainability in this study primarily considers the environmental and economic aspects of sustainability. However, the decisions that affect these components directly affect society. Regarding the environmental aspect of sustainability, it is important to consider some specific criteria for environmental impact, especially in the construction industry:

- **Energy consumption:** The energy consumed during construction and maintenance could contribute to the environmental impact. This criterion is mainly used in the selection of design alternatives for high-rise buildings or in the installation of equipment with low energy consumption. In maintenance management of corroded RC structures in marine environment, it is difficult to make qualitative comparisons of energy consumption between different repair strategies.
- **Greenhouse gas emissions:** CO₂ emissions are identified as a major cause of global warming and climate change. The production of construction materials such as cement, steel, and concrete, as well as the emissions that result from the transportation of materials, equipment, and waste, cause significant greenhouse gas emissions. In maintenance management, these environmental impacts could be reduced by using recycled or low-carbon repair materials or by reducing maintenance frequency.

- Waste generation: construction projects generate significant amounts of waste, including construction and demolition debris, packaging materials, and excess building materials. This waste can end up in landfills, contributing to environmental problems such as soil contamination and greenhouse gas emissions. Construction professionals can reduce waste generation by using recycled materials, implementing waste reduction strategies, and recycling materials at the end of their useful life. In the case of concrete cover repair, the waste could come from the removal of the damaged concrete.
- Pollution: construction projects can cause environmental pollution in the form of dust, noise, air and water pollution. Dust and noise pollution can have negative impacts on the health and well-being of construction workers and nearby residents. Air and water pollution can have negative impacts on the environment and public health. Construction professionals can reduce pollution by implementing pollution prevention strategies, such as using low-emitting equipment and environmentally friendly cleaning products. This is a criterion that is also difficult to quantify, so it is not considered in the sustainable maintenance management process in this study.

Therefore, with sustainability in mind, strict standards should apply to the repair and maintenance of concrete structures, both in terms of environmental impact and financial constraints (Bastidas-Arteaga and Schoefs, 2015). Consequently, a comparison of three factors could be used to assess the long-term feasibility of repair methods:

1. cost,
2. waste generation, and
3. CO₂ emissions.

The proposed approach for comparison between different repair alternatives, which includes cost and environmental criteria, is shown in Figure 4.3. Due to different repair strategies in the shape and thickness of the replacement concrete or by using different repair materials, each strategy has a different profile of long-term durability and service life extension. These three criteria are then calculated for each maintenance plan. Finally, a multi-criteria optimization approach is applied to compare and select a sustainable maintenance solution.

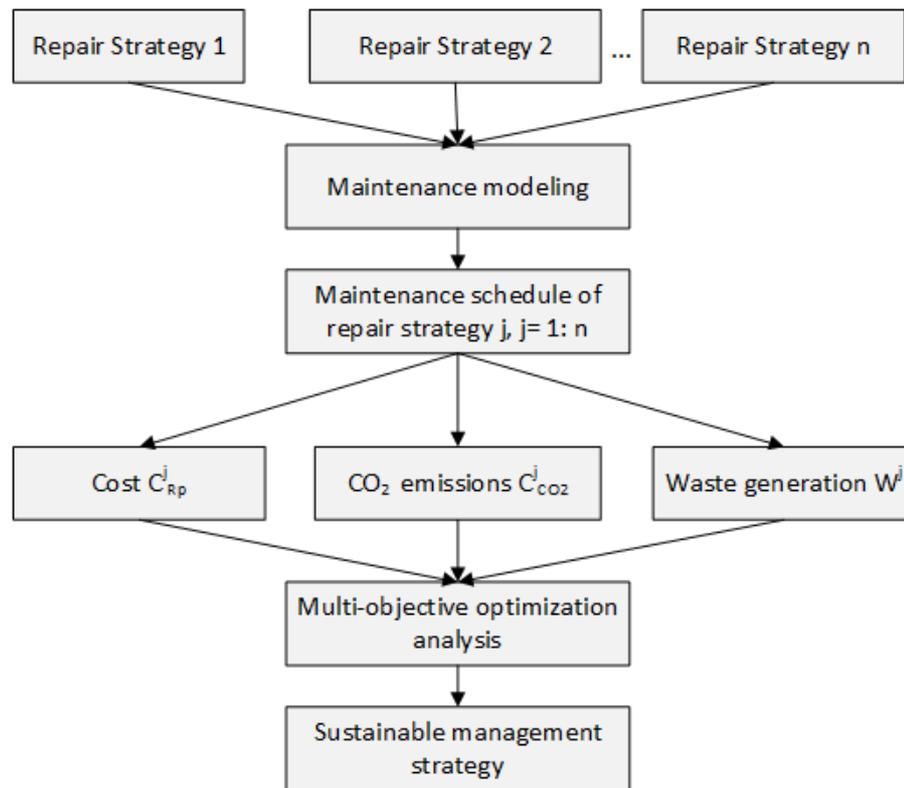


Figure 4.3: Framework of sustainable maintenance management

4.4.1. Costs

This study primarily relies on publicly available data on direct costs for comparing costs of the various repair strategies for deteriorating RC structures. Direct agency costs are limited to those associated with maintenance (inspection and repair). The analysis does not consider initial construction and residual costs because the study assumes repairs are made to identical RC components. The detail of cost assessment has already been mentioned in Section 4.3.

4.4.2. CO₂ emissions

The environmental impacts of concrete repair are classified into several categories (Årskog, Fossdal and Gjørsv, 2013). In this study, the category of global warming impact is examined in relation to CO₂ emissions from the replacement of the chloride-polluted concrete. In this repair technique, the chloride-polluted concrete is removed and the structure is rehabilitated by installing new concrete or repair mortar without damaging the reinforcing bars. The new material is usually placed as wet shotcrete, dry shotcrete, formed concrete or by manual repair (BSEN 1504-10:2017). Therefore, the six processes that affect the amount of CO₂ released are considered, including manufacturing of repair materials, transportation, water blasting to remove concrete cover, cleaning of reinforcement, protective coating of reinforcement, and placement of shotcrete (Petcherdchoo, 2015; Lifecon Deliverable D 5.3). The estimated CO₂ emissions per cover repair measure are shown in Table 4.2.

It should be noted that the functional CO₂ units are not the same for all processes (as shown in Table 4.2). Therefore, the life cycle CO₂ emission (kg CO₂) for each repair strategy can be calculated as follows:

$$C_{CO_2} = (405 V_{RP} + 121.8 S_{RP})n_{RP} \quad (4.3)$$

where V_{RP} is the volume of concrete repaired (m³), S_{RP} is the surface area of the structure to be repaired (m²), and n_{RP} is the number of repairs.

Table 4.2: CO₂ emissions per cover repair measure

Processes	CO ₂ (kg CO ₂ eq/m ³)	CO ₂ (kg CO ₂ eq/m ²)
Concrete Production	405	-
Hydro-jetting	-	84
Cleaning of reinforcement	-	22
Protective coating on reinforcement	-	1.4
Shotcrete application	-	4.4
Transportation	-	10
Total	405	121.8

4.4.3. Waste generation

Concrete demolition waste accounts for 67% of global construction waste by weight (53% by volume) and only 5% is typically recycled. Waste generation (concrete waste) should be considered a criterion in sustainable assessment of repair and maintenance activities (Bastidas-Arteaga and Schoefs, 2015).

A comprehensive assessment would consider waste generated during actual repair and maintenance activities but also waste generated during the process of production of repair materials. However, because it is difficult to measure the waste generated during concrete production, this study only focuses on waste generated during repair works. The waste generated by cover replacement repair of 1 m³ of concrete has been determined to be 1.3 m³ of waste (Bastidas-Arteaga, 2010). Therefore, the waste generation, $W(m^3)$, during the whole life cycle for each repair strategy, can be calculated from the repair volumes calculated in the repair analysis in this study as follows:

$$W = 1.3 V_{RP} n_{RP} \quad (4.4)$$

where V_{RP} is the volume of concrete removed per cover replacement application (m³), n_{RP} is the number of repairs of each repair strategy.

4.4.4. *Methods for sustainable maintenance management*

The complexity of multicenter management of deteriorating structures presents challenges for operating agencies of concrete facilities. Agencies have conflicting objectives to reduce costs, environmental impacts, and traffic disruptions while improving serviceability, functionality, and safety. A multi-criteria or multi-objective optimization approach can help to solve these conflicts. There are several methods for multicriteria optimization.

- The weighted sum approach: in this method, each objective is assigned a weight and they are combined into a single objective function that is minimized or maximized to find an optimal solution.
- The Pareto optimization method: it finds a set of non-dominated solutions that cannot be improved without affecting the performance of another objective.
- Goal programming: in this method, goals or constraints are defined for each objective, and the optimization problem is to find a solution that satisfies all of them.
- Compromise programming finds a solution that is a compromise between conflicting objectives.
- Constraint programming defines constraints that the solution must satisfy. The choice of method depends on the problem and the goals of the optimization.

The choice of method depends on the specific problem and the goals of the optimization. Researchers and practitioners should carefully consider the trade-offs between the objectives, the complexity of the problem, and the computational resources available when selecting an appropriate method. In the context of finding an appropriate method for a finance/engineering problem, the advantage of the compromise programming is that it allows the decision maker to explicitly control the tradeoffs between objectives, which can be useful in situations where the relative importance of objectives varies depending on the context. Therefore, this study considers the compromise programming approach, namely VIKOR (Opricovic, 1998).

The VIKOR method is a multi-criteria decision-making technique used to evaluate alternatives based on multiple criteria. This method is widely used in various fields, including engineering, management, economics, and environmental sciences. The VIKOR approach allows the selection of a compromise solution with competing evaluation criteria. Ranking is based on similarity to an ideal alternative, where a compromise is defined as an agreement reached by mutual concessions, evaluating each alternative according to each criterion function (Opricovic, 2011; San Cristóbal Mateo, 2012). The L_p -metric, used as an aggregation function in a compromise programming approach, is the basis for the multi-criteria compromise ranking:

$$L_{p,j} = \left\{ \sum_{i=1}^n \left[\frac{w_i (f_i^* - f_{ij})}{(f_i^* - f_i^-)} \right]^p \right\}^{\frac{1}{p}}, \quad 1 < p < \infty \quad (4.5)$$

where f_{ij} is the value of the i^{th} criterion function for the alternative j ; and f_i^* and f_i^- are the best and worst values of alternative j , respectively. n is the number of criteria, in this strategy there are three evaluation criteria: (1) costs, (2) CO₂ emission and (3) waste generation. w_i is the weight factor of the i^{th} criterion function, the value of w_i varies between 0 and 1 with $\sum w_i = 1$. The weight factors are subject to the operator's attitude towards each criterion.

$L_{1,j}$ (the value of S_j) and $L_{\infty,j}$ (the value of R_j) are used to formulate a ranking measure. S_j represents the maximum utility of alternative groups, while R_j stands for the least effective individual alternative. In addition, comprehensive indicator Q_j is used to rank the alternatives, where Q_j is the average weighted, normalized values S_j and R_j in the interval [0;1]. The VIKOR method is used to calculate the positive and negative points of the solution and finally the alternatives are ranked by sorting the values R , S , and Q in descending order. The alternative with the lowest value of Q is the best alternative. The steps to implement the VIKOR method and the conditions to achieve a compromise solution are well described in previous studies (San Cristóbal Mateo, 2012; Mardani *et al.*, 2016; Mateusz *et al.*, 2018) and will be presented in the Appendix B.

4.5. Case study 1

4.5.1. Problem description and basic considerations

Using the same problem and results presented in the Case Study 2, Section 2.3, Chapter 2, the purpose of this example is to show how the proposed methodology can be used to estimate the maintenance cost and environmental impact of a repair method and then choose the best compromise repair option for a chloride-polluted structure. It is assumed that this structure (i.e., a harbor) consists of 86 RC column piles with a cross-section of 0.4 m x 0.4 m, placed in an aggressive marine environment and in need of repair during their 75-year lifespan. The total surface area of the structure to be repaired is about 400 m² for each maintenance procedure. Accordingly, the cost and environmental impact depend on the volume of concrete to be repaired and the number of repairs. Figure 4.4 shows again the three maintenance strategies where only repairs are performed throughout the 75-year life of the structure:

1. OCR1: Orthogonal concrete replacement, repair thickness X_p is equal to the existing cover X_c (Figure 4.4a)
2. OCR2: Orthogonal concrete replacement, repair thickness X_p is thicker than the existing cover X_c (Figure 4.4b), and

3. CCR3: Circular concrete replacement, where the boundary line between two materials is a circular curve (Figure 4.4c)

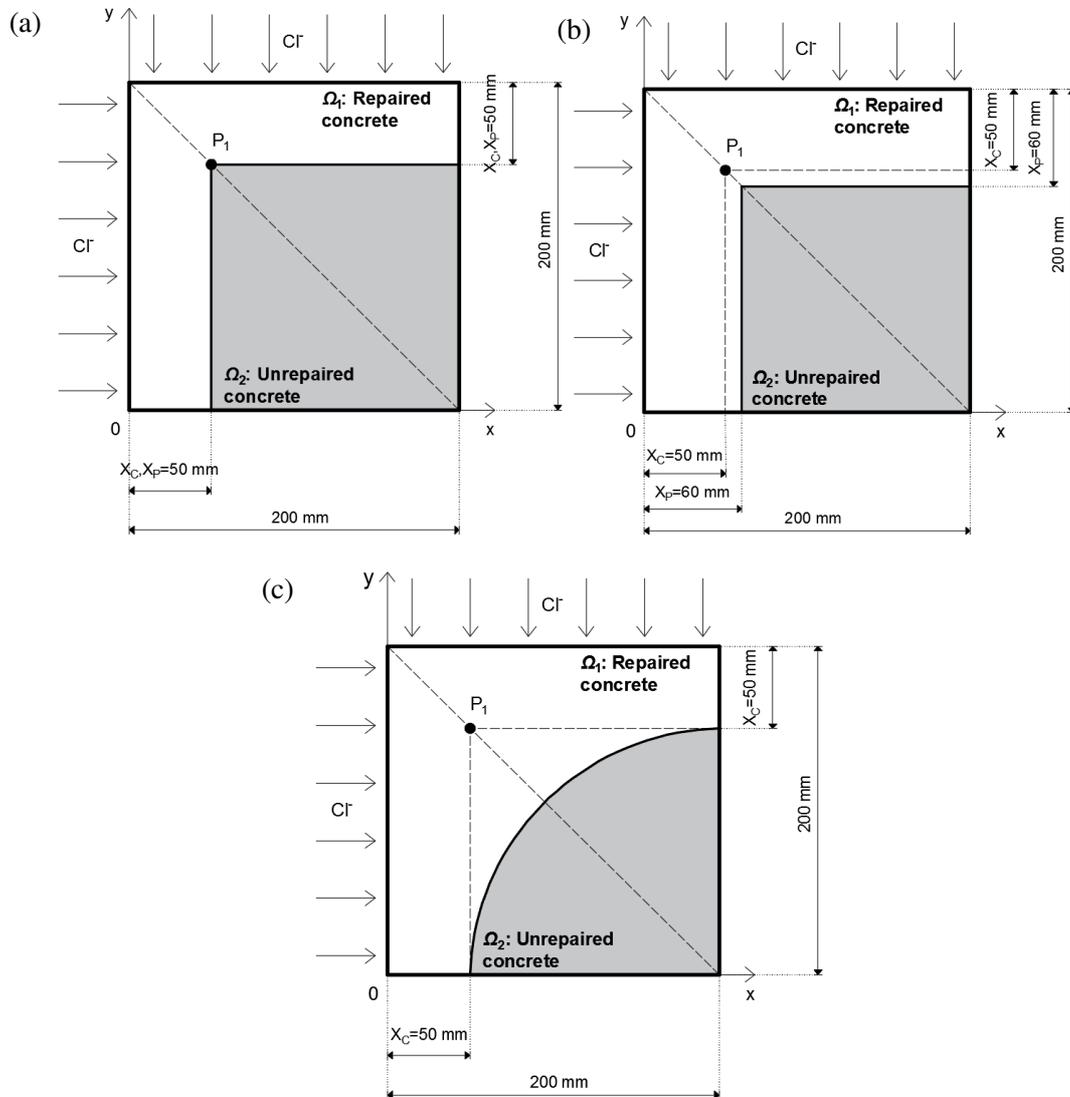


Figure 4.4: Three repair strategies (a) OCR1 with $X_p = X_c$, (b) OCR2 with $X_p > X_c$ (c) CCR3

4.5.2. Maintenance schedule of each repair strategy

As previously analyzed in Case Study 2, Section 2.3, Chapter 2, the outer column corners (point P_1) are the most crucial locations used for predicting the repair time of structures. Table 2.3 also indicated the times for repair based on a reactive approach, where the initiation of corrosion is considered as a repair criterion. In practice, maintenance planning often follows a periodic schedule with predetermined repair times, resulting in preventative rather than reactive repairs. Therefore, this section proposes a periodic, preventive repair schedule, where repairs for each strategy are scheduled at annual intervals. The intervals between repairs are listed in Table 4.3

and each satisfies the condition of reducing the corrosion initiation risk to the structure based on the deterministic approach.

Table 4.3: Number of repairs for the three repair strategies

Strategy	Repair time (year)							Average number of repairs over 75 years*
	1 st	2 nd	3 rd	4 th	5 th	6 th	7 th	
OCR1	13	24	35	46	57	68	>75	6.6
OCR2	13	25	37	49	61	73	>75	6.2
CCR3	13	26	39	52	65	>75		5.8

* Note: The decimal values are estimated by dividing the remaining life of the structure after the first repair by the respective repair interval specified for each strategy. The result of the above calculation plus the first repair is the expected number of repairs during the life of the structure. This approach allows us to avoid neglecting and underestimating the long-term effectiveness of different repair options and can be applied to structures with different expected lifetime assumptions.

In Table 4.3, the first preventive repair for all three strategies is scheduled 13 years after the structure is put into service. Subsequent to the initial repair, periodic repairs for OCR1, OCR2, and CCR3 are scheduled at repeating 11, 12, and 13-year intervals, respectively, throughout the remainder of the structure's service life. Repair strategies OCR2 and CCR3, which utilize thicker concrete replacement, extend the time until the onset of corrosion and thus result in fewer repairs over the course of a structure's service life compared to strategy OCR1. Table 4.5 shows the number of repairs required during a 75-year service life for the three strategies OCR1, OCR2, and CCR3. It is observed that CCR3 requires less repairs than OCR2.

4.5.3. Results and discussion

To facilitate the calculation, the cost, CO₂ emissions, and waste generation are calculated per 1 m² of repaired concrete surface area, assuming damage on all four sides of the column. The parameters used to estimate the maintenance cost and environmental impact of each repair strategy during the analysis period are shown in Table 4.4.

Table 4.4: Parameters for cost and environment impact assessment

Parameter	Value
Cross-sectional dimensions of concrete column	0.4 m x 0.4 m
Surface area of the structure to be repaired, S_{RP}	1 m ²
Volume of concrete repaired per one repair OCR1, V_{RP}^{OCR1}	0.04375 m ³
Volume of concrete repaired per one repair OCR2, V_{RP}^{OCR2}	0.051 m ³
Volume of concrete repaired per one repair CCR3, V_{RP}^{CCR3}	0.056 m ³
The number of repairs of OCR1	6.6
The number of repairs of OCR2	6.2
The number of repairs of CCR3	5.8
Maintenance Cost per one repair activity OCR1 (Repair thickness equal to the existing concrete cover)	323 €/m ²
Maintenance Cost per one repair activity OCR2 (Repair thickness 10 mm greater than existing concrete cover)	336 €/m ²
Maintenance Cost per one repair activity CCR3 (Repair thickness is converted to be 62 mm, 12 mm greater than existing concrete cover)	339 €/m ²

Based on Eqs. (4.1)-(4.4) and Table 4.4, the calculated cost, CO₂ emissions, and waste generation for three repair strategies are placed in the decision matrix, as shown in Table 4.5. It is noted that the r_1 and r_2 are not used in this example.

Table 4.5: Decision Matrix

Alternatives	Cost (€/m ²)	CO ₂ (kg CO ₂)/m ³	Waste m ³
OCR1	852720	368320	150
OCR2	833280	353280	164.4
CCR3	786480	335200	168.8

As shown in Table 4.5, repair method OCR1, with a lower repair thickness but a larger number of repairs, has the highest total maintenance cost of €852,720 and the highest CO₂ emissions for a 75-yr service life. The repair method CCR3, with the most damaged concrete removed but the least number of repairs, is the cheapest solution with a final cumulative cost of €786,480. In terms of environmental impact, CCR3 has the lowest CO₂ emissions but the highest waste generation due to a larger amount of concrete removed.

The VIKOR technique presented in Section 4.4.4 is used here to solve the multi-criteria decision-making problem and find the best compromised solution. Table 4.6 shows the ranking

of the alternatives based on the S, R, and Q values, where S_j and R_j denote the utility measure and regret measure for alternative j, and Q_j is the comprehensive indicator derived from the average weighted, normalized values S_j and R_j . The best rank is assigned to the alternative with the smallest VIKOR value (Q value). These results were obtained by assuming that the agency assigns equal importance to costs and environmental impacts – i.e., $w_{cost} = 0.5$, $w_{CO2} = 0.25$ and $w_{Waste} = 0.25$ in Eq. (4.5).

Table 4.6: The ranking list for three repair strategies

Strategies	R value	Rank in R	S value	Rank in S	Q value	Rank in Q
OCR1	0.5	3	0.75	3	1	3
OCR2	0.353	2	0.681	2	0.638	2
CCR3	0.25	1	0.25	1	0	1

As a result, the repair strategy CCR3 is the selected alternative. The strategies OCR1 and OCR2, that follow the method of orthogonal concrete replacement, are not chosen because of their higher maintenance cost and larger environmental impact. Therefore, these repair methods are not recommended to repair chloride-polluted RC columns due to 2D chloride penetration. In addition, it is worth noting that when cost is weighted higher than 0.5, and total environmental factor therefore weighted less than 0.5, the VIKOR analysis leads to the same result, namely that CCR3 is the best option from the perspective of cost and environmental impact. Given these repair characteristics, the total number of repairs governs alternative selection.

4.6. Case study 2

In the Case Study 3, Section 3.6, Chapter 3, two different maintenance strategies were proposed, referred to as S1 and S2, using the method of circular concrete replacement with different repair materials (as shown in Figure 4.5)

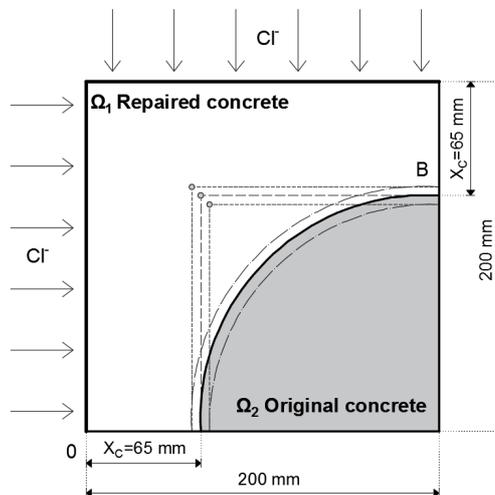


Figure 4.5: Circular concrete replacement method with S1 and S2 repair options

The optimal repair schedule for each strategy was determined based on the repair criterion of allowable threshold damage using a probabilistic approach presented in Chapter 3. The results presented in Table 4.7 show that strategy S1 requires earlier and more frequent repairs (six in total) during the 75-year life of the structure, while S2 requires only five repairs. This indicates that the use of higher quality repair material in S2 is a more beneficial maintenance strategy. However, using the new, higher-quality material (S2) for a single repair incurs a higher cost than using material S1. A comparison of lifetime maintenance costs is required to determine the most cost-effective repair strategy.

Table 4.7: Number of repairs for repair strategies S1 and S2

Strategy	Repair time (year)							Expected number of repairs over 75 years
	1 st	2 nd	3 rd	4 th	5 th	6 th	7 th	
S1	12	24	36	48	60	72	>75	6
S2	12	27	42	57	72	>75		5

It is assumed that both repair strategies (S1 and S2) are applied to the same structure in the harbor, which consists of 100 concrete columns with dimensions of 0.4 m x 0.4 m. The structure, which is exposed to chloride attack in a marine environment, must be maintained over a 75-year service life. Each maintenance operation involves repairing about 500 square meters of the structure's surface concrete.

Based on the repair cost data presented in Section 4.3.2, the cost of a single S1 repair is estimated to be approximately 344 €/m² (the repair thickness is calculated to be 15 mm greater than the existing concrete cover). For the repair material S2, the cost is assumed to be higher by a factor of 1.2, which means that a single repair S2 costs about 412 €/m². The values of the capitalization rate r_1 and the inflation rate r_2 are 1.04 and 1.03, respectively, are used in this example.

Accordingly, Table 4.8 shows the repair costs incurred at the scheduled time according to the maintenance schedule for each of strategies S1 and S2, calculated based on the cost estimation method and Eq. (4.1) in Section 4.3.

Table 4.8: Costs incurred for S1 and S2 repair strategies during a 75-year lifespan (€)

Repair Strategy	Number of repairs						Total cost
	1 st	2 nd	3 rd	4 th	5 th	6 th	
S1	153170	136402	121470	108172	96330	85784	701329
S2	183448	158698	137287	118765	102741		700941

Using the results in Table 4.8, the cumulative maintenance costs of the repair strategies are calculated and shown in Figure 4.6.

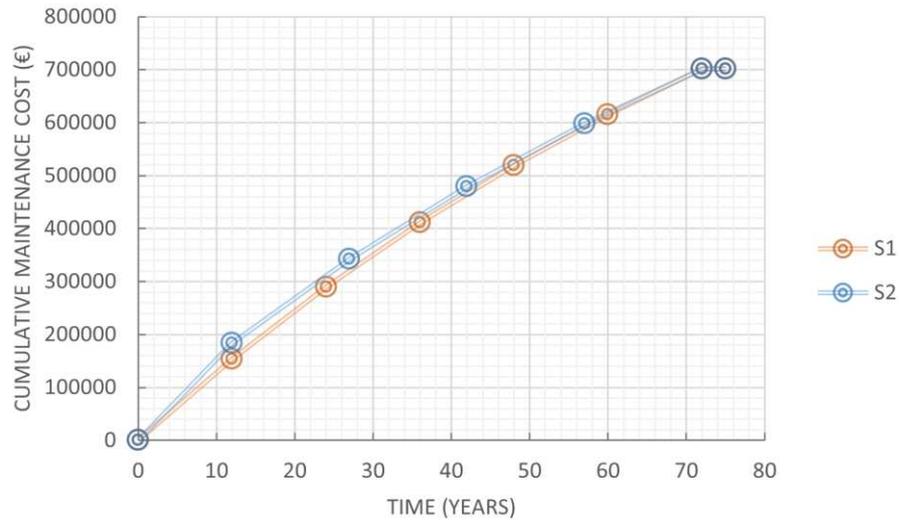


Figure 4.6: Cumulative maintenance cost of each repair strategy S1 and S2

It can be seen in Figure 4.6 that repair strategy S1, which incurs lower individual repair costs but requires a greater number of repairs, results in a little higher maintenance costs over the 75-year lifetime of the structure. Conversely, repair strategy S2, which involves higher individual repair costs due to the use of better repair materials but requires a smaller number of repairs, is considered to be the more cost-effective solution. In fact, the final cumulative costs for S2 are a little lower than those for S1, amounting to 700,941€ and 701,329€ respectively. Therefore, it can be concluded that S2 seems to be the more efficient option from an economic point of view.

Regarding the environmental impact of each repair strategy, the difference in CO₂ emissions and waste generation for each repair activity is not significant because the same amount of concrete is removed in S1 and S2. However, the difference in the repair frequency of each strategy could contribute to the evaluation of these criteria. Based on Eqs. (4.3)-(4.4), CO₂ emissions and waste generation for two repair strategies are calculated. Values of three criteria are placed in the decision matrix, as shown in Table 4.9:

Table 4.9: Decision Matrix

Alternatives	Cost (€/m ²)	CO ₂ (kg CO ₂)/m ³	Waste m ³
S1	701,329	443,451	250,536
S2	700,941	369,543	208,78

From Table 4.9, S2 strategy has the lowest CO₂ emissions and also the lowest waste generation. This is due to its efficient maintenance schedule, which requires less frequent repairs compared to S1. Therefore, S2 seems to be the optimal choice when considering both the cost and

environmental impact of concrete repair. These results suggest that, in addition to reducing repair times or increasing the amount of contaminated concrete replaced, the resistance of both the patch and the original concrete, which serves as an indicator of material quality, are important factors affecting the effectiveness of maintenance (Soleimani *et al.*, 2010). However, the consideration of the uncertainty inherent to waste generation and CO₂ emissions when formulating sustainable maintenance strategies is needed in the future work.

4.7. Conclusions

1. Chloride-induced corrosion is a significant threat to the durability and safety of reinforced concrete structures and infrastructure. Incorporating maintenance into the management of these structures is critical to ensure optimal levels of safety and serviceability during their operational life. While current studies focus on optimizing the effectiveness and cost of maintenance strategies, environmental constraints must also be considered since repair work generates a significant amount of waste, and cement production releases CO₂ emissions. Therefore, this chapter proposed the approach for integrating cost and sustainability in the selection of repair strategies. Three criteria are considered in the proposed framework for sustainable maintenance management, including: (1) costs, (2) CO₂ emissions and (3) waste generation.
2. Agencies have conflicting objectives to reduce costs, environmental impacts, and traffic disruptions while improving serviceability, functionality, and safety. A multi-criteria or multi-objective optimization approach can help to solve these conflicts. The compromise programming approach, namely the VIKOR method was proposed here to evaluate alternatives based on multiple criteria. This tool can improve the economic and environmental performance of a repair technique or compare the sustainability of different repair techniques. The proposed approach could be improved by considering uncertainties when formulating sustainable maintenance strategies in future works.
3. The remainder of the chapter provides examples to illustrate the proposed methodology of the cost-effective management strategy and the sustainable management strategy. In the first example, the effectiveness of different repair strategies can be compared in terms of the shape and thickness of the concrete replacement, taking into account three factors, including cost, waste generation, and CO₂ emissions. The results of the multi-criteria optimization analysis using the VIKOR method show that CCR3 is the most economical repair option and also the optimal solution for sustainable maintenance management.

4. In the second example, two repair strategies for rebuilding the cover using different repair materials are considered. For a sustainable maintenance management, repair strategy S2, which uses higher quality repair material, is found to be more efficient from an economic perspective and in terms of the environmental impact. However, there is a need for future experimental studies to investigate the long-term mechanical compatibility between the patch and the original concrete with different properties.

Closure

Summary and Conclusions

The closure section of this thesis summarizes the main findings and draws conclusions based on the research objectives outlined in the introduction. Through a comprehensive analysis of the data and information collected, this section aims to provide a deeper understanding of the research topic and its implications:

1. The first chapter discussed the life cycle stages of corroded RC structures and emphasized the importance of repair and maintenance management. It also emphasized the need to develop a maintenance model that can be used to evaluate the effectiveness of repair strategies. The model should take into account the complex process of chloride transport in repaired concrete, which involves a system of two layers of material. Despite the challenges in constructing this model, the main objective of this work is to provide a reliable tool for evaluating the response of structures to maintenance actions, for comparing the effectiveness of different repair options, and for efficiently performing probabilistic assessments.

In addition, different probabilistic approaches commonly used for maintenance management are found in the literature, including: time-dependent reliability analysis, risk analysis, Markov chain models, and Bayesian networks. Each approach has its own strengths and weaknesses and may be applicable to a particular problem. Among these methods, time-dependent reliability analysis is a promising approach for managing the maintenance of such structures. The approach involves predicting the probability of corrosion initiation and determining the time period for repair based on the allowable risk threshold. Considering the preventive and corrective maintenance strategies is useful to effectively manage the probability of corrosion to contribute to cost-effective maintenance planning.

Among the reliability methods, the LHS and surrogate-based methods are selected to perform probabilistic assessment of maintenance modeling subjected to one- and two-dimensional chloride attack. The choice of the method depends on the application and data characteristics. For surrogate-based method, the PCE method was chosen in this work because of its computational efficiency and high accuracy, which is advantageous for evaluating complex models that are time-consuming or expensive to run.

Finally, the importance of a sustainable maintenance management strategy for chloride-polluted RC structures was discussed, considering factors such as structural safety, life-cycle cost, and environmental impact. A multi-criteria optimization method was proposed to select repair solutions that minimize the cost and environmental impact of repair and maintenance activities.

2. Chapter 2 presented the formulation of the model for chloride ingress in repaired concrete, which is a significant advance over previous models because it is able to account for several factors of deterioration and repair processes. By including temperature, humidity, chloride binding capacity, and concrete aging factor, the model can provide more accurate predictions of the long-term effectiveness of various repair configurations. Furthermore, the model accounts for both one-dimensional and two-dimensional chloride flux in a two-layer system, which allows for more precise targeting of repair actions and potentially leads to more appropriate maintenance strategies.

The results of this chapter have important implications in maintenance planning for chloride-polluted RC structures. By considering factors such as the number of expected repairs, depth of repair concrete, and repair interval, decision-makers can develop optimal strategies that balance costs, environmental impact, and corrosion initiation risks. For RC structures subjected to 2D chloride attack, the circular concrete replacement strategy is recommended over orthogonal concrete replacement for the repair of RC columns. Although the given example may be challenging to execute perfectly in practice, the results of the analysis indicate the importance of carefully considering the geometry of a structure when developing repair strategies. This provides a foundation for the authors to further pursue advanced research in optimizing and feasibility in the shape and size of concrete repairs. Finally, the strong influence of local weather conditions on the corrosion process emphasizes the importance of continuous monitoring and maintenance to ensure the serviceability and durability of concrete structures over time.

3. Chapter 3 presented the probabilistic assessment and sensitivity analysis for repair modeling, which aim to estimate the probability of corrosion initiation and optimize

maintenance planning. Given the numerical requirements of the deterministic model, the study employed LHS for reliability analysis of 1D maintenance models and a surrogate-based method of PCE to approximate 2D maintenance models. The study focused on the probability of corrosion initiation associated with the serviceability limit state, since the selected maintenance policy is preventive.

As expected, the numerical examples showed that the probability of corrosion initiation can be reduced by applying shorter repair intervals and performing deep concrete cover rebuilding. The use of higher quality repair materials can also help reduce the risk of corrosion during repair cycles and extend the service life of RC structures.

The results of the sensitivity analysis indicated that controlling the thickness of the cover and the amount of concrete replaced during the construction and repair processes are critical parameters for corrosion protection.

Overall, the chapter provided valuable insights into maintenance modeling and optimization that can help practitioners formulate effective maintenance strategies that minimize corrosion risk and ensure the longevity of RC structures. However, real data (related to concrete chloride diffusivity and assessing criteria) is required to estimate correlations between the parameters used in the model and to improve the performance of maintenance model. The results of this study can serve as a basis for future theoretical and experimental efforts, so that the model-based approach to maintenance planning can be applied in practice.

4. Chapter 4 proposed a sustainable maintenance management strategy that takes into account both cost and sustainability criteria to ensure an optimal level of safety and serviceability of structures during their operational lifetime.

To achieve this, a multi-criteria or multi-objective optimization approach, such as the VIKOR method proposed in this chapter, can be used to evaluate the performance of repair alternatives based on multiple criteria. This approach can help resolve conflicting agency goals of reducing cost and environmental impact while improving serviceability, functionality, and safety.

The examples presented in this chapter demonstrate the importance of considering both economic and sustainability criteria in maintenance management. The use of higher quality repair materials may be more efficient from an economic and sustainable perspective, but it is important to investigate their mechanical compatibility with different concrete properties. In addition, comparing different repair strategies in terms of concrete replacement shape and thickness, taking into account sustainability criteria

such as cost, waste generation, and CO₂ emissions, can help decision-makers choose the most sustainable and cost-effective repair strategy for a given infrastructure project.

Perspectives

There are several ways in which the models of deterioration and repair/maintenance can be advanced. These include incorporating other physical factors, refining probabilistic models, and exploring alternative repair methods or adopt different approach for optimization. The following sections discuss these suggestions for improving the *repair model* and *maintenance strategies*

Chloride penetration model in repaired concrete (repair model)

- Develop a comprehensive model considering more realistic exposure conditions, chloride diffusion-convection coupling system in concrete, heat and moisture transfer.
- Investigate the effect of seasonal variations in temperature and humidity on chloride penetration in repaired concrete;
- Collect data for model parameters (related to chloride diffusivity of concrete and evaluation criteria) and consider their correlations from experimental studies;
- Examine the performance of repair adaptation strategies when considering climate change.
- Develop and apply a repair model that considers the relationship between concrete cracking and chloride penetration.

Maintenance strategies

- Investigate and model the effects of other repair techniques, such as coatings, waterproofing materials and corrosion inhibitors.
- Apply probabilistic cost-benefit assessment to determine the optimal maintenance schedule.
- Implement another surrogate-based reliability method, such as artificial neural networks for probabilistic assessment of both corrective and preventive maintenance models.
- This could be done by incorporating the Bayesian methods in maintenance planning and optimizing.
- Integrating of inspection results into the decision-making framework for optimising the maintenance of existing structures.
- Incorporate user costs and values beyond the expected costs in the life-cycle cost analysis for maintenance activities.
- Consider uncertainties when formulating sustainable maintenance strategies.

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Appendix A *Methods for reliability analysis*

A.1. Time-dependent reliability analysis

Time-dependent reliability analysis studies the evolution of failure probability over time. RC structures subjected to chloride-induced corrosion is an example of time-dependent problem. Therefore, the performance function $g(\mathbf{X},t)$ that takes into account the resistance $R(\mathbf{X})$ and the demand on the system $S(\mathbf{X})$ could be expressed as follows:

$$g(\mathbf{X}, t) = R(\mathbf{X}, t) - S(\mathbf{X}, t) \quad (\text{A.1})$$

The vector \mathbf{X} contains the random variables of the problem, some of them can be time-dependent. The safe region is defined as $\{\mathbf{x}: g(\mathbf{x}, t) > 0\}$, while the failure region is $\{\mathbf{x}: g(\mathbf{x}, t) \leq 0\}$, where \mathbf{x} is a realization of \mathbf{X} . To estimate the probability of failure (p_f), engineers use the time-dependent joint probability density function of \mathbf{X} , $f(\mathbf{x})$, and the time dependent probability of failure $p_f(t)$ could be computed as:

$$p_f(t) = P[g(\mathbf{X}, t) \leq 0] = \int_{g(\mathbf{x}, t) \leq 0} f(\mathbf{x}, t) d\mathbf{x} \quad (\text{A.2})$$

A.2. Methods for reliability assessment

Reliability methods are statistical techniques used to analyze the probability of failure or other performance problems in a system or structure. There are several reliability methods for structural analysis, including simulation-based (MCS, LHS), approximation-based (FORM, SORM), and surrogate-based (PCE, Kriging, ANNs). Given the nature of the problem and the complexity of the system (Chapter 3), this study has implemented Latin Hypercube sampling (LHS) and Polynomial Chaos Expansions methods for 1D and 2D maintenance modeling. Besides, the crude Monte Carlo Simulation method (MCS) is also performed on the constructed PCE-based surrogate model of maintenance modeling. Therefore, this section presents the methods of MCS and LHS, which are two sampling methods used to generate random samples

from a given probability distribution. MCS uses pure random sampling to obtain a sample, while LHS generates a pseudo-random sample that mimics a random structure. To give a general idea, MCS can be compared to a simple random sampling, while LHS can be compared to a stratified sampling strategy.

A.2.1 Monte-Carlo Simulation

In simple (crude) Monte Carlo simulation, samples of \mathbf{X} are generated according to the original joint probability density function $f_{\mathbf{X}}(\mathbf{x})$. Using an indicator function $I[g(\mathbf{x})]$, integration for the failure probability in equation (A.2) is extended over the whole domain:

$$p_f = \int_{\text{all } \mathbf{x}} I_{[g(\mathbf{x})]} f_{\mathbf{X}}(\mathbf{x}) d\mathbf{x} \quad (\text{A.3})$$

The state of the structure (either failure or survival) for each sample point is given by the indicator function:

$$\begin{aligned} I_{[g(\mathbf{x}_i)]} &= 1 \text{ if } g(\mathbf{x}_i) \leq 0 \\ I_{[g(\mathbf{x}_i)]} &= 0 \text{ if } g(\mathbf{x}_i) > 0 \end{aligned} \quad (\text{A.4})$$

The failure probability is estimated from eq. (A.3) as:

$$\bar{p}_f = \frac{\sum_{i=1}^{N_{sp}} I_{[g(\mathbf{x}_i)]}}{N_{sp}} \quad (\text{A.5})$$

Simulation results are subject to a statistical sampling error. Variance of the estimated failure probability is:

$$\text{Var}_{(\bar{p}_f)} = \frac{\sum_{i=1}^{N_{sp}} (I_{[g(\mathbf{x}_i)]} - \bar{p}_f)^2}{N_{sp}(N_{sp}-1)} \quad (\text{A.6})$$

Due to the small failure probability of engineering structures, usually a very large number of samples is required in crude Monte Carlo in order to keep the variance within acceptable limits. To reduce the dispersion of results, we can increase the number of samples or use importance sampling, or stratified random sampling (LHS).

A.2.2 Latin Hypercube Sampling

The Latin Hypercube Sampling (LHS) method is a Monte Carlo sampling technique particularly suited to sampling from multidimensional distributions. LHS builds on the concept of the Latin square, which is a square of dimension K^2 containing K observations, where each observation occurs once in each row and column into an arbitrary-dimensional hypercube.

To create a Latin hypercube, each dimension of the multidimensional distribution is divided into M divisions, and N_{sp} possible samples are generated according to a particular algorithm, that shows an exponential relationship, as follows:

$$N_{sp} = (M!) K^{-1} \quad (\text{A.7})$$

Compared to traditional random sampling methods, LHS provides more efficient coverage of the parameter space with relatively few samples. This is because the samples are uniformly distributed across the hyperplane, which provides a better approximation of the empirical distribution function. This efficiency is particularly advantageous when dealing with high-dimensional parameter spaces. Figure A.1 shows a comparison between random sampling and LHS for $K=2$ and $N_{sp} = 100$ and illustrates the advantage of LHS in achieving a more uniform coverage of the parameter space.

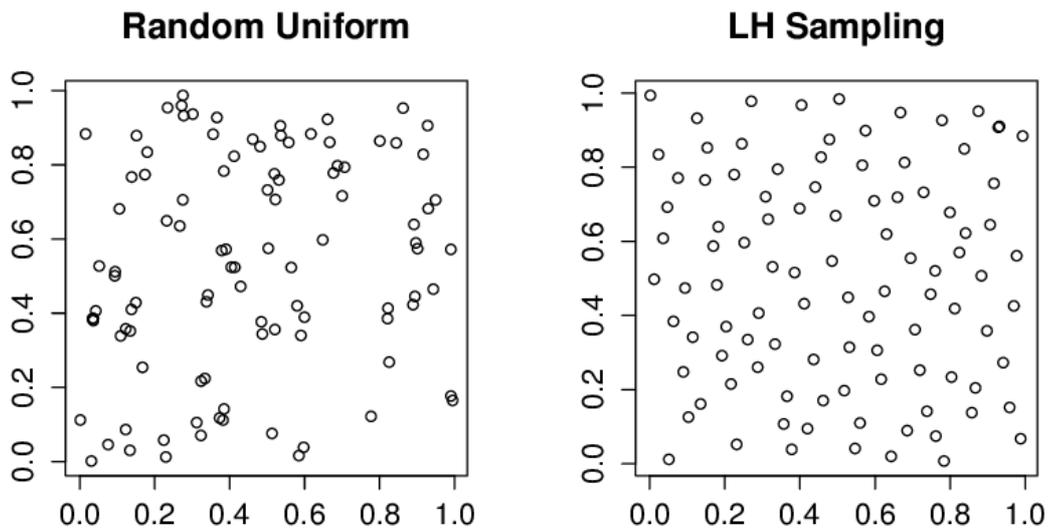


Figure A.1: Comparison between random sampling and LHS

Appendix B *The VIKOR method for multi-criteria optimisation*

VIKOR (VIsekriterijumska KOmpromisno Rangiranje) is a method of multi-criteria compromise programming used for decision-making problems with multiple criteria. The VIKOR method aims to find the best compromise solution among all possible alternatives based on the distance between the alternatives and the ideal solution. It helps to solve multiple criteria decision making problems regarding its two advantages; it provides a maximum group utility of the majority and a minimum of the individual regret of the opponent ((San Cristóbal Mateo, 2012). The VIKOR method includes the following steps:

Step 1: Determine the best value f_i^* and the worst value f_i^- of all criteria i ($i = 1, 2, \dots, n$) for each solution j

If criterion i is a profit criterion:

$$f_{ij}^* = \max f_{ij} \text{ and } f_{ij}^- = \min f_{ij} \quad (\text{B.1})$$

If criterion j is a cost or risk criterion:

$$f_{ij}^* = \min f_{ij} \text{ and } f_{ij}^- = \max f_{ij} \quad (\text{B.2})$$

Step 2: Normalization of the criteria

$$r_{ij} = \frac{f_{ij}^* - f_{ij}}{f_{ij}^* - f_{ij}^-} \quad (\text{B.3})$$

Step 3: Weighting the criteria

The criteria are then weighted based on their relative importance. This can be done using various methods such as the Analytic Hierarchy Process (AHP) or the entropy method.

Step 4: Calculating the utility value (S_j) and regret value (R_j) for each alternative

$$S_j = \sum_{i=1}^n w_i \left[\frac{(f_{ij}^* - f_{ij})}{(f_{ij}^* - f_{ij}^-)} \right] = \sum_{i=1}^n w_i r_{ij} \quad (\text{B.4})$$

$$R_j = \max \left[w_i \left[\frac{(f_{ij}^* - f_{ij})}{(f_{ij}^* - f_{ij}^-)} \right] \right] = \max[w_i r_{ij}] \quad (\text{B.5})$$

Step 5: Calculating of the VIKOR index, Q_j . The VIKOR index is calculated for each alternative using the following formula:

$$Q_j = \nu \left[\frac{S_j - S^-}{S^+ - S^-} \right] + (1 - \nu) \left[\frac{R_j - R^-}{R^+ - R^-} \right] \quad (\text{B.6})$$

where $S^+ = \max S_j$ and $S^- = \min S_j$

$$R^+ = \max R_j \text{ and } R^- = \min R_j$$

$0 \leq \nu \leq 1$, ν is introduced as the weight of the "group utility" and $(1-\nu)$ is the weight of "individual regret". Typically, $\nu = 0.5$.

Step 6: Ranking of Alternatives

Rank the solutions according to the values of S_j , R_j , Q_j in ascending order. The solution with the smallest value of Q is the best solution, and it must satisfy the following two conditions:

- *Condition 1:* Acceptable advantage: $Q(A_2) - Q(A_1) \geq 1/(j-1)$. (B.7)

where A_1, A_2 being the solutions ranked 1st and 2nd in the ranking table.

- *Condition 2:* Acceptable stability in decision-making: Solution A_1 must have S or R value ranked 1st.

There can be three cases: 1) the condition of an acceptable advantage and the condition of acceptable stability of the decision are met. Solution A_1 is the best and has no alternative; 2) the condition of acceptable advantage is not met, and the condition of acceptable decision stability is met. Solution A_1 is the best, but has alternative solution A_2 ; 3) the condition of acceptable advantage is not met. The condition for solution A_k from Q ranking is:

$$Q(A_k) - Q(A_1) < 1/(j-1) \quad (\text{B.8})$$

for every k in range from 2 to n . The first variant, which will meet the above condition is the last alternative solution.

Résumé étendu en Français

1. Contexte

Les systèmes d'infrastructure civile en béton armé sont des atouts importants pour le développement socio-économique de tout pays. La conception et la maintenance de ces systèmes pour une durée de vie donnée ont été mondialement reconnues comme des questions critiques (Bastidas-Arteaga et al., 2011). Les structures en béton armé se caractérisent par une grande durabilité ; cependant elles sont exposées au cours de leur durée de vie à des influences internes et externes qui affectent leur performances, aptitude au service et la sécurité. Entre autres, la corrosion de l'acier dans le béton due à la pénétration des chlorures est la principale cause de la détérioration des structures en béton armé et du raccourcissement éventuel de leur durée de vie (Stipanovic Oslakovic, Bjegovic et Mikulic, 2010). Après la phase de construction, le processus d'endommagement induit par la corrosion peut être divisé en deux phases principales : la phase d'initiation et la phase de propagation. L'alcalinité élevée de la pâte de ciment entourant l'acier d'armature crée un état réactif et passif. Cependant, une quantité élevée de chlorure à proximité des barres d'armature peut rompre ce film passif pour initier la corrosion (Tuutti, 1982 ; Liu et al., 2022). La période de propagation suit l'initiation de la corrosion et est définie comme la période entre l'initiation et la destruction critique de l'acier. Cette phase de propagation est relativement courte par rapport à la phase d'initiation (Bastidas-Arteaga et al., 2011 ; Ma et al., 2017). Par conséquent, la période entre l'état initial d'une structure et le début de la corrosion est critique puisqu'elle détermine la durée de vie de la structure (Tuutti, 1982).

On sait que les environnements à forte teneur en chlorure accélèrent la détérioration par corrosion et peuvent réduire la durée de vie des structures dans certains cas d'une fourchette déjà prévue de 50 à 100 ans à une fourchette de 10 à 30 ans (Engelund, Sorensen et Sorensen, 1999 ; Zhao et Zin, 2006). En outre, les dommages induits par les chlorures peuvent être un processus lent et silencieux pouvant conduire à des dommages importants avant qu'ils ne soient remarqués. L'entretien et la réparation des structures peuvent prolonger leurs aptitudes au service et maintenir leur sécurité et, par conséquent, leur durée de vie (Liu et Frangopol, 2004 ; Cheng et al., 2022). En effet, dans le cadre d'une perspective préventive, les opérations de

maintenance et de réparation devraient être mises en œuvre avant ou au moins dès le début de la corrosion. Par conséquent, la détection et la surveillance précoces de la corrosion sont essentielles pour éviter des réparations coûteuses et importantes. À cette fin, des inspections visuelles, des méthodes d'essais destructifs (DT) et d'autres techniques avancées d'essais non destructifs (END) peuvent être utilisées, mais ces méthodes peuvent être coûteuses et chronophages ; dans certains cas, ces techniques peuvent conduire à un diagnostic erroné (inspection visuelle), ou causer des dommages supplémentaires à la structure. Par conséquent, le développement d'un modèle robuste pour prédire la durée de vie des structures en béton armé polluées par les chlorures et des stratégies efficaces pour l'inspection et la réparation périodiques jouent un rôle important pour atteindre l'objectif de durabilité et assurer le fonctionnement continu des structures réparées (Liu et Frangopol, 2004 ; Bastidas-Arteaga et Schoefs, 2012). Plusieurs modèles de complexité variable ont été proposés pour prédire et décrire la dégradation (Song et al., 2009 ; Soleimani et al., 2010), mais l'application déterministe peut être trompeuse en raison des incertitudes liées aux paramètres et aux modèles clés. Des modèles de dégradation probabilistes ont été développés pour tenir compte de ces incertitudes. Cependant, la modélisation de la pénétration des ions chlorures dans les structures en béton armé réparées est une tâche complexe, car il faut tenir compte non seulement du processus de dégradation initial, mais aussi de l'état après la réparation (Barkey, 2004 ; Shi, 2020), étant donné que la réparation ne garantit pas le rétablissement complet de la structure dans son état d'origine. Par conséquent, la modélisation probabiliste des processus de dégradation des structures en béton armé réparées doit être traitée comme un sujet de recherche distinct et important. Sur cette base, la modélisation probabiliste de la maintenance peut aider à la sélection des stratégies et des matériaux de réparation et fournir une estimation plus réaliste de la durée de vie restante des structures en béton armé.

Il faut également noter que ces activités de maintenance génèrent des coûts et des impacts environnementaux. Si les considérations économiques sont souvent au premier plan des décisions de gestion, il est important de considérer également les questions environnementales et sociales (Bastidas-Arteaga et Schoefs, 2015 ; Petcherdchoo, 2015). La production de ciment qui est un composant important du béton, contribue de manière significative aux émissions de gaz à effet de serre ; et les déchets générés pendant les travaux de réparation peuvent avoir un impact négatif sur l'environnement. Par conséquent, une gestion efficace des structures en béton armé corrodées nécessite une approche durable qui équilibre les facteurs économiques, environnementaux et sociaux. Par exemple, la sélection de stratégies et de matériaux d'entretien appropriés peut avoir une incidence significative sur l'impact environnemental des structures en béton armé au cours de leur cycle de vie. L'approche traditionnelle de maintenance réactive, qui consiste à attendre qu'un dommage se produise avant d'agir, peut être coûteuse et dommageable pour l'environnement. En revanche, une approche proactive de la maintenance, qui comprend des inspections régulières et des mesures préventives, peut réduire la nécessité d'effectuer des réparations importantes et l'impact environnemental associé (Bastidas-Arteaga et Schoefs,

2015). Il est donc essentiel de mener une étude complète qui prenne en compte tous ces facteurs afin de formuler un cadre décisionnel qui identifie les solutions optimales pour maximiser la durabilité et minimiser les coûts. Dans ce contexte, la méthodologie développée dans cette étude intègre les aspects suivants :

- un modèle complet de pénétration des chlorures dans le béton avant/après réparation,
- une approche probabiliste efficace pour l'optimisation des stratégies de réparation, et
- l'intégration des coûts et de l'impact environnemental dans la gestion de la maintenance.

2. Objectifs et champ d'application

L'objectif de cette étude est de développer une modélisation probabiliste et d'optimiser les stratégies de réparation pour un entretien durable des structures en béton armé soumises à des infiltrations des ions chlorures. L'étude se concentrera sur la réalisation des objectifs spécifiques suivants :

- Évaluer les méthodes actuelles qui peuvent résoudre les aspects considérés de la gestion des structures en béton armé corrodées, mettre en évidence leurs avantages et leurs inconvénients, et identifier une approche efficace pour résoudre ce problème.
- Construire un modèle complet pour la pénétration des ions chlorures dans le béton réparé, qui sera utilisé pour étudier l'efficacité des différentes stratégies de réparation, la planification et l'optimisation de la maintenance.
- Intégrer le modèle de maintenance dans un cadre probabiliste pour tenir compte des incertitudes associées aux processus de détérioration et de réparation.
- Développer une méthode pour optimiser les performances de la stratégie de maintenance et de sa planification.
- Intégrer les contraintes de coût et d'environnement dans le processus de prise de décision en utilisant une approche appropriée pour évaluer la durabilité des stratégies de maintenance.

3. Modèle de détérioration

3.1 *Équations de base du modèle de pénétration des chlorures*

La pénétration des ions chlorures dans les structures marines en béton armé est généralement régie par la diffusion, la convection, l'adsorption, la perméation et la liaison des chlorures, entre autres mécanismes. Cependant, la seconde loi de diffusion de Fick est souvent considérée comme le moteur principal pour caractériser la distribution de la teneur en chlorure dans le béton (Tuutti, 1982). La réduction du mécanisme de transport du chlorure à un modèle basé sur la diffusion est raisonnable et acceptable dans la mesure où les structures en béton armé fonctionnent dans un environnement marin considéré comme saturé dans certaines zones d'exposition, par exemple les zones atmosphériques ou submergées (Wang, Gong et Wu, 2019 ;

Li et al., 2022 ; Zacchei et Bastidas-Arteaga, 2022b). En fait, le modèle basé sur la diffusion nous permet de développer un cadre de maintenance pour simuler des réparations de reconstruction de couvertures multiples avec une difficulté technique et un temps de calcul raisonnable, ce qui est l'aspect principal de la modélisation probabiliste (chapitre 3). Sous ces hypothèses, l'équation principale régissant le transport unidirectionnel des chlorures dans le béton est présentée ci-dessous :

$$\frac{\partial C_{fc}}{\partial t} = D_c^* \frac{\partial^2 C_{fc}}{\partial x^2} \quad (1)$$

Pour un écoulement bidimensionnel, l'équation (1) devient :

$$\frac{\partial C_{fc}}{\partial t} = D_c^* \left(\frac{\partial^2 C_{fc}}{\partial x^2} + \frac{\partial^2 C_{fc}}{\partial y^2} \right) \quad (2)$$

où C_{fc} (kg/m³ béton) est la concentration de chlorures au point (x, y) d'une section horizontale de béton au temps d'exposition t (s), D_c^* (m²/s) représente le coefficient apparent de diffusion, x (m) et y (m) sont les distances de la surface d'exposition dans les directions X et Y, respectivement. D_c^* (m²/s) dans l'équation (1)-(2) est estimée comme suit :

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

La relation entre les chlorures libres et les chlorures liés, $\partial C_{bc}/\partial C_{fc}$ est définie comme la capacité de liaison des chlorures, qui peut être formulée par l'isotherme de liaison de Langmuir :

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

Compte tenu de la température $f_1(T)$, de l'âge du béton $f_2(t)$ et de l'humidité relative $f_3(h)$, le coefficient de diffusion du chlorure dépendant du temps D_c dans l'équation (3) est défini comme suit:

$$D_c = D_{c,ref} f_1(T) f_2(t) f_3(h) \quad (5)$$

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

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

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

La description des paramètres utilisés dans le modèle de pénétration des chlorures, tels que B_c , w_e , α , β , D_{ref} , U_c , R , T_{ref} , m_c , t_{ref} , et h_c , est présentée dans le tableau 1. Les valeurs de ces paramètres peuvent être trouvées dans des études antérieures (Baant et Najjar, 1971 ; Page, Short et El Tarras, 1981 ; Anna V. Saetta, 1993 ; Luping et Nilsson, 1993 ; L. O. Nilsson, 1994 ; Xi, Bažant et Jennings, 1994 ; Glass et Buenfeld, 2000 ; Martín-Pérez, Pantazopoulou et Thomas, 2001 ; Val, 2006).

Tableau 1. Paramètres utilisés dans le modèle de pénétration des chlorures

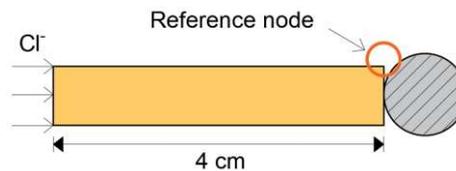
Paramètre	Description	Unité	Valeur
w_e	Teneur en eau évaporable		0.1316
α	Constantes de liaison		0.09
β			0.1185
$D_{c,ref}$	Coefficient de diffusion du chlorure de référence	m^2/s	3×10^{-11}
U_c	Énergie d'activation du transport du chlorure	KJ/mol	41.8
R	Constante de gaz	J/ (mol K)	8.314
T_{ref}	Température de référence	K	296
T	Température absolue	K	dépend des conditions environnementales externes
m_c	Facteur de vieillissement		0.15
t_{ref}	Temps de référence auquel $D_{c,ref}$ est mesuré	days	28
h_c	Humidité relative à laquelle D_c chute de moitié entre sa valeur maximale et sa valeur minimale		0.75
h	Humidité relative à l'intérieur du béton		dépend des conditions environnementales externes

3.2 Validation du modèle

Pour le modèle de diffusion 1D, une étude comparative est réalisée entre les résultats obtenus avec : (1) le code développé dans FreeFem++ ; et (2) la solution analytique du problème. La solution analytique basée sur la fonction d'erreur est formulée comme suit:

$$C_{fc}(x, t) = C_{fc}^S \left[1 - \operatorname{erf} \left(\frac{x}{2\sqrt{D_c t}} \right) \right] \quad (9)$$

où $C_{fc}(x, t)$ est la concentration de chlorures libres à la profondeur x après le temps t , $\operatorname{erf}()$ est la fonction d'erreur. La figure 1 montre la configuration du cas étudié.

**Figure 1:** Configuration de l'étude de cas

Les résultats de cette analyse se concentrent sur le nœud de référence, qui est situé à une profondeur de 4 cm. Certaines hypothèses sont formulées comme suit :

- le coefficient de diffusion effectif du chlorure est défini comme constant dans la solution analytique : $D_c = 2.10^{-12} m^2/s$;

- la température et l'humidité sont considérées comme constantes dans le modèle, $T = 15^\circ\text{C}$, $h = 0.7$, le modèle prend également en compte les effets du vieillissement du béton et de la fixation du chlorure ; et
- la concentration des chlorures à la surface est constante : $C_{fc}^s = 4 \text{ kg/m}^3$;

La figure 2 montre la concentration de chlorures à la profondeur étudiée pour une entrée de chlorure unidimensionnelle. On peut observer qu'il y a une différence entre la solution numérique et la solution analytique après 20 ans d'exposition, mais elle n'est pas significative. Le programme FreeFem++ semble sous-estimer l'évolution des chlorures dans la dernière phase de la vie utile d'une structure. Ceci est dû au fait que les effets de la température, de l'humidité, du vieillissement du béton et de la liaison du chlorure sont négligés dans la formulation analytique. En particulier, le vieillissement du béton en relation avec la pénétration des chlorures, peut entraîner une diminution substantielle du coefficient de diffusion apparent au fil du temps. Par conséquent, cette comparaison peut être considérée comme efficace pour valider le modèle de transport unidimensionnelle.

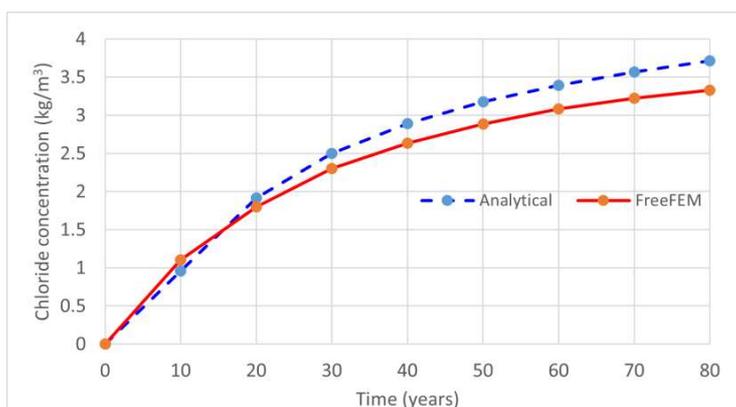


Figure 2: Validation du modèle pour la pénétration unidimensionnelle du chlorure

Pour le modèle 2D, la simulation numérique est validée par comparaison avec les expériences de diffusion de chlorures menées par Yu et al. (2021). Dans leurs travaux, des cubes de béton de 100 mm x 100 mm x 400 mm ont été immergés dans un environnement marin artificiellement saturé. Après une période de durcissement de 28 jours à la température de 20°C , quatre surfaces du béton ont été recouvertes de résine époxy, permettant aux ions chlorure de l'environnement de pénétrer par les deux surfaces adjacentes restantes. Les échantillons de béton ont ensuite été placés dans un système de simulation automatisé et exposés à de l'eau de mer synthétique. Des données expérimentales sur la distribution de la concentration de chlorure ont été obtenues en examinant leur teneur aux coins et bords des échantillons après 70 et 180 jours d'exposition à l'environnement marin artificiel. Afin de vérifier les conditions de l'environnement marin artificiellement simulé, un examen des expériences pertinentes (Wang et al., 2018 ; Wang, Gong et Wu, 2019 ; Wu et al., 2020) a été effectué pour déterminer la température ambiante et l'humidité relative pendant les essais. En général, les échantillons sont placés dans des conditions environnementales contrôlées avec une température d'environ 20°C et une humidité relative de 70 % en moyenne (allant de 60 à 80 %). La valeur exacte étant inconnue, trois températures ambiantes différentes (18°C , 20°C et 23°C) et une valeur moyenne d'humidité de 0,7 ont été utilisées pour valider le modèle. La concentration de chlorure simulée a été comparée aux résultats de chlorure testés

après 70 jours (désignés par M70-T18, M70-T20, M70-T23) et 180 jours (désignés par M180-T18, M180-T20, M180-T23), comme le montre la Figure 3. Même si ces résultats sont encore basés sur certaines hypothèses, la concordance entre les courbes de profil de chlorure modélisées et les mesures expérimentales de chlorure indique la validité du modèle 2D de transport.

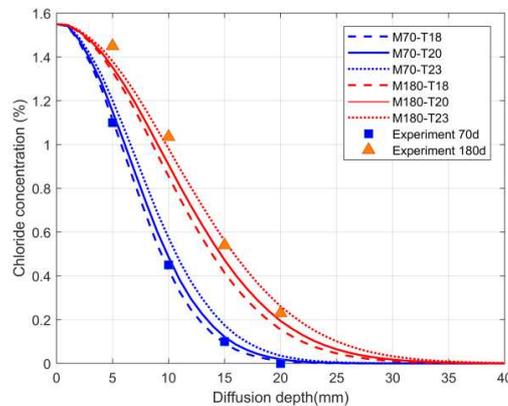


Figure 3: Chloride diffusion model validation with data taken from (Yu *et al.*, 2021)

Le travail expérimental de Roy, Liam Kok Chye et Northwood (1993) est également examiné pour valider les résultats du modèle numérique. Les échantillons de béton étaient des prismes rectangulaires de 100 mm x 100 mm x 500 mm. Les données ont été obtenues à partir de l'essai de pénétration du chlorure dans un béton standard après une période d'exposition de 80 semaines dans une zone submergée à une température de 20°C et une humidité relative de 0,75. La figure 4 montre la tendance de la distribution des chlorures libres à partir de la modélisation FreeFEM++. La comparaison des résultats obtenus avec la modélisation numérique et les expériences précédentes confirme la précision de la prédiction du modèle pour la modélisation du transfert en 2D.

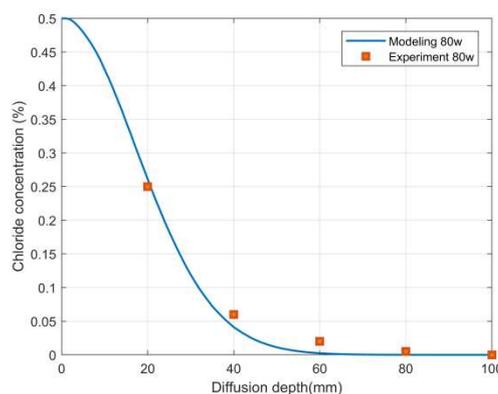


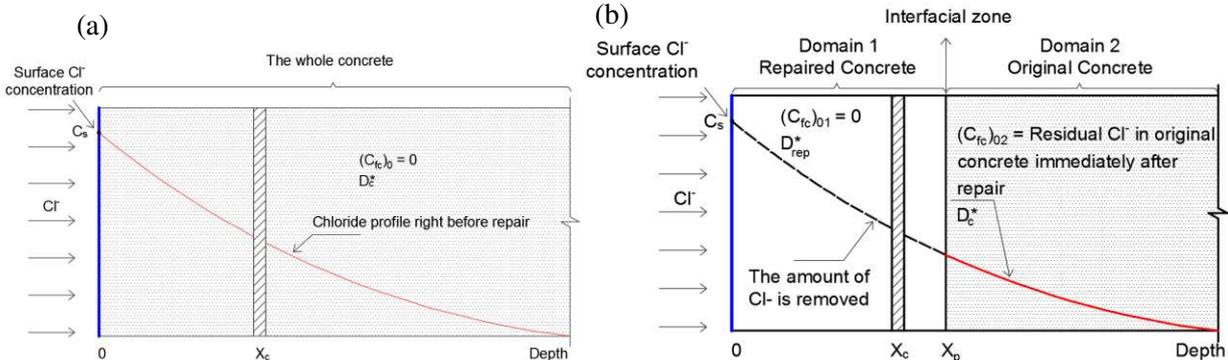
Figure 4: Chloride diffusion model validation with data taken from (Roy, Liam Kok Chye and Northwood, 1993)

Cette validation est essentielle pour le développement d'un modèle d'entretien qui incorpore l'influence des réparations de remplacement du béton, où la diffusion des ions chlorure reste le principal mécanisme de contrôle à l'intérieur du béton. Le modèle de pénétration des chlorures prenant en compte la réparation sera présenté dans la section suivante.

4. Modèle de pénétration des chlorures tenant compte des actions de réparation/entretien

Dans cette section, une description détaillée du transport des chlorures dans les structures en béton armé est illustrée en y intégrant la réparation. Le modèle de maintenance unidimensionnel est considéré en premier lieu. La figure 5a illustre la pénétration du chlorure immédiatement avant la réparation. À ce stade, la concentration de chlorure adjacente à la surface du béton est la principale source d'ions de chlorure. La teneur initiale en chlorure est supposée égale à 0, $(C_{fc})_0 = 0$ pour tout le volume de béton et $D_c^*(t)$ est défini comme le coefficient de diffusion du béton d'origine.

La figure 5b montre le profil de chlorure immédiatement après la réparation de la réfection de la couverture. Au moment de la réparation t_i , la couverture de béton contaminée par les chlorures est enlevée jusqu'à la profondeur X_p . X_p est défini comme la profondeur réparée ou l'épaisseur du béton remplacé, qui n'est pas nécessairement la même profondeur que la couverture de béton, X_c ($X_p > X_c$). Par conséquent, la quantité d'ions chlorure entre la surface du béton et la profondeur de réparation est éliminée, et un matériau de réparation sans chlorure, à base de ciment, est appliqué au domaine de réparation. Les domaines de béton réparé et de béton résiduel non réparé sont représentés par Ω_1 et Ω_2 , respectivement. À ce stade, seul Ω_1 est exposé à la concentration de chlorure en surface dans l'environnement et sera d'abord soumis à la pénétration de chlorure avant d'atteindre le béton d'origine. Le principal mécanisme de diffusion de la pénétration des chlorures dans le matériau réparé est également modélisé par les équations (1)-(8), où la condition limite est toujours le flux de chlorures traversant la surface du béton, mais où les conditions initiales dans Ω_1 et Ω_2 sont différentes. Alors que la teneur initiale en chlorure dans le domaine du matériau réparé est nulle, $(C_{fc})_{01} = 0$, cette concentration dans l'ancien béton correspond aux ions chlorure résiduels, qui n'ont pas été éliminés après la réparation. En outre, le coefficient de diffusion du béton de réparation, D_{rep}^* , et les paramètres du modèle liés aux propriétés des matériaux sont mis à jour pour refléter le nouveau matériau de réparation et le vieillissement du nouveau béton sur la base des Eqs. (5)-(8), tandis que le processus de diffusion dans l'ancien béton est régi par les paramètres du modèle d'origine.



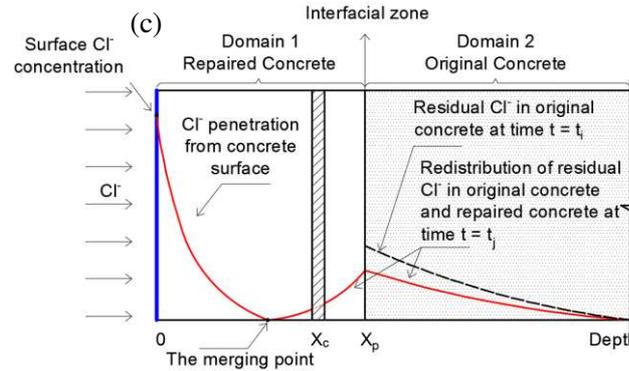


Figure 5: (a) Profil des chlorures immédiatement avant la réparation, (b) Profil des chlorures immédiatement après la réparation à $t = t_i$, (c) Profil des chlorures après la réparation à $t = t_j$ ($t_j > t_i$)

Après la réparation, il y a trois étapes principales de distribution des chlorures, comme le montre la Figure 5c. Tout d'abord, les ions chlorures résiduels dans Ω_2 seront redistribués dans le domaine du béton non réparé. Deuxièmement, ces chlorures résiduels se diffusent également à travers la zone de transition et dans le nouveau domaine de béton offrant une faible résistance à la diffusion des chlorures immédiatement après réparation. Troisièmement, à l'instant t_j (avec $t_i > t_j$), la diffusion des ions chlorure dans le béton d'origine et le béton réparé est une combinaison de la pénétration des chlorures à partir de la surface exposée du béton et de la redistribution des chlorures à l'intérieur du béton. En raison de la différence des coefficients de diffusion dans deux domaines Ω_1 et Ω_2 , le problème implique un coefficient de diffusion dépendant du domaine spatial après la réparation $D_{ar}^*(x, t)$:

$$D_{ar}^*(x, t) = \begin{cases} D_{rep}^*(t_{rp}) & \text{if } 0 < x < X_p \text{ (Domaine 1)} \\ D_0^*(t) & \text{if } x \geq X_p \text{ (Domaine 2)} \end{cases} \quad (10)$$

où t_{rp} est le temps d'exposition pour le nouveau domaine de béton de réparation qui est calculé à partir du moment où le nouveau matériau est installé pour remplacer la partie de béton contaminée par les chlorures ($t > t_{rp}$). Par conséquent, l'équation aux dérivées partielles (EDP) utilisée dans cette étape, basée sur la loi de diffusion de Fick, peut être écrite comme suit :

$$\frac{\partial C_{fc}}{\partial x} = \frac{\partial}{\partial x} D_{ar}^*(x, t) \frac{\partial C_{fc}}{\partial x} \quad (11)$$

Pour l'écoulement bidimensionnel des chlorures dans le béton, les équations (10) et (11) deviennent :

$$D_{ar}^*(x, y, t) = \begin{cases} D_{rep}^*(t_{rp}) & \text{if } (x, y) \in \text{Domaine 1} \\ D_c^*(t) & \text{if } (x, y) \in \text{Domaine 2} \end{cases} \quad (12)$$

$$\frac{\partial C_{fc}}{\partial t} = \left(\frac{\partial}{\partial x} D_{ar}^*(x, y, t) \frac{\partial C_{fc}}{\partial x} + \frac{\partial}{\partial y} D_{ar}^*(x, y, t) \frac{\partial C_{fc}}{\partial y} \right) \quad (13)$$

Les problèmes liés à la résolution de l'EDP se posent à nouveau et peuvent être compliqués par de multiples réparations au cours de la durée de vie de la structure en béton armé.

5. Etude probabiliste pour l'analyse de fiabilité

Pour réparer le béton corrodé, il est généralement admis que le moment optimal pour effectuer les réparations se situe avant la fin de la phase de corrosion (Bastidas-Arteaga et Schoefs, 2012). En effet, la période de propagation de la corrosion, définie comme le temps écoulé entre le début de la corrosion et la destruction critique de l'acier, est généralement courte par rapport à la phase d'initiation de la corrosion. Si la corrosion progresse rapidement, la phase de propagation peut être réduite à quelques mois dans le pire des cas, ce qui laisse peu de temps pour une réparation adéquate. Par conséquent, les stratégies de maintenance mettent souvent l'accent sur des mesures préventives plutôt que correctives. La probabilité de défaillance utilisée dans cette étude est donc la probabilité d'initiation de la corrosion, qui est calculée sur la base de l'état limite d'aptitude au service.

5.1 Probabilité de déclenchement de la corrosion

Le moment de l'initiation de la corrosion, T_{ini} , est défini comme l'instant où la concentration de chlorure dans l'épaisseur de la couverture atteint un niveau critique, C_{crit} . La concentration de chlorure à laquelle la couche passive sur les barres d'acier est rompue, entraînant le début de la corrosion de l'acier, est représentée par cette valeur critique. On suppose que C_{crit} est une variable aléatoire en raison de l'incertitude des propriétés des matériaux en béton. Pour les structures en béton armé réparées par la technique de reconstruction par recouvrement, le moment d'initiation de la corrosion peut se produire à plusieurs reprises en fonction des conditions d'exposition, de l'évaluation de la durée de vie et du nombre de réparations. Il est déterminé en évaluant la concentration de chlorure en fonction du temps à la profondeur de la couverture en utilisant les solutions des équations différentielles partielles non linéaires présentées dans la section 3.1. La fonction de distribution cumulative du temps écoulé avant l'initiation de la corrosion, $F_{T_{ini}}(t)$, est définie comme suit :

$$F_{T_{ini}}(t) = \text{Prob} \{T_{ini} \leq t\} = \int_{T_{ini} \leq t} f(\mathbf{x}) d\mathbf{x} \quad (14)$$

où \mathbf{X} est le vecteur des variables aléatoires considérées et $f(\mathbf{x})$ la fonction de densité de probabilité conjointe de \mathbf{X} . La fonction d'état limite devient alors :

$$g(\mathbf{X}, t) = C_{fc}(\mathbf{X}, t, X_c) - C_{crit} \quad (15)$$

où $C_{fc}(\mathbf{X}, t, X_c)$ est la concentration de chlorure à la profondeur de la couverture et au temps t , X_c est l'épaisseur de la couverture en béton. La probabilité de défaillance, p_f , est transformée en probabilité d'initiation de la corrosion dans ce cas, et est calculée en intégrant la fonction de probabilité conjointe sur le domaine de défaillance (Monteiro et Gonçalves, 2022) :

$$p_f = P[g(\mathbf{X}) \leq 0] = \int_{g(\mathbf{X}) \leq 0} f(\mathbf{x}) d\mathbf{x} \quad (16)$$

5.2 Méthodologie de l'analyse probabiliste

Récemment, la méthode de simulation de Monte-Carlo (MCS) et la méthode de fiabilité du premier ordre (FORM) ont été appliquées pour l'évaluation probabiliste de la durabilité des structures en béton armé (Ma et al., 2020 ; Lehner, Horňáková et Hrabová, 2021). Notamment, la plupart des études utilisant des analyses basées sur la fiabilité telles que MCS, FORM ou SORM (méthode de fiabilité de second ordre) ont été réalisées sur des modèles simplifiés d'infiltration de chlorure utilisant des solutions analytiques basées sur les lois de diffusion de Fick (Nogueira, Leonel et Coda, 2012 ; Yu, Ning et Li, 2017). Cependant, il est largement démontré que FORM et SORM peuvent être inexacts pour des problèmes probabilistes avec des fonctions d'état limite non linéaires, tandis que le coût de calcul de MCS peut être prohibitif, en particulier lorsqu'il est associé à une analyse par éléments finis (EF) coûteuse en temps de calcul. En particulier, l'utilisation de modèles d'éléments finis incorporant des éléments de zone cohésive dans la modélisation du transport de chlorure dans le béton réparé est très exigeante en termes de calcul (Beaurepaire et al., 2012). Ceci est dû à plusieurs facteurs tels que la présence simultanée de deux mécanismes de diffusion de chlorure, la nécessité de modéliser la progression temporelle de la concentration de chlorure à travers de multiples réparations, la prise en compte de différentes méthodes de réparation et la présence d'incertitudes dans plusieurs variables aléatoires. C'est pourquoi une autre méthode d'échantillonnage, appelée échantillonnage hypercube latin (LHS), a été adoptée pour surmonter ces contraintes. La technique LHS propose une distribution uniforme des points d'échantillonnage sur l'ensemble du spectre des valeurs possibles, ce qui permet de réduire le nombre d'échantillonnages nécessaires pour obtenir des résultats satisfaisants. En couvrant l'ensemble du spectre de chaque variable aléatoire, la méthode LHS garantit une couverture complète (Bastidas-Arteaga et al., 2011). La méthode LHS s'est avérée être une alternative plus efficace à l'échantillonnage aléatoire de base dans divers contextes, ce qui en fait une approche populaire et robuste pour la conception d'expériences et d'analyses probabilistes visant à évaluer la probabilité de ruine (Olsson, Sandberg et Dahlblom, 2003 ; Dastmalchi et Burton, 2021). En raison de sa facilité de mise en œuvre et de sa durée raisonnable, la méthode LHS est utilisée dans cette étude pour réaliser une analyse probabiliste de la modélisation de la maintenance liée à la pénétration unidimensionnelle des chlorures. De plus amples informations sur cette méthode sont disponibles à l'annexe A.

Concernant le modèle bidimensionnel de transfert, l'application de méthodes basées sur la simulation telles que LHS pour effectuer une modélisation probabiliste de la maintenance reste un défi en raison de leur coût de calcul important. Dans ce cas, les méthodes basées sur les substituts ou métamodèles fournissent une alternative appropriée, couramment utilisée dans les problèmes d'ingénierie pour imiter le comportement des modèles déterministes. Ces méthodes, telles que la méthode de la surface de réponse, les réseaux neuronaux artificiels (RNA), le krigeage et l'expansion du chaos polynomial (PCE), peuvent être utilisées pour approximer les résultats des simulations par éléments finis tout en réduisant les besoins de calcul. Par ailleurs,

ces méthodes sont particulièrement adaptés à l'analyse de la fiabilité, qui exige de nombreux calculs de la fonction de performance (Sudret, 2014 ; Hariri-Ardebili et Sudret, 2020).

Le choix de la méthode dépend de la complexité du système à modéliser, des données disponibles et du niveau de précision et d'efficacité de calcul souhaité. Parmi ces méthodes, la PCE est choisie parce qu'elle peut capturer des relations non linéaires entre les variables d'entrée et les réponses de sortie sans nécessiter des évaluations de modèle excessives (Sudret, 2014), en particulier pour un système compliqué de modèles de maintenance soumis à une pénétration bidimensionnelle de chlorure. En outre, la méthode PCE peut être utilisée pour effectuer des analyses de sensibilité afin de quantifier les effets des variables d'entrée sur la sortie. Les indices de Sobol, par exemple, peuvent être calculés à partir des coefficients PCE pour déterminer l'influence de chaque variable d'entrée sur la réponse. Cette méthode est particulièrement utile pour les problèmes à haute dimension où les méthodes conventionnelles d'analyse de sensibilité, telles que la simulation de Monte Carlo, ne sont pas réalisables en raison de leur coût informatique élevé (Deman et al., 2016).

Par conséquent, cette étude utilise la méthode LHS pour effectuer une évaluation probabiliste d'un modèle de maintenance 1D et la méthode PCE pour évaluer un modèle de maintenance probabiliste 2D pour la corrosion induite par les chlorures dans le béton réparé. Leurs applications détaillées sont présentées dans les sections suivantes.

5.3 *Cadre de la modélisation probabiliste de la maintenance*

Cette section présente le cadre de la modélisation probabiliste de la maintenance. La figure 7 illustre les approches proposées à l'aide d'un organigramme.

L'organigramme est composé de deux boucles principales. La boucle externe illustre la stratégie probabiliste consistant à utiliser le LHS pour générer le nombre requis de variables aléatoires, ou à construire un modèle de substitution basé sur l'ECP à partir d'un nombre limité de simulations, tandis que la boucle interne représente le code numérique de FreeFEM++ utilisé pour modéliser le transport du chlorure dans le béton existant et le béton réparé. Dans la boucle interne, le "critère de réparation" est défini sur la base de deux approches principales. La première approche consiste à effectuer la réparation lorsque la teneur en chlorure au niveau de la couverture des barres d'armature atteint une valeur critique spécifique. La seconde approche considère l'application d'intervalles de réparation préprogrammés pour les structures comme la condition pour effectuer une réparation. Ces approches nous permettent de calculer la probabilité d'initiation de la corrosion, qui est l'un des principaux critères d'évaluation de la fiabilité des structures en béton armé corrodées. L'organigramme de calcul de la pénétration des chlorures dans le béton après réparation est également présenté, comme déjà mentionné dans la section 4. Les résultats des deux approches de réparation pourraient être les temps d'initiation de la corrosion ou les profils de chlorures à un instant donné de la réparation. Sur cette base, le nombre prévu de réparations, n , est déterminé.

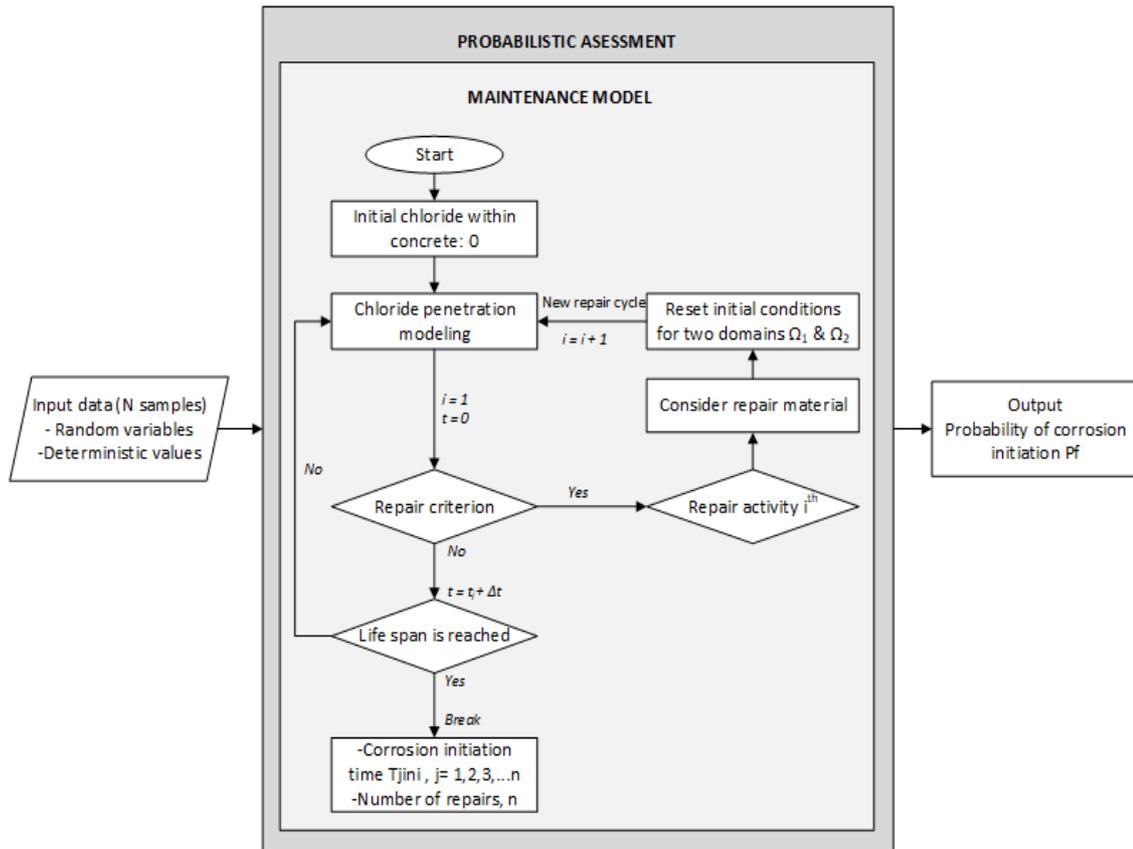


Figure 7. Organigramme de la modélisation probabiliste de la maintenance

6. Réparation et gestion durable de l'entretien des structures en béton armé polluées par les chlorures

L'objectif principal de cette section est de proposer un cadre pour la réparation et la gestion de l'entretien durable des structures en béton armé polluées par les chlorures. En raison des différences dans la forme et l'épaisseur du béton de remplacement, ou dans l'utilisation des matériaux de réparation, chaque stratégie a une performance de durabilité à long terme et un profil d'extension de la durée de vie différents. Le calendrier d'entretien de chaque stratégie de réparation est établi sur la base du critère de réparation, tel que le seuil de réparation ou l'intervalle de réparation. Dans une optique de durabilité, des normes strictes doivent s'appliquer à la réparation et à l'entretien des structures en béton, tant en termes d'impact environnemental que de contraintes financières (Bastidas-Arteaga et Schoefs, 2015). L'approche proposée pour la comparaison entre différentes solutions de réparation incluant des critères de coût et d'environnement est présentée dans la figure 8. Une comparaison de trois facteurs pourrait être utilisée pour évaluer la faisabilité à long terme des méthodes de réparation : le coût, la production de déchets et les émissions de CO₂. Enfin, l'utilisation d'une approche d'optimisation multi-objectifs est adoptée pour comparer et sélectionner une solution de maintenance durable.

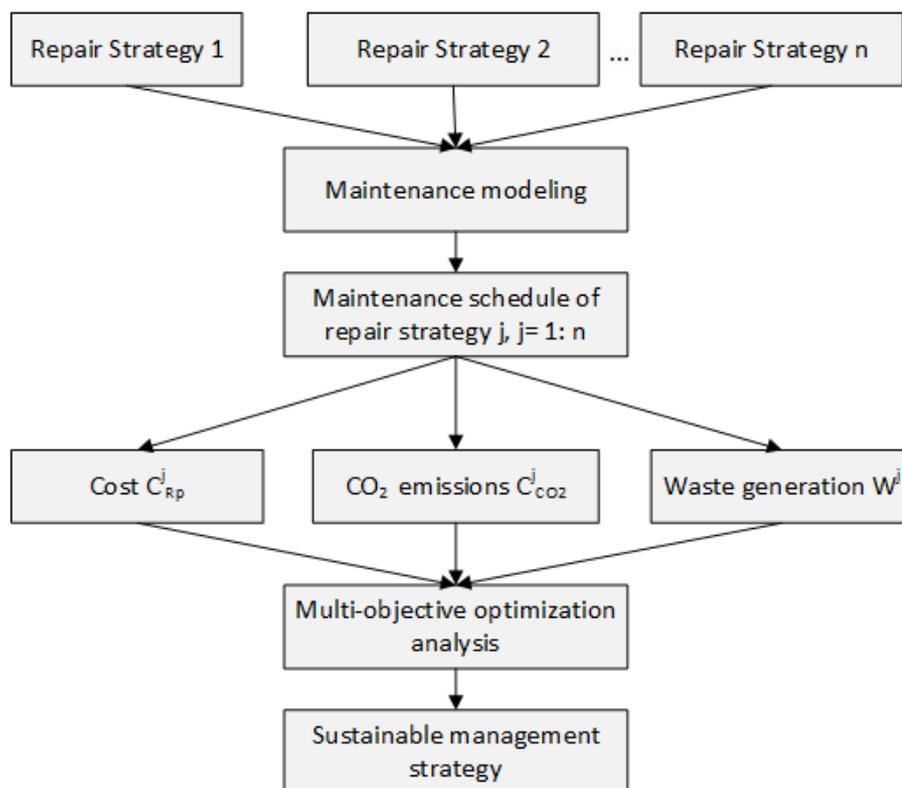


Figure 8 : Cadre de la gestion durable de la maintenance

6.1 Coûts

Cette étude s'appuie principalement sur des données publiques relatives aux coûts directs pour comparer les coûts des différentes stratégies de réparation des structures en béton armé en cours de détérioration. Les coûts directs de l'agence sont limités à ceux associés à la maintenance (inspection et réparation). L'analyse ne prend pas en compte les coûts initiaux de construction et les coûts résiduels car l'étude suppose que les réparations sont effectuées sur des éléments en béton armé identiques.

Pour la méthode de remplacement de la couverture, selon Bastidas-Arteaga et Stewart (2014), le coût de remplacement de la profondeur de la couverture en béton, comprenant l'inspection, la démolition, la réparation et la perte d'exploitation, est de $C_{\text{cover}} = 323 \text{ €/m}^2$ (coûts de réparation estimée pour le port de Nantes-Saint-Nazaire en France). A l'exclusion du remplacement des barres corrodées, la réparation comprend l'enlèvement et la reconstruction de la couverture de béton polluée par les chlorures entre 4cm et 8cm pour les dalles et les poutres. Le coût de réparation supplémentaire par mm de profondeur de réparation supplémentaire derrière la couverture est indiqué dans le tableau 2 (Bastidas-Arteaga et Stewart, 2014). Comme le montre le tableau 2, le coût de réparation est plus élevé pour un poteau carré car il prend en compte les dommages survenant sur les quatre côtés du poteau.

Tableau 2 : Coûts de réparation supplémentaires pour divers éléments structurels

Structural element	Dimensions (mm)	C _{extra} (€/mm)
Dalles	100	0.85
Dalles	300	0.49
Poutres	200 to 800	0.98
Colonnes carrées	300	1.56
Colonnes carrées	400	1.38
Colonnes carrées	600	1.01

Par conséquent, le coût de construction d'une activité de remplacement du béton, C_{RP}, est défini comme suit :

$$C_{RP} = [C_{cover} + \text{Depth over cover (mm)} * C_{extra}] S_{RP} \quad (17)$$

où S_{RP} (m²) est la surface de la structure à réparer, C_{cover}(€) sont les coûts de remplacement de l'ensemble de la couverture en béton, Depth over cover (mm) est la profondeur de réparation supplémentaire derrière la couverture. Il convient de noter que les informations sur les coûts utilisées dans cette étude proviennent de la construction d'infrastructures côtières françaises. Les résultats escomptés ne peuvent donc pas être comparés de manière adéquate à d'autres situations de réparation, car les coûts de réparation peuvent varier considérablement en fonction des conditions de l'installation et de la situation géographique. Néanmoins, il est possible, dans le cadre de cette étude, d'effectuer une évaluation du rapport coût-efficacité pour différentes stratégies de réparation.

6.2 Émissions de CO₂

Les impacts environnementaux de la réparation du béton sont classés en plusieurs catégories (Årskog, Fossdal et Gjörv, 2013). Dans cette étude, la catégorie de l'impact sur le réchauffement climatique est examinée en relation avec les émissions de CO₂ provenant du remplacement du béton pollué par les chlorures. Dans cette technique de réparation, le béton pollué par les chlorures est enlevé et la structure est réhabilitée par la mise en place d'un nouveau béton ou d'un mortier de réparation sans endommager les barres d'armature. Le nouveau matériau est généralement mis en place sous forme de béton projeté humide, de béton projeté sec, de béton formé ou par réparation manuelle (BSEN 1504-10:2017). Par conséquent, les six processus qui affectent la quantité de CO₂ libérée sont pris en compte, notamment la fabrication des matériaux de réparation, le transport, le décapage à l'eau pour enlever la couverture de béton, le nettoyage de l'armature, l'enrobage protecteur de l'armature et la mise en place du béton projeté (Petcherdchoo, 2015 ; Lifecon Deliverable D 5.3). Les émissions de CO₂ estimées par mesure de réparation de l'enrobage sont présentées dans le tableau 3.

Il convient de noter que les unités fonctionnelles de CO₂ ne sont pas les mêmes pour tous les processus (comme indiqué dans le tableau 3). Par conséquent, l'émission de CO₂ sur l'ensemble du cycle de vie (kg CO₂) pour chaque stratégie de réparation peut être calculée comme suit :

$$C_{CO_2} = (405 V_{RP} + 121.8 S_{RP})n_{RP} \quad (18)$$

où V_{RP} est le volume de béton réparé (m³), S_{RP} est la surface de la structure à réparer (m²), et n_{RP} est le nombre de réparations

Tableau 3 : Émissions de CO₂ par mesure de réparation de la couverture

Processus	CO ₂ (kg CO ₂ eq/m ³)	CO ₂ (kg CO ₂ eq/m ²)
Production de béton	405	-
Hydrojet	-	84
Nettoyage de l'armature	-	22
Enduit protecteur sur l'armature	-	1.4
Application de béton projeté	-	4.4
Transport	-	10
Total	405	121.8

6.3 Production de déchets

Les déchets de démolition en béton représentent 67% des déchets de construction mondiaux en poids (53% en volume) et seulement 5% sont généralement recyclés. La production de déchets (déchets de béton) doit être considérée comme un critère d'évaluation durable des activités de réparation et d'entretien (Bastidas-Arteaga et Schoefs, 2015).

Une évaluation complète prendrait en compte les déchets générés pendant les activités de réparation et d'entretien proprement dites, mais aussi les déchets générés pendant le processus de production des matériaux de réparation. Cependant, comme il est difficile de mesurer les déchets générés pendant la production de béton, cette étude se concentre uniquement sur les déchets générés pendant les travaux de réparation. Les déchets générés par le remplacement d'un couvercle de 1m³ de béton ont été déterminés comme étant 1,3 m³ de déchets (Bastidas-Arteaga, 2010). Par conséquent, la production de déchets, W (m³), pendant tout le cycle de vie pour chaque stratégie de réparation, peut être calculée à partir des volumes de réparation calculés dans l'analyse des réparations de cette étude comme suit :

$$W = 1.3 V_{RP} n_{RP} \quad (19)$$

où V_{RP} est le volume de béton enlevé par application de remplacement de couverture (m³), n_{RP} est le nombre de réparations de chaque stratégie de réparation.

6.4 Méthodes de gestion durable de la maintenance

La complexité de la gestion multicentrique des structures en cours de détérioration pose des problèmes aux agences chargées de l'exploitation des installations en béton. Les agences ont des objectifs contradictoires : réduire les coûts, les impacts environnementaux et les perturbations du trafic tout en améliorant l'aptitude au service, la fonctionnalité et la sécurité. Une approche d'optimisation multicritères ou multi-objectifs peut aider à résoudre ces conflits. Il existe plusieurs méthodes d'optimisation multicritères :

- L'approche de la somme pondérée
- La méthode d'optimisation de Pareto
- La programmation des objectifs
- La programmation de compromis
- La programmation par contraintes

Le choix de la méthode dépend du problème spécifique et des objectifs de l'optimisation. Dans le contexte de la recherche d'une méthode appropriée pour un problème de finance/ingénierie, l'avantage de la programmation par compromis est qu'elle permet au décideur de contrôler explicitement les compromis entre les objectifs, ce qui peut être utile dans les situations où l'importance relative des objectifs varie en fonction du contexte. C'est pourquoi cette étude considère l'approche de la programmation par compromis (ou méthode VIKOR) pour la gestion de la maintenance durable (Opricovic, 1998).

La méthode VIKOR est une technique de prise de décision multicritère utilisée pour évaluer des alternatives sur la base de critères multiples. Cette méthode est largement utilisée dans divers domaines, notamment l'ingénierie, la gestion, l'économie et les sciences de l'environnement. L'approche VIKOR permet de sélectionner une solution de compromis avec des critères d'évaluation concurrents. Le classement est basé sur la similarité avec une alternative idéale, où un compromis est défini comme un accord obtenu par des concessions mutuelles, en évaluant chaque alternative selon chaque fonction de critère (Opricovic, 2011 ; San Cristóbal Mateo, 2012). La métrique L_p , utilisée comme fonction d'agrégation dans une approche de programmation de compromis, est la base du classement de compromis multicritères :

$$L_{p,j} = \left\{ \sum_{i=1}^n \left[\frac{w_i (f_i^* - f_{ij})}{(f_i^* - f_i^-)} \right]^p \right\}^{\frac{1}{p}}, \quad 1 < p < \infty \quad (20)$$

où f_{ij} est la valeur de la i^{th} fonction critère pour l'alternative j ; et f_i^* et f_i^- sont respectivement les meilleures et les pires valeurs de l'alternative j . n est le nombre de critères, dans cette stratégie il y a trois critères d'évaluation : (1) les coûts, (2) les émissions de CO_2 et (3) la production de déchets. w_i est le facteur de pondération de la i^{th} fonction de critère, la valeur de w_i varie entre 0 et 1 avec $\sum w_i = 1$. Les facteurs de pondération dépendent de l'attitude de l'opérateur à l'égard de chaque critère.

$L_{1,j}$ (la valeur de S_i) et $L_{\infty,j}$ (la valeur de R_i) sont utilisés pour formuler une mesure de classement. S_j représente l'utilité maximale des groupes alternatifs, tandis que R_j représente l'alternative individuelle la moins efficace. En outre, l'indicateur global Q_j est utilisé pour classer les alternatives, Q_j étant la moyenne pondérée et normalisée des valeurs S_j et R_j dans l'intervalle $[0;1]$. La méthode VIKOR est utilisée pour calculer les points positifs et négatifs de la solution et enfin les alternatives sont classées en triant les valeurs R , S et Q par ordre décroissant. La solution ayant la valeur Q la plus faible est la meilleure. Les étapes de mise en œuvre de la méthode VIKOR et les conditions pour parvenir à une solution de compromis sont bien décrites dans des études antérieures (San Cristóbal Mateo, 2012 ; Mardani et al., 2016 ; Mateusz et al., 2018) et seront présentées dans l'annexe B.

7. Principaux résultats et discussions

7.1 *Modélisation de la pénétration des chlorures dans le béton réparé*

La modélisation de la pénétration des chlorures dans le béton après réparation est un défi en raison de l'interaction des deux principales sources d'ions chlorure : la concentration ambiante de chlorure et les ions chlorure résiduels dans le béton restant. Dans cette partie, un modèle de maintenance est développé pour simuler différentes stratégies de reconstruction de la couverture. Une solution numérique implémentée dans FreeFem++ utilisant une technique de décomposition de domaine est présentée. Enfin, plusieurs exemples déterministes sont utilisés pour évaluer l'influence de certains paramètres principaux sur l'efficacité de la réparation et l'influence des conditions météorologiques locales sur le processus de pénétration des chlorures. Les principaux résultats de cette partie sont les suivants :

1. Une analyse déterministe d'un modèle de maintenance 1D qui prédit la durée de vie des structures en béton polluées par les chlorures avec des réparations multiples est effectuée. Trois facteurs affectant l'efficacité de la méthode de réparation sont pris en compte : (1) le nombre de réparations prévues, (2) la profondeur du béton de réparation et (3) l'intervalle de réparation. Il s'avère qu'une plus grande profondeur de réparation entraîne un plus petit nombre de réparations, tandis qu'un intervalle de réparation plus court réduit la probabilité d'initiation de la corrosion mais augmente les coûts d'entretien. Par conséquent, une politique d'entretien globale doit tenir compte de ces principaux paramètres pour trouver une stratégie optimale qui minimise à la fois les coûts des matériaux et l'impact sur l'environnement et qui prévient l'apparition de la corrosion. D'autres facteurs de contrôle, tels que les coûts de réparation, doivent également être pris en compte car ils jouent un rôle important pour les décideurs lorsqu'ils comparent différentes stratégies de réparation.
2. Pour le modèle de maintenance 2D, les résultats montrent que pour les structures en béton armé soumises à une attaque bidimensionnelle de chlorure, les coins d'une

colonne sont les zones les plus critiques pour évaluer la concentration de chlorure, prédire le moment de l'apparition de la corrosion et estimer les temps de réparation au cours de la durée de vie de la conception. En outre, la stratégie de remplacement circulaire du béton est recommandée par rapport au remplacement orthogonal du béton pour la réparation des poteaux en béton armé parce que le remplacement d'une plus grande quantité de béton d'angle qui suit une forme incurvée peut aider à réduire la concentration de chlorure qui s'accumule dans la région de l'angle au fil du temps, ce qui prolonge le délai d'apparition de la corrosion.

3. Enfin, il est démontré que le processus de corrosion induit par les chlorures est fortement influencé par les conditions météorologiques locales. En particulier, les températures et l'humidité élevées dans de nombreuses villes du Vietnam peuvent accélérer la pénétration du chlorure dans le béton et raccourcir la durée de vie des structures en béton armé avant et après les réparations. Par conséquent, des politiques d'entretien appropriées et des stratégies de réparation comprenant des méthodes supplémentaires de protection contre la corrosion doivent être bien planifiées et mises en œuvre pour assurer la durabilité des structures en béton armé corrodées, tout en respectant les normes actuelles en matière de qualité du béton et d'épaisseur de l'enrobage.

7.2 Modélisation et planification de la maintenance basée sur les probabilités pour les structures en béton contaminées par les chlorures

Cette partie réalise des évaluations probabilistes et des analyses de sensibilité pour la modélisation de la maintenance, l'estimation de la probabilité d'initiation de la corrosion et l'optimisation des plans de maintenance. Compte tenu de la complexité du système, la méthode LHS est d'abord utilisée pour l'analyse de fiabilité du modèle de maintenance 1D. Ensuite, une méthode probabiliste basée sur des métamodèles est proposée pour réduire le temps de calcul, utilisant la méthode PCE pour approximer le modèle de pénétration du chlorure en deux dimensions (2D) dans le béton réparé. Les principales conclusions de cette partie sont présentées comme suit :

1. Une évaluation probabiliste complète de la durée de vie des structures en béton armé réparées soumises à une pénétration de chlorure unidimensionnelle utilisant la technique LHS est réalisée. L'évaluation probabiliste porte à la fois sur des stratégies de réparation correctives et préventives. Pour la stratégie de réparation corrective, la réparation est effectuée lorsque la teneur en chlorure au niveau de la couverture des barres d'armature atteint une valeur critique spécifique de chlorure, et le nombre total attendu de réparations peut être utilisé comme facteur pour comparer les différentes stratégies de réparation. On observe qu'à l'exception de la première période de réparation plus longue, la variation des comportements de pénétration du chlorure dans les

structures en béton armé n'est pas significative entre les périodes de réparation. En cas de contraintes de temps et de coûts de calcul élevé, la simulation de la prédiction de la durée de vie pour quelques étapes de réparation initiales est suffisante en tant que résultat de base pour formuler le programme de maintenance par l'approche préventive. Pour le modèle de réparation préventive, cette approche de maintenance semble être efficace pour la prédiction de la durée de vie et la planification de la maintenance, car la probabilité acceptable d'initiation de la corrosion peut être atteinte en adoptant certaines stratégies améliorées. Il est démontré que l'application d'intervalles de réparation plus courts pourrait contribuer à la diminution du risque de corrosion pour la première réparation, tandis que la sélection de la profondeur de réparation pourrait principalement influencer la probabilité d'initiation de la corrosion après la première réparation.

2. La méthode PCE est ensuite utilisée pour propager les incertitudes dans la modélisation de la maintenance en 2D et optimiser les calendriers de maintenance. L'évaluation probabiliste des modèles de maintenance proactive basée sur la méthode PCE s'avère efficace et précise avec des modèles d'évaluation peu coûteux. Grâce à elle, plusieurs réparations proactives sont évaluées pour obtenir un plan de maintenance optimal qui répond à l'exigence d'une probabilité acceptable d'initiation de la corrosion.
3. En ce qui concerne les résultats de l'analyse de sensibilité, les variables aléatoires influentes sont X_c (représentant l'épaisseur de la couverture et la quantité de béton remplacée), la concentration de chlorure en surface C_{fc}^S , le coefficient de diffusion du chlorure $D_{c,ref}$ et le facteur de vieillissement m_c , où X_c est le paramètre le plus important qui devrait être contrôlé dans le processus de construction/réparation.
4. En outre, les résultats montrent que l'utilisation d'un matériau de réparation de meilleure qualité contribue à réduire le risque de corrosion pendant les cycles de réparation, ce qui se traduit par un délai plus long pour l'initiation de la corrosion et donc par un nombre plus faible de réparations.

7.3 *Sélection d'une stratégie de réparation rentable et durable pour les structures en béton contaminées par les chlorures*

1. Cette partie fournit des exemples pour illustrer la méthodologie proposée pour la stratégie de gestion rentable et la stratégie de gestion durable. Dans le premier exemple, l'efficacité de différentes stratégies de réparation peut être comparée en termes de forme et d'épaisseur du béton de remplacement, en tenant compte de trois facteurs, à savoir le coût, la production de déchets et les émissions de CO₂. Les résultats de l'analyse d'optimisation multicritère utilisant la méthode VIKOR montrent que la stratégie de remplacement circulaire du béton est l'option de réparation la plus économique et également la solution optimale pour une gestion durable de la maintenance.
2. Dans le deuxième exemple, deux stratégies de réparation utilisant différents matériaux de réparation sont considérées. Pour une gestion durable de la maintenance, la stratégie

de réparation S2, qui utilise un matériau de réparation de meilleure qualité, s'avère plus efficace d'un point de vue économique et en termes d'impact sur l'environnement. Cependant, de futures études expérimentales sont nécessaires pour étudier la compatibilité mécanique à long terme entre la rustine et le béton d'origine avec des propriétés différentes.

8. Conclusions et perspectives

La corrosion des structures en béton armé est un problème très répandu qui peut compromettre leur sécurité et leur durabilité. Il est donc crucial d'avoir une gestion efficace des réparations et de la maintenance pour assurer leur performance et leur sécurité à long terme. Cette étude a développé des modèles probabilistes et optimisé les stratégies de réparation pour les structures en béton armé exposées aux infiltrations de chlorure, dans le but de mettre en place des pratiques de maintenance durables. Pour ce faire, un modèle de maintenance complet a été développé pour simuler les infiltrations de chlorure lorsque la structure est réparée par la reconstruction de la couverture en béton. Le modèle prend en compte des facteurs tels que l'épaisseur et les intervalles de réparation, la qualité du béton et les conditions environnementales. Il a permis d'évaluer l'efficacité des stratégies de réparation afin d'améliorer les opérations de maintenance.

Une approche probabiliste, basée sur l'échantillonnage hypercube latin et les expansions du chaos polynomial, a été appliquée pour tenir compte de l'incertitude, prédire le risque de corrosion et optimiser le calendrier d'entretien. En outre, un cadre décisionnel prenant en compte les coûts, l'impact environnemental et les indicateurs de sécurité a été utilisé pour sélectionner une stratégie de réparation durable. Tout au long du manuscrit, plusieurs exemples numériques et études de cas ont illustré les points ci-dessus. Dans l'ensemble, les modèles et les cadres proposés peuvent aider les ingénieurs et les praticiens à développer des stratégies de maintenance et de réparation efficaces et durables afin d'améliorer la sécurité et la longévité des structures en béton armé corrodées. Les perspectives en recherche sont donc classées par rapport au type de modèle à explorer.

Modèle de transport de chlorure dans le béton réparé (modèle de réparation)

- Développer un modèle complet prenant en compte des conditions d'exposition plus réalistes, le système de couplage diffusion-convection du chlorure dans le béton, le transfert de chaleur et d'humidité.
- Étudier l'effet des variations saisonnières de température et d'humidité sur la pénétration du chlorure dans le béton réparé .
- Collecter des données pour les paramètres du modèle (liés à la diffusivité du chlorure dans le béton et aux critères d'évaluation) et examiner leurs corrélations à partir d'études expérimentales ;

- Examiner les performances des stratégies d'adaptation des réparations en tenant compte du changement climatique.
- Développer et appliquer un modèle de réparation qui prend en compte la relation entre la fissuration du béton et la pénétration des chlorures.

Stratégies de maintenance

- Étudier et modéliser les effets d'autres techniques de réparation, telles que les revêtements, les matériaux d'étanchéité et les inhibiteurs de corrosion.
- Appliquer l'évaluation probabiliste des coûts et des bénéfices pour déterminer le calendrier d'entretien optimal.
- Mettre en œuvre une autre méthode de fiabilité basée sur les métamodèles, telle que les réseaux neuronaux artificiels pour l'évaluation probabiliste des modèles de maintenance corrective et préventive.
- Explorer les méthodes bayésiennes dans la planification et l'optimisation de la maintenance.
- Intégrer des résultats d'inspection dans le cadre décisionnel pour l'optimisation de la maintenance des structures existantes.
- Prendre en compte les incertitudes lors de la formulation de stratégies d'entretien durables.

Titre : Modélisation probabiliste et optimisation des stratégies de réparation pour l'entretien durable des structures en béton armé soumises à la pénétration des ions chlorure

Mots clés : pénétration des chlorures, modèle d'entretien, béton armé, corrosion, reconstruction de la couverture, durabilité

Résumé : La corrosion des structures en béton armé est un problème répandu qui peut compromettre leur sécurité et leur durabilité. Il est donc crucial d'avoir une gestion efficace de la réparation et de la maintenance pour assurer leur performance et leur sécurité à long terme. Cette thèse vise à développer des modèles probabilistes et à optimiser les stratégies de réparation pour les structures en béton armé exposées à la pénétration des chlorures, dans le but d'atteindre des pratiques de maintenance durables. Pour y parvenir, un modèle de maintenance a été développé pour simuler les processus de transfert de chlorures lorsque la structure est réparée avec la reconstruction de la couverture. Le modèle prend en compte des facteurs tels que l'épaisseur et les intervalles de réparation, la qualité du béton et les conditions environnementales. Cela permet d'évaluer l'efficacité des stratégies de réparation pour améliorer les opérations de maintenance.

Une approche probabiliste, basée sur l'échantillonnage Latin Hypercube et le chaos polynomial, a été proposée pour tenir compte de l'incertitude, prédire le risque de corrosion et optimiser les actions de maintenance. De plus, un cadre décisionnel qui prend en compte des indicateurs de coûts, d'impact environnemental et de sécurité a été mis en œuvre pour sélectionner une stratégie de réparation durable. Tout au long du manuscrit, plusieurs exemples numériques et études de cas illustrent les points susmentionnés. Dans l'ensemble, les modèles et cadres proposés peuvent aider les ingénieurs et les praticiens à développer des stratégies de maintenance et de réparation efficaces et durables pour améliorer la sécurité et la durée de vie des structures en béton armé corrodées.

Title : Probabilistic modeling and optimization of repair strategies for sustainable maintenance of reinforced concrete structures subjected to chloride ingress

Keywords: chloride penetration, maintenance model, reinforced concrete, corrosion, cover rebuilding, sustainability

Abstract: The corrosion of reinforced concrete structures is a widespread problem that can compromise their safety and durability. Therefore, it is crucial to have effective repair and maintenance management to ensure their long-term performance and safety. This thesis aims to develop probabilistic models and optimize repair strategies for reinforced concrete structures exposed to chloride ingress, with the goal of achieving sustainable maintenance practices. To achieve this, a comprehensive maintenance model is developed to simulate chloride ingress when the structure is repaired by concrete cover rebuilding. The model considers factors such as repair thickness and intervals, concrete quality, and environmental conditions. This enables the evaluation of the efficiency of repair strategies to improve maintenance operations.

A probabilistic approach, based on Latin Hypercube Sampling and Polynomial Chaos Expansions, is proposed to account for uncertainty, predict the risk of corrosion, and optimize the maintenance schedule. Furthermore, a decision framework that considers costs, environmental impact, and safety indicators is introduced to select a sustainable repair strategy. Throughout the manuscript, several numerical examples and case studies illustrate the above points. Overall, the proposed models and frameworks can assist engineers and practitioners in developing effective and sustainable maintenance and repair strategies to improve the safety and longevity of corroding reinforced concrete structures.