Universidade de Aveiro Departamento de Engenharia Civil
Ano 2018

PABLO DANIEL BENÍTEZ MONGELÓS

Estratégias de apoio à manutenção de estruturas de betão armado com risco de corrosão

Maintenance support strategies for reinforced concrete structures under corrosion risk

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Tese apresentada à Universidade de Aveiro para cumprimento dos requisitos necessários à obtenção do grau de Doutor em Engenharia Civil, realizada sob a orientação científica da Doutora Maria Fernanda da Silva Rodrigues, Professora Auxiliar do Departamento de Engenheira Civil da Universidade de Aveiro, e coorientação científica do Doutor Humberto Salazar Amorim Varum, Professor Catedrático da Faculdade de Engenharia da Universidade do Porto.

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Maintenance support strategies for reinforced concrete structures under corrosion risk

This thesis is submitted to the University of Aveiro to fulfil the necessary requirements for the degree of Doctor of Philosophy in Civil Engineering, performed under the scientific supervision of Maria Fernanda Rodrigues, Assistant Professor of the Department of Civil Engineering at the University of Aveiro, and the co-supervision of Humberto Salazar Amorim Varum, Full Professor of the Department of Civil Engineering at the University of Porto.

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palavras-chave

degradação do betão, alterações climáticas, carbonatação, vida útil, análise de eficiência, dados reais de carbonatação, otimização de manutenção, tomada de decisão, manutenção preventiva.

resumo

As estruturas de betão armado (BA) constituem uma grande parte das estruturas e infraestruturas construídas em todo o mundo. A sua maioria foi construída na primeira metade do século passado, logo, a sua vida útil apresenta atualmente, em muitos casos, um estado crítico, na perspetiva da sua manutenção. Um dos mecanismos de degradação mais frequentes e dispendiosos neste tipo de estruturas está associado à corrosão das armaduras. Esta tese analisa a corrosão induzida por carbonatação, considerando que é a principal causa de degradação em estruturas de betão no Paraguai.

O problema crescente advindo das alterações climáticas alertou os responsáveis da manutenção deste tipo de estruturas, desde o final do século passado. Vários estudos mostraram que podem ser afetadas pelo impacto desse fenômeno no que respeita à sua durabilidade, salientando a redução na expectativa de vida útil dessas estruturas, causada por um aumento da taxa de corrosão, associada ao aumento da temperatura e às concentrações atmosféricas de CO_2. Estas conclusões levaram ao desenvolvimento desta tese, cujo objetivo principal é desenvolver uma metodologia otimizada para a formulação de estratégias de manutenção de estruturas de BA submetidas à degradação por corrosão induzida por carbonatação, considerando os efeitos das alterações climáticas.

Para o cumprimento dos objetivos desta tese, efetuou-se uma análise de modelos numéricos de carbonatação em estruturas de betão armado, para a obtenção das curvas de degradação. As curvas de degradação obtidas com o modelo matemático escolhido e modificado, mostram a profundidade de carbonação esperada no Paraguai para os próximos 50 anos, considerando diferentes cenários climáticos. Por sua vez, com esta análise, foi possível determinar os tempos de início e de propagação da corrosão previstos para as estruturas, considerando diversas configurações dos principais parâmetros que influenciam a durabilidade: a qualidade do betão e a espessura do recobrimento.

Definidas as condições de degradação foram formuladas estratégias de manutenção preventiva baseadas em modelos de decisão. Estes modelos numéricos foram estabelecidos em duas etapas que compreendem o planeamento das inspeções e o planeamento das reparações. Para a primeira etapa foi proposta uma análise de eficiência, complementada com o processo de otimização dos momentos de inspeção e das técnicas de intervenção mais apropriadas. Para o planeamento da reparação, foi desenvolvido um modelo dinâmico para apoio à tomada de decisões que considera a análise de custos da estratégia de manutenção e a capacidade das técnicas de inspeção e reparação para garantir a durabilidade das estruturas, de forma eficaz, através de manutenção preventiva.

Os resultados desta tese mostram que o risco de degradação de estruturas de betão armado induzidas por carbonatação, pode aumentar no futuro devido ao efeito das alterações climáticas. Assim, para o pior cenário climático, estima-se um aumento médio de 25% na profundidade máxima de carbonação na segunda metade deste século em relação a um cenário de controle. Enquanto que o tempo para atingir a mesma profundidade máxima de carbonatação do cenário de controle pode ser reduzido entre 7 e 10 anos para o melhor cenário, dependendo da qualidade do betão. Além disso, o modelo de manutenção desenvolvido é facilmente aplicável e permite a formulação de estratégias de longo prazo que otimizem recursos ao menor custo, para lidar com esse mecanismo de degradação.

keywords concrete degradation, climate change, carbonation, service life, efficiency analysis, real carbonation data, maintenance optimisation, decision-making, preventive maintenance.

abstract Reinforced concrete (RC) structures comprise a large part of the structures and infrastructures around the world. Most of them have been built in the first half of the 20th-century, so their service life is currently at a critical stage from the maintenance perspective. One of the most frequent and expensive degradation mechanisms they present is associated with the reinforcement corrosion. This research is focused on the carbonation-induced corrosion considering that it is the primary degradation cause of RC structures in Paraguay.

The growing problem of climate change has caught the attention of maintenance managers of these structures since the end of the last century. Several studies shown that structures can be affected by the impact of this phenomenon considering its durability highlighting the reduction of its expected service life caused by an increase in the corrosion rate associated with the rise in temperature and the atmospheric concentrations of CO_2. These conclusions led to the development of this thesis whose main objective is to develop an optimised methodology for the formulation of maintenance strategies of RC structures subjected to the carbonation-induced corrosion, considering the effects of climate change.

For the fulfilment of the objectives of this thesis, an analysis has been developed on the numerical modelling of carbonation in RC structures to obtain the degradation curves. The degradation curves obtained by the chosen and modified model show the expected carbonation depth in Paraguay for the next 50 years under the consideration of different climate scenarios. With this analysis, it was possible to determine the corrosion initiation and corrosion propagation times for the structures considering several configurations of the principal parameters influencing durability: the quality of the concrete and the cover thickness.

Once the degradation conditions were defined, preventive maintenance strategies based on decision models were formulated. These numerical models were established in two stages comprising the inspections and the repairs planning. For the inspections planning, an efficiency analysis was proposed that is complemented by the optimisation process of the inspection schedule and the most appropriate intervention techniques. For the repairs planning, a dynamic model for decision-making has been developed which considers the cost analysis of the maintenance strategy and the capabilities of the inspection and repair techniques to effectively ensure the durability of the structures through the preventive maintenance approach.

The main results of this thesis shown that the carbonation-induced corrosion risk of structures can increase in the future due to the climate change effect. Thus, for the worst climate scenario, in the second half of this century is expected an increase by 25% in the maximum carbonation depth regarding a control scenario. Meanwhile, the time to reach the same maximum carbonation depth of the control scenario can be reduced between 7 and 10 years for the best climate scenario, depending on the quality of the concrete. Furthermore, the maintenance model developed in this research is easily applicable and allows the formulation of long-term strategies that optimise resources at the lowest cost to deal with this degradation mechanism.

palabras claves degradación del concreto, cambio climatico, carbonatación, vida útil, análisis de eficiencia, datos reales de carbonatacion, optimización del mantenimiento, toma de decisiones, mantenimiento preventivo.

resumen

Las estructuras de hormigón armado (H°A°) comprenden una gran parte de las infraestructuras en todo el mundo. La mayoría de estas estructuras se han construido en la primera mitad del Siglo XX, por lo que su vida útil se encuentra actualmente en una etapa crítica desde la perspectiva del mantenimiento. Uno de los mecanismos de degradación más frecuentes y costosos en este tipo de infraestructura está asociado con la corrosión del refuerzo. Esta investigación se centra en la corrosión inducida por la carbonatación, ya que es la principal causa de degradación de las estructuras de H°A° en Paraguay.

El creciente problema del cambio climático ha llamado la atención de los administradores del mantenimiento de las estructuras desde finales del siglo pasado. Varios estudios han demostrado que las infraestructuras podrían verse afectadas por el impacto de este fenómeno desde el punto de vista de la durabilidad. Estos estudios han demostrado una reducción en la vida útil prevista de las estructuras causada por un aumento en la tasa de corrosión asociada con el aumento de temperatura y las concentraciones atmosféricas de CO_2. Esta condición ha llevado a la elaboración de esta tesis cuyo principal objetivo es desarrollar una metodología optimizada para la formulación de estrategias correspondientes al mantenimiento de estructuras H°A° sometidas a la corrosión inducida por la carbonatación, considerando los efectos del cambio climático.

Para el cumplimiento de los objetivos de esta tesis, se ha desarrollado un análisis sobre la modelación numérica de la carbonatación en el hormigón para obtener las curvas de degradación en estas estructuras. Estas curvas muestran la profundidad de carbonatación esperada en Paraguay durante los próximos 50 años bajo la consideración de diferentes escenarios climáticos. A su vez, con este análisis se ha podido determinar el tiempo de inicio y propagación de la corrosión previstos para las estructuras considerando varias configuraciones de los parámetros principales para la durabilidad: la calidad del hormigón y el espesor del recubrimiento.

Una vez que se han definido las condiciones de degradación, se han formulado estrategias de mantenimiento preventivo basadas en modelos de decisión. Estos modelos numéricos se han establecido en dos etapas que comprenden la planificación de inspecciones y la planificación de reparaciones. Para la planificación de las inspecciones, se ha propuesto un análisis de eficiencia que se complementa con el proceso de optimización de los tiempos de inspección y las técnicas de intervención más adecuadas. Para la planificación de reparaciones, se ha desarrollado un modelo dinámico para la toma de decisiones que considera el análisis de costos de la estrategia de mantenimiento y las capacidades de las técnicas de inspección y reparación para garantizar la durabilidad de las infraestructuras de manera efectiva a través del enfoque de mantenimiento preventivo.

Los resultados de esta tesis han demostrado que el riesgo de corrosión inducida por la carbonatación de las estructuras podría aumentar en el futuro debido al efecto del cambio climático. Así, para el peor escenario climático, podría esperarse un aumento medio de 25% de la profundidad máxima de carbonatación en la segunda mitad de este siglo respecto a un escenario de control. Mientras tanto, el tiempo para alcanzar la misma profundidad máxima de carbonatación del escenario de control podría reducirse entre 7 y 10 años para el mejor escenario climático, dependiendo de la calidad del hormigón. Además, el modelo de mantenimiento desarrollado en esta investigación es fácilmente aplicable y permite la formulación de estrategias a largo plazo que optimizan los recursos al menor costo para hacer frente a este mecanismo de degradación.

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SYMBOLOGY

A	Vector for the set of alternatives in the decision problem
A_D	Empirical parameter for effective diffusion of CO_2
a_{ij}	Individual weights of each alternatives in the comparison matrix
b	Cover thickness of concrete
c	Cement content
$Ca(OH)_2$	Calcium Hydroxide
$CaCO_3$	Calcium carbonate
CI	Consistency Index
CO_2	Carbon Dioxide
CR	Consistency Ratio
$C\text{-}S\text{-}H$	Calcium-Silicate-Hydrate
$\hat{C}$	Consistency factor for the proposed comparison matrix
C_0	Initial construction cost
C_{NPV}^{γ}	Present cost for applying the repair method in the future
C_{NPV}^{θ}	Present cost for applying the inspection technique in the future
$CO_{2(aq)}$	Aqueous carbon dioxide solution
$CO_{2(g)}$	Atmospheric concentration of CO_2
$Ca(OH)_{2(aq)}$	Aqueous concentration of calcium hydroxide
C_{air}	Air content coefficient
C_d	Carbonation depth
C_{env}	Environmental coefficient
C_{insp}^{θ}	Unit cost of the inspection technique θ
C_{rep}^{γ}	Unit cost of the repair method γ
C^{ρ}	Cost of the inspection sequence ρ
D_{CO_2}	Effective diffusion of CO_2
D_{ref}	Diffusion coefficient of reference
$D(T)$	Diffusion of CO_2 as a function of temperature
D	Global effective diffusion of CO_2
d	Diameter of the reinforcement

d_0	Initial diameter of the reinforcement
$d(t)$	Diameter of the reinforcement over time
E_c	Modulus of elasticity of concrete at 28 days
E_{eff}	Effective modulus of elasticity of concrete
eff_R	Efficiency of the repair method
f_{ck}	Characteristic compressive strength of concrete
f_{ct}	Yield stress of concrete
F	Faraday's constant
$G(X)$	Generic limit state function
$g(\eta(t))$	Limit state function for damage degree
H	Henry constant
H_{ref}	Reference Henry's constant
$I(t)$	Corrosion current density over time
I_i; I_j	Stochastic indexes for the decision-making analysis
I	Set of inputs for the efficiency analysis
I_o	Current flow density at the reference temperature
J	Set of outputs for the efficiency analysis
K_{sp}	Solubility product of calcium hydroxide
K_η	Coefficient of degradation improvement given by the repair method
k_c	Reaction rate constant between CO_2 and $Ca(OH)_2$
L	Longitudinal dimension of the concrete specimen
$LogN$	Log-Normal distribution
m	Humidity constant
M	Molar mass of the steel rebar
$MEA(\rho)$	Multidirectional efficiency analysis score for an inspection sequence ρ
N^ρ	Number of different inspection techniques necessary for the inspection sequence
O_ρ	Optimisation problem for the best inspection sequence
$p_1; p_2$	Polynomials for the detectability expression
$\mathcal{P}_D^\theta$	Proposed expression for the detectability of the inspection technique
P_{DBF}^ρ	Probability of damage detection before failure
$P_{DR}(\eta(t))$	Probability of Doing Repair
P_D^θ	Probability of damage detection of the inspection technique θ
PD^ρ	Maximum probability of damage detection before failure attained by the inspection sequence ρ

P_{RR}^{γ}	Probability of improve the damage condition in the structure
P_f	Probability of failure
$P_m^{\alpha}(\bar{\rho});\ P_j^{\beta}(\bar{\rho})$	Linear programming optimisation problems for the efficiency analysis
$P^{\gamma}(\alpha,\beta,\bar{\rho})$	Global solution for the efficiency analysis
Q	Diffusion activation energy
R^2	Coefficient of determination
R_1	Gas constant for effective diffusion
R_2	Gas constant for Henry's Law
R_G	Design resistance for limit state function
RH	Relative Humidity
RI	Random Index for consistency
r	Annual discount rate of money
S_{N_s}	Total inspection sequence possible
S_{ij}	Factor of reciprocity between the elements of the comparison matrix
S_G	Design load for limit state function
$SSTO$	Total sum of square
$SFA(\rho)$	Stochastic frontier analysis score for an inspection sequence ρ
SSR	Regression sum of squares
T_0	Initial temperature as reference of corrosion propagation period
T_{SL}	Service life time
T_f	Failure time
T_{icorr}	Time of corrosion initiation
T_{ref}	Temperature of reference
T^{ρ}	Set of the inspection sequence time
$T(t)$	Temperature for a given time
T	Temperature of interest
t	Time
$t(C_d)$	Time for attaining the C_d
$\bar{t_\iota}$	Optimal inspection time
U	Reaction activation energy
V_{corr}	Corrosion rate
V_p	Pore volume in the cement paste
W_{rust}	Mass of the corrosion product per unit length of the reinforcement

W^{ρ}	Time frame between the first and the last inspection time
w	Water content
w/c	Water/Cement ratio
$x(\rho)$	Vector of all the inputs for the efficiency analysis
$y(\rho)$	Vector of all the outputs for the efficiency analysis
Y	Minimum time admissible between consecutive inspections
$\bar{y}$	Mean value of the sample in the correlation analysis
y_i	Measured value from real carbonation data
$\hat{y_i}$	Theoretical value calculated for the correlation analysis
z	Valence of reaction in corrosion process
α_D	Empirical parameter for effective diffusion of CO_2
α_{ae}	Activation energy constant
$\alpha_{insp}^{\rho_i}$	Individual cost of the inspection technique associated with ρ_i
α_{insp}^{θ}	Fraction of cost associated with an inspection technique
$\alpha_m^*(\rho);\ \beta_j^*(\rho)$	Optimal solutions to the optimisation problems for the efficiency analysis
α_t	Coefficient of binding agent in Häkkinen formula
α^{γ}	Fraction of cost associated with a repair method
β_r	Pre-exponential factor
β_t	Coefficient of binding agent in Häkkinen formula
β	Reliability Index
δ	Thickness of the porous zone of concrete
γ_{sr}	Relation between steel mass and rust
γ	Repair method
$\eta_{0.5}^{\gamma}$	Mean damage degree associated to the repair method γ
$\eta_{0.5}^{\theta}$	Damage intensity at which the inspection technique has a 50% probability of detection
η_{cr}	Critical damage degree in the structure
η_{max}^{θ}	Maximum threshold damage degree for the detectability of the inspection technique θ
η_{min}^{θ}	Minimum threshold damage degree for the detectability of the inspection technique θ
η_{max}^{γ}	Maximum boundary point for the capability of the repair method γ
$\eta_{min}^{\rho_i}$	Minimum damage degree for the detectability of the inspection technique associated with ρ_i
η_{min}^{γ}	Minimum boundary point for the capability of the repair method γ
η_{rep}	Damage degree after the repair

η_{th}	Threshold damage to perform the repair in the structure
$\eta(t)$	Corrosion damage degree over time
Λ^N	Efficiency measurement for a number of sequence under study
λ_{CR}	Correction factor for the consistency ratio
λ_{max}	Eigenvalue of the comparison matrix
λ	Vector of intensity variables regarding the linear combination of $x(\rho)$ and $y(\rho)$
ξ_{ij}	Parameter for the level of importance of each alternative regarding a criterion
$\lvert\rho\rvert$	Number of inspection techniques applied in the inspection sequence ρ
ρ	Particular inspection sequence
ρ_c	Absolute density of cement
ρ_i	Inspection technique applied at the position i in the inspection sequence ρ
ρ_{rust}	Density of the rust
ρ_{steel}	Density of the steel
ρ_w	Absolute density of water
θ	Set of inspection techniques
σ^{γ}	Standard deviation of the damage degree associated to the repair method γ
σ^{θ}	Standard deviation for the detectability of the inspection technique θ
σ	Standard deviation of the mean value
$\psi_{T_f}(t)$	Probability density function regarding the time to failure
$\psi_{T_{icorr}}(t)$	Probability density function regarding the time to corrosion initiation
ϱ^{θ}	Expression for the damage parameters of the inspection technique θ
μ	Mean value
ϑ	Poisson's ratio of concrete
ϖ^{γ}	Coefficient of maximum repair achieved by the repair method
$\pounds$	British Pound
Δ	Enthalpy constant
Φ	Standard normal cumulative distribution function

ACRONYMS

ABNT	Associação Brasileira de Normas Técnicas
ACI	American Concrete Institute
AE	Acoustic Emission
AHP	Analytic Hierarchy Process
ANS	Automated Neural network Search
AR5	Fifth Assessment Report
ASTM	American Society for Testing and Materials
BEM	Boundary Elements Method
BIM	Building Information Modelling
BS	British Standard
CAF	Corporación Andina de Fomento
CBH	Norma Boliviana de Hormigón Armado
CDF	Cumulative Distribution Function
CEB	Comite Euro-International du Betón
CEPAL	Comisión Económica para América Latina y el Caribe
CIRSOC	Centro de Investigación de los Reglamentos Nacionales de Seguridad para las Obras Civiles
CNCC	Comisión Nacional de Cambio Climático
COIN	Concrete Innovation
CORDEX	Coordinated Regional Climate Downscaling Experiment
CPH	Comisión Permanente del Hormigón
CRS	Constant Returns to Scale
DEA	Data Envelopment Analysis
DGEEC	Dirección General de Estadísticas, Encuestas y Censos
DRS	Decreasing Returns to Scale
EHE	Instrucción de Hormigón Estructural
EN	Europäische Norm
ESRL	Earth System Research Laboratory
FDH	Free Disposability Hull
FEM	Finite Elements Method
fib	Federation International du Betón

FRH	Free Replicability Hull
GA	Genetic Algorithms
GDP	Gross Domestic Product
GPV	Global Priority Vector
HadGEM2-ES	Hadley Global Environment Model 2 – Earth System
IIASA	International Institute for Applied Systems Analysis
IPCC	International Panel on Climate Change
IRS	Increasing Returns to Scale
ISO	International Organization for Standardization
JGCRI	Joint Global Change Research Institute
LPR	Linear Polarisation Resistance
MADM	Multi Attribute Decision-Making
MCDM	Multi-Criteria Decision-Making
MCS	Monte Carlo Simulation
MEA	Multidirectional Efficiency Analysis
MODM	Multi Objective Decision-Making
NDM	Non-Destructive Methods
NDT	Non-Destructive Technique
NIES	National Institute for Environmental Studies
NOAA	National Oceanic & Atmospheric Administration
NPV	Net Present Value
NSGA	Non-dominated Sorting Genetic Algorithm
NSR	Norma Sismo Resistente
PCFV	Partnership for Clean Fuels and Vehicles
PNUD	Programa de las Naciones Unidas para el Desarrollo
PPM	Parts Per Million
PZT	Piezoelectric lead Zirconate Titanate
RAC	Recycled Aggregate Concrete
RC	Reinforced Concrete
RCD	Regional Climate Downscaling
RCP	Representative Concentration Pathway
RILEM	International Union of Laboratories and Experts in Construction Materials, Systems and Structures
RTS	Returns To Scale
SCM	Supplementary Cementitious Materials
SEAM	Secretaría del Ambiente

SFA	Stochastic Frontier Analysis
SRES	Special Report on Emissions Scenarios
TGA	Thermogravimetric Analysis
TRRL	Transport and Road Research Laboratory
UCD	Ultimate Carbonation Depth
UNFCCC	United Nations Framework Convention on Climate Change
USA	United States of America
USD	United States Dollard
VRS	Variable Returns to Scale
WMO	World Meteorological Organization

CHAPTER 1

Introduction

CHAPTER 1

1 INTRODUCTION

1.1 Context of the Research

Reinforced concrete (RC) structures constitute a large part of all existing infrastructures throughout the world. Its successful is given by its great geometric adaptability, versatility and durability of this type of construction material. Nonetheless, the concept of concrete durability is linked to a certain period often associated with a poor design, defective construction, inadequate materials selection or a more severe environment than expected. Then, the service life is based on the ability of a structure to overcome degradation, which means that there is considerable interest from the infrastructure managers in maintaining and extending that period (Broomfield, 2007).

According to the ISO 15686-1, the service life of structures may be defined as the period of time after installation during which a building or its parts meets or exceeds the performance requirements. In these terms, the service life planning must comprise the preparation of the brief and design for the building and its parts to achieve the desired design life to reduce the costs of building ownership and facilitate maintenance and refurbishment. Therefore, when the service life of the building and its parts are estimated, maintenance planning and value engineering techniques can be applied. Likewise, the skill and expertise of the technician or organization undertaking the service life planning will be crucial to the reliability of the planning (ISO 15686-1, 2000). It should be mentioned that in this research, the term lifespan will be used to define the "age" of the structure (i.e., the time from which the structure was built) to differentiate it to the definition of service life.

In this way, the maintenance of structures has captured particular interest on the part of structural engineers from the end of the last century. Concepts and theories such as the reliability of a structure, the life cycle analysis and the adaptation of new materials to the durability requirements have been the primary approach of the maintenance studies performed so far. Therefore, to control the initial stages of degradation and prevent the failure of the structural elements, it is paramount to perform maintenance strategies. The formulation of a most cost-effective and suitable maintenance strategy can enable better budget allocation and can also minimise the decline in the performance of infrastructures during their entire life cycle (Flores-Colen and de Brito, 2010).

Currently, the challenge for engineers lies in need to ensure that structures can withstand the environmental factors that determine their degradation, which must often be considered under the constraint of a limited budget. The study of the degradation of structures caused by environmental phenomena did not reach a significant interest until a few decades ago when climate change became tangible as a critical problem that affects the daily integrity of human beings. Furthermore, such as the buildings built in earlier times (e.g., bridges, cathedrals, castles, and so on), it is now possible to classify several of the 20th-century concrete structures as modern heritage, whose maintenance has cultural and historical value for each country.

Climate change is a highly controversial issue from the political point of view. However, regarding scientific evidence, it is a reality that cannot be avoided for a long time. This problem encourages the need to develop innovative strategies to adapt the structures to the degradation risks and guarantee their durability for as long as possible. Therefore, several studies are found in the literature regarding the optimisation of inspection and maintenance planning for infrastructures. This optimisation should seek to cover the most important aspects of a life cycle analysis such as the cost of the project, the durability, and the safety of the construction system.

This research thesis has as primary objective the development of an optimised maintenance methodology for concrete structures with corrosion risks by carbonation. Furthermore, this degradation risk will be analysed considering the potential effect of global climate change on the accentuation of the degradation process. The scientific methodology applied in this research is associated with the hypothetic-deductive method corresponding to the logical-theoretical approach. This method of investigation consists on the repeated observation of facts and comparable phenomena to extract the hypotheses. Then, these hypotheses are used through the inductive process of interpolation to make predictions of individual phenomena.

This thesis includes a section that addresses the problem of carbonation in concrete structures in Paraguay. This problem led to the elaboration of maintenance strategies based on decision models. These models can be used as an analysis tool for engineers working in the field of building maintenance in the country, and other countries with similar degradation problems. Therefore, the purpose of this study is to provide a numerical model to optimally formulate maintenance strategies in concrete structures under the corrosion risk of reinforcement by carbonation phenomenon.

Carbonation is a chemical phenomenon that affects most of the concrete structures in the world. In the case of Paraguay, and as will be seen later, it is one of the main causes in the premature degradation of infrastructure. On the other hand, carbonation is a natural process that is directly related to environmental parameters such as the concentration of carbon dioxide (CO_2), temperature and relative humidity. Considering this approach, climate change plays a meaningful role in the future degradation of concrete structures caused by this phenomenon.

Several countries with rapidly developing infrastructure, economies or poor supervision and quality control procedures in construction have led to poor quality concrete and low concrete cover to the steel leading to carbonation problems (Broomfield, 2007). This is the case of Paraguay regarding the degradation of its infrastructures. The result of this thesis involves a first step for the development of optimised planning of the integral maintenance of buildings, which is highly necessary for structures of Paraguay. In turn, although this study focuses mainly on existing infrastructures, the results obtained concerning the advancement of carbonation can serve as a reference for the future standardisation of the cover thickness values for the concrete structures in the country.

1.1.1 The Climate Change

The Intergovernmental Panel on Climate Change (IPCC) has conducted numerous investigations through which they have concluded that much of the current warming of the earth's surface during the last 50 years is due to human activities, commonly referred to as anthropogenic activities. The climate change caused by these anthropogenic activities will probably persist for several centuries if the effects that this phenomenon produces in all aspects of daily life are not consciously taken into account (IPCC, 2007).

The climatic impact and its effects such as the increase in the intensity of rainfall, winds, temperatures and the generation of greenhouse gases cause degradation and considerable damage to the existing infrastructure every year. A study carried out in the United Kingdom indicates that the increase in wind speed by 6% is likely to cause damage to one million buildings at an approximate cost of between 1 and 2 billion pounds (£). Likewise, droughts in the summer could worsen causing an increase between 50% and 100% in the number of claims for subsidies to the state by the inhabitants in more vulnerable areas (Graves and Phillipson, 2000).

Each year, the lives of millions of people are affected when they are displaced from their residences due to climate impacts and risks. Between 2008 and 2014, an annual average of at least 22.5 million people was displaced by the direct threat or impact of floods, landslides, storms, forest fires and extreme temperatures (Yonetani, 2016). Several evaluations made by scientists have determined that many of the extreme weather events in 2011-2015, especially those related to temperature and drought, have had a considerable increase concerning the events expected because of anthropogenic activities. This assumes that climate change trends are likely to have effects that would worsen in the coming years in the medium and long-term (WMO, 2016a).

In the Latin American context, and more specifically in the region corresponding to Paraguay, the situation is not encouraging. According to the Andean Development Corporation (CAF, for its acronym in Spanish), Paraguay is in the eighth position in the ranking of the countries with the highest vulnerability index against climate change. The country is classified with an "extreme" risk category, which corresponds to a high exposure index. For this reason, it is necessary that the country join efforts to develop strategies for adaptability against the durability of the structures in the face of climate change and its effects (CAF, 2014).

The climatic impact to which a building may be affected, which is exposed to a limited period of time, can be described as a stochastic process. In this way, regional climate development based on global warming can be described as a stochastic process selected from the effects generated by climate or by climate change scenarios (Lisö et al., 2003). Therefore, the study of the maintenance of structures under the consideration of climate change must entail a dynamic analysis. This analysis should allow the adaptability of the degradation models to the predicted climate conditions for the future.

This research includes a chapter where a literature review regarding general aspects of climate change such as the effects, trends and climate scenarios expected by scientists is developed. Climate change as a global phenomenon entails the development of a very broad topic. However, this thesis aims to cover an analysis only of those climatic parameters that directly affect the degradation of concrete structures due to carbonation corrosion, namely temperature, relative humidity and concentration of carbon dioxide. The durability of the building elements depends not only on its physical, chemical or mechanical properties but also on the conditions of maintenance and environmental exposure to which they are subject (Sarja et al., 2005). In this way, climate change and its effects play an important role regarding the environmental exposure that determines the durability of the elements of the infrastructures.

Extreme weather events have always been present. However, these extreme events have been changing their average values, affecting the vulnerability of infrastructures, forcing construction professionals to generate new proposals for adaptability. Consequently, climate change is a real problem which cannot be ignored, but rather it is necessary to deal with the uncertainty of the climate process. In this way, it is essential to develop methods that are increasingly optimised so that resources can be used more efficiently to ensure the durability of the construction system.

1.1.2 Degradation in Concrete Structures

Concrete is a material widely used in infrastructures since the beginning of the last century. These structures, when exposed to aggressive environments, can present processes of degradation that compromise their service life. These degradation mechanisms can be categorised according to physical, chemical, biological and structural processes. The physical processes comprise the degradation caused by exposure to extreme environmental changes such as fires, freeze/thaw cycle, among others. The chemical processes are caused by reactions between the composition of the material and the environment such as sulphate attack, alkali-acid reaction, among others. Lastly, the biological and structural processes include the presence of bacteria (mould) and the overload effects respectively (Aguirre and Gutiérrez, 2013).

Several studies may be found in the literature regarding the degradation of structures. These studies are generally focused on specific aspects such as the type of structures, the type of composite material in the construction elements, the specific degradation mechanism, among others. In this research, the study focuses specifically on the degradation of concrete structures caused by reinforcement corrosion. In turn, the corrosion process considered is centred on the corrosion caused by the carbonation of the concrete.

Regarding carbonation-induced degradation, the interpretation of reinforcement corrosion requires a quantitative understanding of the environment, physical deterioration process, transport mechanism through the concrete, cracking process and the corrosion phenomenon. So, development of service life prediction model for RC structures exposed to a carbonated environment is often a complicated process (Taffese and Sistonen, 2013).

The largest infrastructure problem in industrialised countries may be associated with the economic loss and damage caused by the corrosion of steel in reinforced concrete structures (Broomfield, 2007). Among the main factors that cause the corrosion of reinforcement in concrete, it is possible to identify two causes commonly presented. These are carbonation and the presence of chloride ions. Corrosion in the reinforcement causes cracking of the surface of the structure and subsequent the spalling of the cover due to the expansion of the corroded rebar. Then, the corrosion rate directly affects the extension of the service life of the RC structures (Ahmad, 2003).

One of the principal factors in the evolution of construction technologies has been motivated by the need to use materials that are capable of lasting for an extended period of time. In these terms, the end of the service life of a structure is reached when it no longer satisfies the functions for which it was designed, i.e., the loss of serviceability. Eventually, this condition can be improved through repair and maintenance activities. However, when these activities become unprofitable or complex to practice, the prolongation of service life becomes complicated. Therefore, the service life of a structure is quite dependent on the nature of the degradation mechanism to which the structure is subject (Dyer, 2014).

According to the *Comite Euro-International du Betón* (CEB), a truly enhanced performance cannot be achieved by improving the materials characteristics alone. Considering the complex nature of environmental effects on structures and the corresponding response, it is necessary to involve in this improvement the elements of architectural and structural design, processes of execution, and inspection and maintenance procedures, including preventive maintenance (CEB, 1989).

1.1.3 Modelling of Optimal Maintenance

Buildings and infrastructure, in general, are structures that suffer the degradation of their initial capabilities and capabilities throughout their service life. In this way, it can be said that a construction system begins to degrade immediately after its placing in service because of two independent variables: the system and the surrounding environment. However, this inevitable degradation process can be managed and controlled through periodic maintenance activities that prolong its service life (Harris, 2001; Chew et al., 2004; Rikey and Cotgrave, 2005).

The British Standard Institution defines maintenance as the combination of all technical and administrative actions, including their control, necessary for the correct operation of a certain element. These maintenance actions include operations such as cleaning, inspections, repair and replacement of building elements (BS:3811, 1984). In this context, maintenance can be established as preventive, corrective or predictive actions. Preventive maintenance refers to all actions performed on a specific schedule to keep an element in functional condition. Corrective maintenance is the unscheduled repair performed when deficiencies or failures have been perceived in order to return an element to a defined state condition, and predictive maintenance constitute the use of modern measurements methods to accurately assess condition state of the constructive element (Dhillon, 2002).

In maintenance management, the repair strategy is decided to rely on the available budget, which is part of the Life-Cycle Cost approach where the repair criterion must be treated as an optimisation parameter. Extensive studies have initiated in maintenance management due to concerns about the huge amount of investment required to maintain existing structures in service and the budget limitation that is often presented. These studies aim to identify the reasons for the observed low performance of several concrete structures and afford tools that allow a more realistic assessment by means of inspections (Malioka, 2009).

The challenge of maintenance optimisation relies on that the state of conservation or performance of a structure cannot be estimated too accurately due to certain random variables that govern the mechanism of degradation. These random variables are given due to the uncertainty principle that governs all particles and phenomena of nature. This principle was developed in 1926, when the German scientist Werner Heisenberg formulated the principle of uncertainty which basically states that it is impossible to know, with high precision, the position and velocity of a particle simultaneously, without altering its natural conditions (Giribet, 2005). This makes it necessary always to admit a certain degree of uncertainty when predicting how a system will behave, which makes it quite complicated to predict the service life of a structure.

As Savage argues, when working with the prediction of the behaviour of a dataset it is common to fall into an error which he calls the "failure of the averages". This failure establishes that considering average values for the planning of activities will lead to a failure in the results. For this reason, the trend of the 21^{st}-century is innovation in the analysis of data dealing with uncertainty, describing and developing new methodologies and models that include simulations, decision trees, and other theories applicable to the real world (Savage, 2003).

Many numerical models can be found in the literature, which seeks to predict the degradation of the materials of a construction system by measuring its performance throughout its service life. There are also numerical models used for the planning of the maintenance of structures. These models, which will be developed later, can be elaborated according to deterministic or probabilistic methods. According to several studies the probabilistic methods are the one that best adapts to the stochastic characteristic of the degradation phenomenon.

This research proposes a decision-making model applied to the maintenance of concrete structures, which can be divided into two parts: the degradation model, and the model formulated for decision-making (Kallen, 2007). The degradation model includes a first analysis of the current state of conservation of the structure, while the decision-making model is used to predict the degradation applied to determine which strategy/policy of maintenance is optimal. Figure 1.1 shows a diagram for the development of a maintenance model.

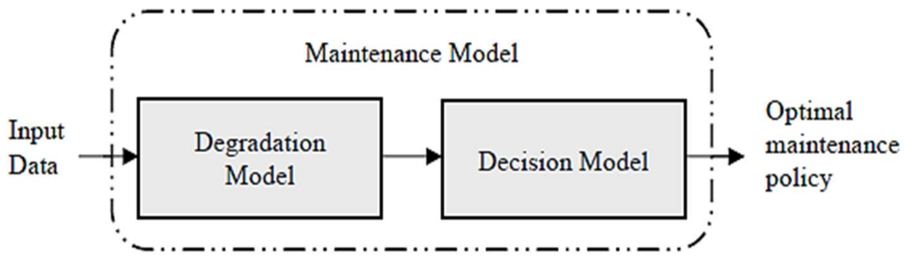


Figure 1.1 Representation of the basic elements of a maintenance model (Kallen, 2007)

For the decision model of this thesis, planning will be formulated for the inspection and maintenance of concrete structures subject to degradation by carbonation-induced corrosion. This planning is carried out with the purpose of achieving the optimisation of the maintenance processes based on the costs of the activities, the quality of the inspection and the effectiveness of the maintenance tasks in search of the extension of its service life.

Minimise maintenance cost, corrosion initiation, and failure probability while serviceability and safety are ensured are key issues of an improved management strategy for the maintenance of RC structures. Maintenance strategies should be oriented to guarantee optimum levels of serviceability and safety during the service life of the structure, minimising both the operational costs and the environmental impact (Bastidas-Arteaga and Schoefs, 2012).

1.2 Significance and Methodology of the Research

The degradation of infrastructure has been influenced by several factors that over the years have been analysed by researchers to develop measures that mitigate the degradation mechanisms and prolong their service life. Nowadays, there is a factor that has become more relevant in the last decades due to the considerable effect that it causes, not only to the integrity of the buildings but also to the health and well-being of the people. This factor corresponds to the effect of climate change and global warming that planet earth is currently experiencing.

Climate change and its effects is a problem that no scientist can ignore today. There are many organisations formed in different countries that seek to adopt palliative measures that slow down the rapid effect that this phenomenon is having. To this end, representatives of the most influential countries at a global level have proposed both short-term and long-term goals that mitigate the degradation of climatic and environmental conditions. Thus, large economic investments have been developed in projects to reduce greenhouse gas emissions and implementation of energy efficiency.

However, this commitment that seeks to address climate change must be carried out conscientiously and equitably in every corner of the planet. Otherwise, all the efforts made by a few will be counteracted by the indifference of others. Currently, the policies of the autonomous states directly influence the success or failure of these projects. That is, it is not possible to completely depend on these policies for an improvement in the effect of climate change. For this reason, it will be necessary to develop measures of adaptability to the consequences that climate change could bring.

The degradation and the corresponding maintenance and repair of the construction systems comprises a problem both from the viewpoint of the safety of its users and from the economic perspective. According to Balaras et al., about half of the expenses in the construction industry in Europe is oriented towards the repair, maintenance and rehabilitation of the existing buildings. The premature degradation of concrete structures is becoming the biggest problem in many countries, especially in urban zones due to adverse environmental conditions (Balaras et al., 2005). The activities related to the rehabilitation of the building stock, as a percentage of the total works concerning the building, have been continuously growing in many countries of Central Europe in the last 20 years. The long-term changes in the demand for buildings will force professionals to move their focus from new constructions to the maintenance and rehabilitation of existing buildings (Kohler and Hassler, 2002).

Regarding specifically to the corrosion, a study has shown that the estimated annual cost of corrosion worldwide exceeds the value of 1.8 trillion US dollars (USD), which translates into a 3-4 % of the Gross Domestic Product of the industrialised countries (Schmitt et al., 2009). In another more recent study, the global scale cost of corrosion was estimated even as 2.5 USD trillion (Koch et al., 2016). However, several studies estimate that between 25% and 30% of annual corrosion costs could be preserved if optimised practices in the treatment of corrosion were employed (Schmitt et al., 2009).

The cost of corrosion has a significant impact on the economic aspect of countries administration worldwide. For instance, a study about cost of corrosion by sector performed on the United States in 1998 has shown that the investment in the construction sector was 50 USD billion and, likewise, the same study performed on 2012 in India has shown an investment of 8015 USD million, of which 1543 USD million was regarding to maintenance and repair activities. Nonetheless, between 15 and 35% of these expenditures could be saved if corrosion control practices available and applied to all sectors (Koch et al., 2016). Figure 1.2 shows the cost of corrosion in the Industry sector, to which the construction sector is part.

Meanwhile, regarding the carbonation-induced corrosion of concrete structures, studies show that global climate change will affect the progression of the carbonation process in infrastructures. In other words, it is expected to observe much higher carbonation depths in the long term due to the effect of climate change (Wang et al., 2010; Stewart et al., 2011). Although climate change could have a minor effect in the very near future on the durability of structures, the real effects of climate change will become evident in approximately 30 years (Talukdar, 2013).

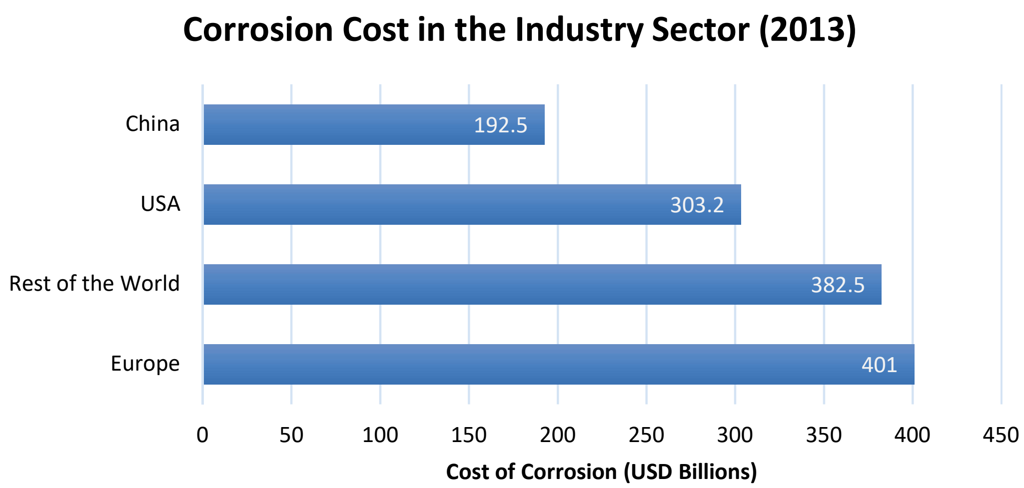


Figure 1.2 Worldwide Cost of Corrosion (Koch et al., 2016).

Paraguay being a developing country socio-economically speaking, the construction industry is experiencing significant growth since the end of the last century. However, the lack of control that occurs many times during the construction stage of the projects puts into question the quality of the constituent material of the structures. Considering its geographical location, Paraguay is a country that does not have maritime coast. This is one of the reasons why the predominant corrosion mechanism of the structures is carbonation, above the chloride attack. Nonetheless, it is not prudent to state that corrosion does not occur in the country because of the chlorides because previously it was common to use additives in the concrete mixture that possessed this chemical compound.

Degradation is a term commonly related to the failure of a system, being the degradation measurements an indirect way of determining the system's failure characteristics. In other words, degradation is nothing more than the measurement of performance, quality, the capacity of the structure or damage that may occur during the lifetime. For this reason, when proposing a numerical model for the maintenance of concrete structures submitted under any failure mechanism, it is important to consider a study on its degradation firstly.

For the study of the degradation of a structure, in general, numerical models are used that make possible the elaboration of what is known as the degradation curve. This curve represents the decrease in the performance of a system throughout its service life. As such, there are innumerable numerical models of degradation in the literature based on different analytical methods. However, not all of these methods have been successfully validated or have managed to represent the degradation process reliably.

It is almost impossible, considering the uncertainty, to predict with perfect accuracy the process of degradation of a structure. However, considering the highest number of variables and possible parameters that govern a particular degradation mechanism makes a model more or less reliable. For this point, after an exhaustive bibliographic research, the numerical model developed by Talukdar manages to meet these requirements (Talukdar, 2013). This supposes that such model achieves to predict with a significant precision the degradation process of the concrete under the consideration of the carbonation corrosion.

The potential of the model applied in this research to obtain the degradation curves is not only given by considering together the climatic and constructive parameters that govern the degradation, but also by considering the variability of these parameters (climatic) over time. Therefore, this feature fit perfectly in the context of climate change which is developed in this research.

These degradation curves are the basis for the formulation of optimal maintenance strategies for structural systems. These strategies include a decision-making model whose main objective is to prevent premature degradation and extend the life cycle of the structure. A decision support system is highly necessary to help engineers understand the benefits of real-time maintenance strategies. Proper repair and maintenance tasks to preserve the required performance or extend a specific service period for a building are essential for sustainable development in construction.

Figure 1.3 shows a diagram that represents the most critical aspects of the formulation of the maintenance strategy proposed by this research. The optimisation task seeks to cover a very sensitive and essential aspect within any constructive system, i.e., the economic perspective of a project. Many of the projects take into consideration the cost of the project about the design and construction of the construction system. Nevertheless, they leave aside the post process of every project: the monitoring and maintenance of the infrastructure.

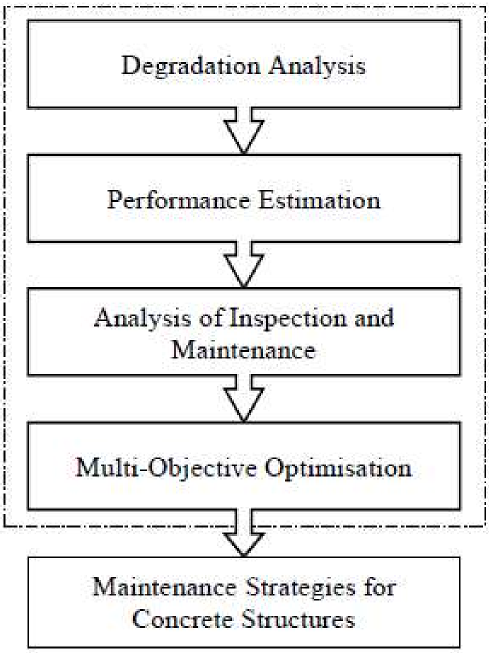


Figure 1.3 Formulation scheme of the support decision model for the maintenance of structures.

This research seeks to make both users of the infrastructures and the professionals involved aware of the immeasurable value provided by the optimised planning of maintenance tasks. Therefore, the significance of the maintenance of the structures must be appreciated from the functional perspective, safety, and above all, from the economic point of view of the project.

1.3 Research Objectives

The main objective of this thesis is to develop an optimised methodology for the formulation of strategies corresponding to the maintenance of reinforced concrete structures subjected to the degradation induced by carbonation, considering the effects of climatic changes. The proposed research seeks to raise awareness about this problem through the formulation of the following specific objectives:

- Analyse the numerical models developed in the literature for the study of carbonation-induced degradation in concrete structures;
- Identify the climatic parameters that affect the degradation mechanism studied and the respective projections estimated considering the effect of climate change;
- Select a numerical model and perform the calibration of the parameters used to obtain the degradation curves adapted to the structures in Paraguay;
- Develop a cost analysis of the inspection and maintenance of the structures;
- Establish the inspection times for the structure throughout its service life through the optimisation of cost and quality of the inspections;
- Establish the most appropriate inspection techniques to be applied to the structure throughout its service life, optimising the probability of damage detection of these;
- Develop a dynamic model for making decisions associated with maintenance activities where structural reliability is guaranteed at the lowest possible cost;
- Establish the appropriate repair methods in concrete structures considering the effectiveness of the repair in the improvement of the performance of the structure;
- Determine optimal maintenance schedule for minimum costs associated with carbonation-induced corrosion damage in reinforced concrete structures.

This research seeks to create a support tool for engineers who work in the field of maintenance of existing buildings. Although the focus of the maintenance model is oriented to the climatic and constructive conditions of Paraguay, it is intended that the method can be emulated for other regions of the world. However, special care must be taken to the specific conditions that govern degradation for each case. Figure 1.4 shows a flowchart of the activities and objectives to be achieved with the elaboration of this research.

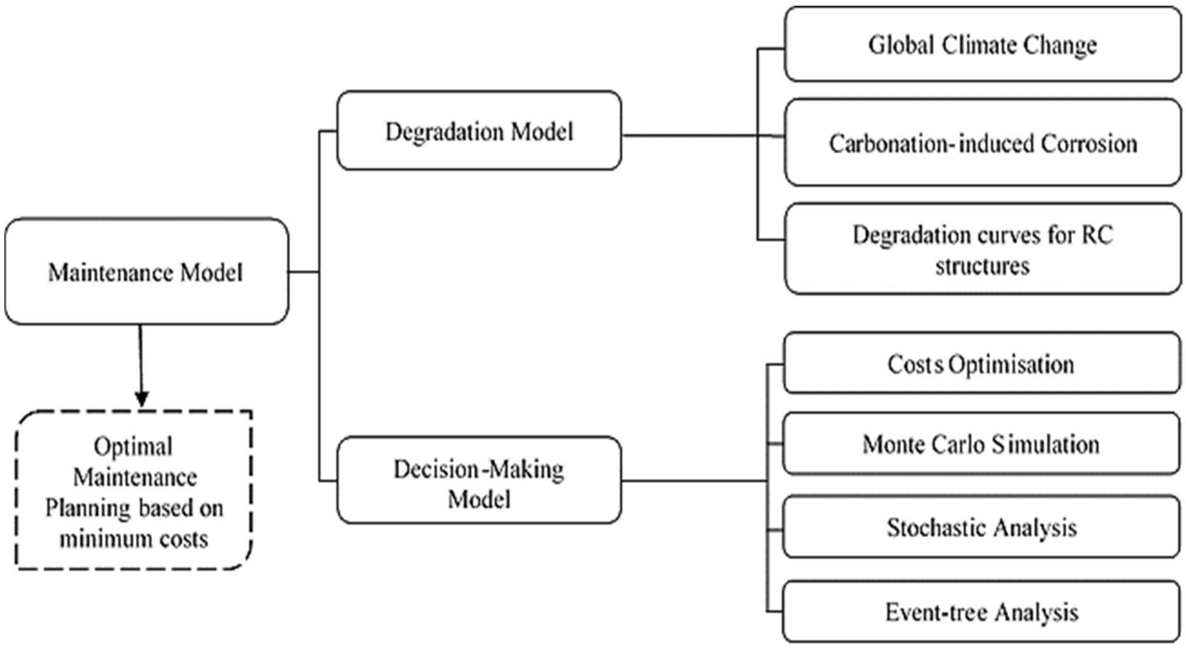


Figure 1.4 Flowchart with activities to develop research.

1.4 Outline of the Research

This research thesis is organised into six chapters of which the first chapter includes a brief overview to the subject to be studied; the objectives set out in the research, the justification of the research and the research methodology implemented. Furthermore, the structure of the topics that will be addressed throughout this investigation is presented.

The second chapter presents a theoretical summary found in the literature on the scientific bases that govern the phenomenon of climate change, its global effects, the trends of the variability of the most influential climatic parameters and the impact that climate change has on the durability of the structures. The effect that climate change is having on the countries of Latin America is also addressed in this chapter where vulnerability to this phenomenon is shown. Special consideration is also developed regarding the effect of climate change expected in Paraguay and the climatic scenarios that will be used in the study.

In the third chapter is described the state of the art regarding the degradation of reinforced concrete structures. The durability of the structures, the degradation mechanisms, the parameters that influence the corrosion of the reinforcement and the process of corrosion by carbonation of the structures are described. A brief overview regarding the expected effects of climate change in the carbonation depth of concrete structures is highlighted. This chapter also presents the methods of inspection and maintenance applied to carbonate structures, as well as a section on the problem of carbonation in the Latin American context. The chapter finally covers a brief case study on the degradation by carbonation of concrete structures in Paraguay. This case study shows the problem of carbonation in structures of the country and remarks on the importance of the quality control to ensure its durability.

The fourth chapter is oriented to the methods applied to the degradation modelling and the most important numerical models that have been developed over the years to the present. Some of the main findings and advances in knowledge referred to this degradation mechanism are exposed. A detailed description of the numerical model adopted in this thesis is made to obtain the degradation curves by carbonation. A description of the variables and parameters considered within the model to obtain the results adapted to the climatic and constructive context of Paraguay is presented. In this chapter, the degradation curves and corrosion initiation times and service life of the structures in the country are presented. The degradation curves show the carbonation depth expected for the concrete structures in Paraguay over the next 50 years considering two climatic scenarios. The reliability of the carbonation model has been validated through real degradation data. This real data comprise 206 carbonation tests performed in existing structures of Paraguay, which have been described in chapter three of this thesis.

The fifth chapter includes the main development, and therefore the most extensive of this research. The second stage of the maintenance methodology proposed in this research is developed. In turn, this chapter comprises the formulation of maintenance strategies through two stages: (i) the formulation of inspection planning, and (ii) the formulation of inspection and repair planning concurrently. Firstly, the optimisation criteria to be applied to obtain the inspection times are defined according to the minimisation of the costs and the maximisation of the inspection effectiveness. In this stage, an efficiency analysis is implemented in the maintenance model as a new perspective for the optimisation problem. The advantages of such analysis in the inspection planning are highlighted.

In the same chapter is formulated a dynamic decision-making model for the maintenance planning. This study comprises the maintenance management from a global perspective, i.e., considering the inspection and repair actions. The model is based on the Analytic Hierarchy Process (AHP) method that corresponds to the family of multi-attribute decision-making methods. The decision model proposed in this research is an extension of the traditional AHP method, where the stochastic approach is introduced in the methodology. The mathematical formulations established for the decision model and the results obtained after an example of application are also presented. The main advantages and the usefulness of the decision model proposed are discussed at the end of the chapter.

Lastly, the sixth chapter addresses the conclusions and final discussions obtained from this thesis. The importance of the research developed, the achievements, some limitations found in the development of the study and the proposals for future research are addressed in this chapter.

1.5 Publications associated with this Thesis

During the elaboration of this thesis, some scientific papers have been developed that have been published or are being considered for publication. These manuscripts are part of the result of this thesis and are presented below in the following list:

Benítez, P.; Rodrigues, F.; Gavilán, S.; Varum, H.; Costa, A. (2018). “Carbonated structures in Paraguay: Durability strategies for maintenance planning”. XIV International Conference on Building Pathology and Constructions Repair – CINPAR 2018. June, 20-22. Firenze, Italy.

Benítez, P.; Rodrigues, F.; Talukdar, S.; Gavilán, S.; Varum, H.; Spacone, E. “Analysis of correlation between real degradation data and a carbonation model for concrete structures”. Cement and Concrete Composites. Article in Press, https://doi.org/10.1016/j.cemconcomp.2018.09.019

Benítez, P.; Rocha, E.; Rodrigues, F. (2018) “Frontier Analysis on Optimal Inspection Times for Reinforced Concrete Structures”. XVI International Conference of Numerical Analysis and Applied Mathematics - ICNAAM 2018. September 13-18. Rhodes, Greece.

Benítez, P.; Rocha, E.; Talukdar, S.; Varum, H.; Rodrigues, F. “Efficiency analysis of optimal Inspection Management for Reinforced Concrete Structures under Corrosion Risk”. Construction and Building Materials. Submitted for Review, September 18th, 2018. Review N°: CONBUILDMAT-D-18-06087

Benítez, P.; Rocha, E.; Varum, H. ; Rodrigues, F. “A dynamic multi-criteria decision-making model for the maintenance planning of Reinforced Concrete structures”. Journal of Building Engineering. Submitted for Review. September 21st, 2018. Review N°: JOBE_2018_1118

CHAPTER 2

Climate Change

CHAPTER 2

2 CLIMATE CHANGE

2.1 Introduction

The development of this chapter covers the basic scientific notions related to climate change collected from the literature. The climate comprises a science with which, at first instance, civil engineers are not deeply familiarised. However, in recent years the climatic factor has reached a significant relevance in the field of engineering. The aim of this chapter is not to develop new knowledge bases, but rather to expose concisely the implication that this phenomenon has for the life of the planet, the durability of infrastructures, and also, for the life of human beings.

This chapter comprises an overview of studies related to the global effects of climate change, the trends of the variability of the most influential climatic parameters, and the effect of climate change on the durability of structures. There are many studies related to the analysis of climate variation, which is carried out through climate models. Some methodologies applied to the formulation of these models that are used to obtain climate parameter curves are also described in this chapter.

Since the beginning of the industrial revolution in the second half of the 18^{th}-century in the United Kingdom first, and then extending to Europe and North America, there has been a considerable increase in the change of land use and the use of fossil fuels. In turn, this has led to an increase in the emission of polluting gases known as greenhouse gases, which has driven to an increase in the effect of global warming. It is estimated that the effects of global warming have values that are five times greater than that presented by climate change under natural causes (McGrath and Lynch, 2008).

Climate change is a subject that is comprehensively studied at the scientific level. One of the main agencies that have managed to develop the most thorough research on climate change is the Intergovernmental Panel on Climate Change (IPCC). The IPCC is made up of some 2500 scientists and technical experts from more than 60 countries. Its function is to analyse the relevant, technical and socioeconomic information to understand the scientific elements of the risk posed by climate change caused by anthropogenic activities. Likewise, this organisation aims to analyse the climate change considering their possible attenuation and the possibilities of adaptation. In this way, the IPCC defines climate change as any climate variation which is produced over time due to natural variability as well as due to the activity of human beings (IPCC, 1995).

On the other hand, the United Nations Framework Convention on Climate Change (UNFCCC) defines this phenomenon differently. Climate change can be understood as a climate variation attributed directly or indirectly to human activity that alters the composition of the global atmosphere and is complemented to the natural variability of climate over comparable periods of time (UNFCCC, 1992). Under this premise, the IPCC would later declare that the contribution to global warming by anthropogenic activities correspond to more than 90% (IPCC, 2007).

Regardless of the conceptual definition that can be formulated on the phenomenon of climate change, the truth about this issue is that human beings are who inhabit this planet, so it is the responsibility of people to find solutions or measures that minimise the effects of change climate. These measures have already been taken by world leaders through agreements and projects with large economic investments around the world. Nevertheless, the political aspect has proved to be an influential factor in recent years, which leads to a somewhat hopeless future. A clear example can be seen in the scope of the United States, where after the presidential elections of 2017, the country has reduced much of the budget for investigation and development in this area.

The climate of our planet is changing and will continue to change in the coming decades. This change will be increasingly intense due to the gases of human activities that accumulate in the atmosphere, creating a barrier that overheats the surface of the planet. However, high temperatures comprise only part of climate change. This phenomenon alters other environmental parameters, increasing extreme events such as droughts, floods, intense rainfall and heat waves.

However, despite being a global problem and of large dimensions, it is still possible to minimise risks through palliative measures. In 2015, world leaders from all countries adopted the Climate Agreement in Paris, France. This agreement aims to find a rapid and deep solution to greenhouse gas emissions worldwide. This historic agreement commits all countries to make unanimous efforts to reduce the expected increase in ambient temperature. Also, this agreement addresses a commitment of financial support to developing countries, the exchange of scientific knowledge among countries on climate change, climate resilience and adaptation, training and public awareness.

The demand for accurate and accessible services related to climate, hydrology, marine and other environmental services will continue to grow in the coming years. They will be carried out, in part, due to the awareness of climate change and variations in weather patterns such as storms, floods and droughts. The aspect of human vulnerability, such as migration and the growth of large cities, will also contribute to the development of new measures to tackle this problem. In this way, the World Meteorological Organization (WMO) will ensure that current decision-makers, and those of future generations, have the tools and information they need to thrive in an environment even more complex and challenging (WMO, 2016b).

2.2 Scientific basis of Climate Change

The climate of planet Earth is constantly in a state of change and evolution according to a natural process of the system. However, these changes also occur due to anthropogenic activities such as deforestation or gaseous emissions in the atmosphere that subsequently produce the so-called greenhouse gases (IPCC, 1990; Cubash et al., 2013). These gases can retain heat and increase the temperature of the air near the earth's surface, causing an environment that resembles the artificial environment generated for crops through greenhouses (UNFCCC, 2006).

Among these gases, carbon dioxide (CO_2) is one of the most concerning gases since it has presented a considerable increase in the last decades. Figure 2.1 shows this trend where the red line represents the monthly mean values, and the black line represents the same, after correction for the average seasonal cycle. According to records, the CO_2 concentration reached a peak of 411.24 parts per million (ppm) in May 2018 (NOAA-ESRL, 2018). The greenhouse effect is a phenomenon that guarantees life on Earth while maintaining optimal temperature levels. However, a slight variation in this effect that causes increases or decreases in air temperature could endanger life as it is currently known.

The warming of the climate system is evident, as the scientific observations now show. These observations clearly denote increases in the global mean temperature of the air and oceans, the widespread melting of snow and ice, and the increase in mean sea level (IPCC, 2007). However, this phenomenon occurs as a consequence of the natural behaviour of the planet earth, which is currently altered due to the human activities that are developed in conjunction with the socioeconomic evolution of the countries.

The solar system is defined as a heliocentric system, in which the planet Earth intercepts the sun's rays from which a third of that energy absorbed is reflected again. The remainder of the energy, which keeps on Earth, is balanced by the emerging radiation and the atmosphere. The radioactive force is transformed into long-wave invisible infrared energy whose magnitude is determined by the atmospheric surface temperature of the planet. In this way, the surface temperature of the Earth is determined by the balance between the incoming solar radiation and the outgoing infrared radiation, as shown in Figure 2.2 (Cubash et al., 2013).

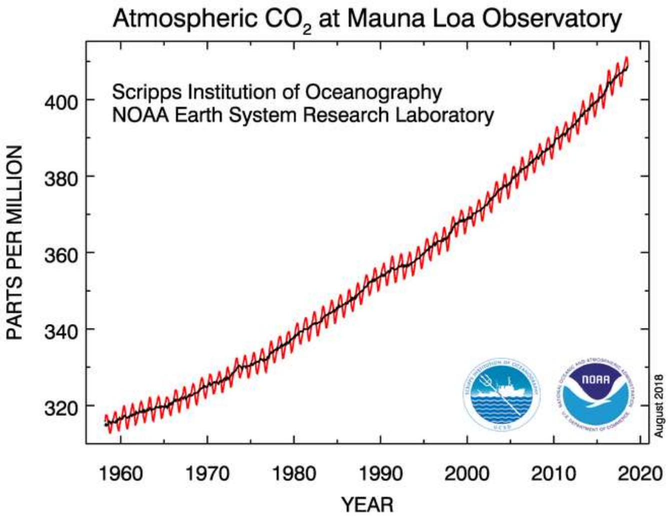


Figure 2.1 Global atmospheric concentration of CO_2 over the last decades (NOAA-ESRL, 2018)

If the amount of energy that enters as solar radiation is equal to the amount of outgoing energy, then there will be no change in the surface temperature of the earth. However, an increase in the concentration of certain gases would cause a greater volume of energy retained by the atmosphere, and consequently, a decrease in the amount of outgoing energy. This effect would cause a net increase in the surface temperature of the Earth, causing what is known as radiative forcing (Betts et al., 2001).

Radiative forcing refers specifically to the change in net vertical irradiation in the tropopause (the boundary between the troposphere and the stratosphere) due to an internal change. Also, radiative forcing may be caused by a change in the external forcing of the climate system, as for example, a change in the concentration of carbon dioxide or sunrise (Baede, 2001). The radioactive forcing calculated from changes in gas concentration for the period 1750-2011 shows that the increase in the amounts of atmospheric CO_2 after the industrial era provided 79% of the greenhouse gases (IPCC, 2013).

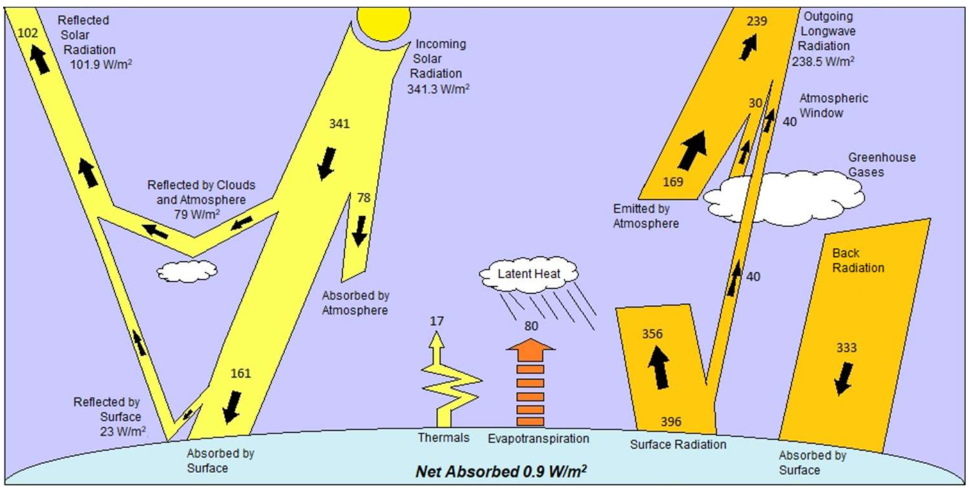


Figure 2.2 Global energy flow in the Earth by the period 2000-2004 (Trenberth et al., 2009)

With the industrial revolution, radioactive forcing increased considerably to 1.6 W/m^2 due to anthropogenic emissions. This value does not seem to be very significant. However, if the total surface of the Earth is considered, and also, that this energy is absorbed every day by the surface, then this increase reflects a much more impressive value that affects the rise in the surface temperature (Cohen and Waddell, 2009). The positive total radiative forcing implies that the absorption of energy by the climate system has taken place. The main contribution to this parameter comes from the increase of CO_2 in the atmosphere that has been occurring since 1750 (IPCC, 2014a).

The mean global surface temperature (i.e. the mean air temperature near the surface and the surface temperature of the sea) has increased since 1861. Throughout the 20^{th}-century, the temperature increase was $0.6 \pm 0.2\ °C$. This value is approximately 0.15°C higher than that estimated by the second IPCC report for the corresponding period up to 1994. This was due to the relatively high temperatures of the additional years (1995 to 2000) and improved processing methods of the data whose registration shows great variability. As an example of this improved processing methods, now it is possible to know that most of the global warming occurred during the 20^{th} century, specifically during two periods: 1910 to 1945, and 1976 to 2000 (Albritton et al., 2001).

Other studies have found that the production and use of energy participate in 46% of the alteration of the radiation of the climatic system of the planet. In turn, the use of fossil fuels corresponds to 70-90% of the total emissions of atmospheric carbon dioxide, with the remaining (30-10%) corresponding to the misuse of land, e.g. deforestation for agriculture (IPCC, 1990). Also, it has been found that for the period between 1970-2010, fossil fuels and industrial activity has contributed to the emission of carbon dioxide by 78%, which generated a considerable increase in greenhouse gases emissions (Edenhofer et al., 2014).

Once climate change was identified as a real and current problem, the IPCC in its first assessment report proposed a set of climate scenarios formulated from different hypotheses of climate behaviour according to several parameters (IPCC, 1990). These scenarios aided in the analysis of the possible effects of climate change in the future and what measures would be necessary to mitigate the impact. However, it was required a more detailed evaluation with more complex models that are consistent with climatic variations. In this way, the IPCC published a new set of climatic scenarios in its third assessment report in which it was established that the determining forces in future emissions of greenhouse gases are the population growth, the socio-economic development and the technological change (IPCC, 2000).

Therefore, the first four families of climatic scenarios emerged, which known as A1, A2, B1, B2. The scenario A1 supposes a rapid economic growth and a quick introduction of new and more efficient technologies. The scenario A2 considers a growing world population and regionally oriented economic growth; more fragmented and slower compared to other scenarios. Then, the scenario B1 supposes a world population similar to scenario A1, but with rapid changes in economic structures towards an economy of services and information. Lastly, the scenario B2 assumes a world in which local solutions to economic, social and environmental sustainability are emphasised, with the continuous growth of the population (lower than in scenario A2) and intermediate economic development (IPCC, 2000).

In 2007, the IPCC requested the scientific community to develop a new set of scenarios, as the existing scenarios (published as the Special Report on Emissions Scenarios, SRES) needed to be updated and expanded in scope. These new scenarios called "Representative Concentration Pathways (RCPs)" cover, by design, a range of radiative forcing levels examined in the open literature and containing relevant information for climate model runs. The RCPs provide an important reference point for new research by standardising on a standard set of year-2100 conditions and exploring alternative pathways and policies that could produce these outcomes (Van Vuuren el al., 2011b). Table 2.1 depicts a summary of the emission scenarios sets progress.

Table 2.1 Historical progress of climatic scenarios

Climatic Scenarios	Report Number	Publication Year
SA90	First Assessment Report	1990
IS92	Second Assessment Report	1992
SRES	Third and Four Assessment Report	2000
RCPs	Fifth Assessment Report	2009

In order to shorten the time between the development of emission scenarios and the use of the resulting climatic scenarios to investigate the possible impacts, scientists have been established a new process. This parallel process begins with the selection of four RCPs, each one corresponding to a specific radiative forcing path: RCP 8.5; RCP 6; RCP 4.5; RCP 2.6. Each of these scenarios represents a range of radiative forcing stabilisation by the end of 2100 in values of 8.5, 6.0, 4.5 and 2.6 W/m^2 respectively (Armenta et al., 2015).

The main difference between the new RCPs and the previous SRES scenarios is that there are no fixed sets of assumptions related to population growth, economic development, or technology associated with any RCP. RCPs provide a quantitative description of concentrations of the climate change pollutants in the atmosphere over time, as well as their radiative forcing in 2100. Considering each scenario, numerous studies using different climate models to obtain the variations of the climate parameters in the future have been carried out (Meinshausen et al., 2011; Riahi et al., 2011; Chong-Hai and Ying, 2012; Nazarenko et al., 2014; Kim et al., 2015; Park et al., 2015; San José et al., 2016; Sun et al., 2017; Tan et al., 2017).

The set of RCP scenarios are representative of the range of baseline and stabilisation radiative forcing, concentration, and emissions pathways. Therefore, these types of RCPs have been defined according to the desirable characteristics of range, number, separation and shape, robustness, and comprehensiveness. This research bases its analysis on these latest climate scenarios formulated by the IPCC. A brief description of some criteria considered for each scenario is presented in Table 2.2.

Table 2.2 Types of representative concentration pathways (Moss et al., 2008).

RCPs Scenarios	Radiative Forcing	Equivalent concentration (CO_2-eq)	Pathway shape
RCP 8.5	>8.5 W/m^2 in 2100	>~1370 CO_2-eq in 2100	Rising
RCP 6.0	~6.0 W/m^2 at stabilisation after 2100	~850 CO_2-eq (at stabilisation after 2100)	Stabilisation without overshoot
RCP 4.5	~4.5 W/m^2 at stabilisation after 2100	~650 CO_2-eq (at stabilisation after 2100)	Stabilisation without overshoot
RCP 2.6	Peak at ~3.0 W/m^2 before 2100 and then decline	Peak at ~490 CO_2-eq before 2100 and then decline	Peak and decline

The RCP 8.5 has been developed through the MESSAGE model and the International Institute for Applied Systems Analysis (IIASA), Austria. An increasing greenhouse emission over time is associated with this scenario. The RCP 6.0 was developed in Japan by the AIM modelling team at the National Institute for Environmental Studies (NIES). The scenario supposes the application of a range of technologies and strategies for reducing the greenhouse gas emissions. Then, the RCP 4.5 has been developed in the United States by the GCAM modelling team of the Pacific Northwest National Laboratory's Joint Global Change Research Institute (JGCRI). Lastly, the RCP 2.6 has been developed by the IMAGE modelling team in the Netherlands. This scenario is associated with low greenhouse gas concentration levels that are reduced substantially over time (Van Vuuren et al., 2011a)

2.3 Climate Change Effects

One of the most important results of recent IPCC reports has shown that between the period 1995 and 2006, the warmest cycle according to the world record of temperature has been presented, whose records date back to 1850. Studies show that the global mean surface air temperature has increased by 0.3°C to 0.6°C over the last 100 years, with the five global-average warmest years being in the 1980s. On the other hand, the average sea level increased due to the warming between 1.3 and 2.3 mm per year between 1961 and 2003. This increment was faster over 1993 to 2003 with an average sea level rise around 3.1 mm per year. Thereafter, it was found an estimated total global average sea level increase of 0.17 mm per year for the 20th-century (Cubash et al., 2013). Another analysis suggests that there are few chances to maintain an increase below 2 °C for the global average surface temperature by the end of this century (Anderson and Bows, 2011).

For the period 1987 to 1998, the average annual number of disasters linked to the climate phenomenon was 195; a value that was increased to 365 (87%) for the period 2000-2006. Of all climate disasters during the 1990s, 75% of them included floods and droughts as the most significant causes. Statistics show that more than 95% of all deaths caused by natural disasters occurred in developing countries. Moreover, the losses due to these disasters would be 20 times higher in developing countries than in industrialised countries (UNFCCC, 2008). During the 1990s, in the United States, the number of natural disasters or catastrophes has doubled over previous frequencies, causing economic losses equal to 5 million dollars (Meehl et al., 2000).

Regarding the economic aspect, climate change has an alarming effect on the consequences of natural disasters. The World Bank states that the funds required for adaptation in developing countries would be worth between 75 to 100 billion dollars per year (World Bank, 2009). On the other hand, studies have determined that the expectation regarding the costs of adaptation to climate disasters is equivalent to between 27 and 67 billion dollars per year for developing countries, and between 44 and 166 billion dollars per year worldwide (UNFCCC, 2008). Therefore, the economic factor also deserves to be taken into account when considering measures on the effects of climate change (Garlati, 2013).

The effects of climate change are also denoted by the general behaviour of natural phenomena. According to the latest IPCC report, the atmosphere and the ocean have warmed up, snow and ice volumes have decreased, sea levels have risen, and greenhouse gas concentrations have increased. It is quite likely (90-100% probability) that, on a global scale, the numbers of cold days and nights have decreased and that the number of warm days and nights has increased. It is also probable (66-100% probability) that heat waves will increase, that more episodes of intense precipitation will occur, higher intensity and/or duration of droughts or a greater intensity of activity of tropical cyclones (IPCC, 2014a).

In the Antarctic Peninsula, the ice barrier known as "Larsen C" has changed its structure in recent years due to a crack. In January 2017 it was observed, through satellites, that this crack already has a length of about 175 km, which means that this ice barrier is linked to the peninsula just by a "thread". When complete separation occurs, it will be an unprecedented event, although it is still difficult to predict when it will happen. It is known that Larsen C plays an important role as an ice barrier acting as a reinforcement of the peninsula. Thus, some of the immediate effects could be seen in an increase of sea level in some parts of the world (De Rydt et al., 2018).

Studies focused on the United States have recorded changes in the number of days that exceed the ambient temperature limits. For the period 1910-1998, there has been a slight decrease in the number of days below the freezing temperature in several regions of the country. On the other hand, evidence of the decrease in the number of cold days since 1930 has been found in northern and central Europe, which seems to be associated with sharp increases in winter minimum temperatures. In Australia and New Zealand, the frequency of cold days has decreased coincidentally with the increase in daily minimum hot temperatures (Cooter and Sharon, 1995; Heino et al., 1999; Plummer et al., 1999).

In Australia, much of the country has experienced an increase in heavy rainfall events throughout the year, except in the southwest of the country where there has been a decline in both rainy days and storms. Similar effects occur in North America, where there has been a significant change in extreme short-term events such as temperature and rainfall. Also, an increase in the relative humidity of the atmosphere has been observed for several regions of North America, which causes an increase in the frequency of intense rainfall (D. Easterling et al., 2000).

Urban areas can concentrate much greater global risks due to extreme rainfall, floods, droughts, air pollution, water scarcity or thermal stress. This aspect is intensified for low-income people, who have poor quality housing and in most cases are in exposed areas. For this reason, one of the first adaptation measures, in general, is to reduce this exposure and vulnerability to current climate variability (IPCC, 2014b). The impacts produced by the recent extreme phenomena of heat waves, floods, droughts, cyclones or forest fires, show the vulnerability of some ecosystems to this climatic variability. Among these impacts, it can be mentioned the damage to infrastructure, alteration of ecosystems, disorganisation in food production and water supply, consequences for mental health and human well-being, among others.

In the Latin American context, historical records for the period 1970-2010 show variations in temperature and precipitation. This variation causes an increase in the occurrence of extreme weather events, an increase in sea level, and a reduction in the water reserve in the glaciers (Samaniego, 2009). Furthermore, changes have been observed in the flow and availability of water in the Rio de la Plata basin, as well as changes in precipitation affecting freshwater systems. As a consequence, the quality and quantity of available water are affected, which in turn will affect the production and quality of food. There was also observed an increase in extreme temperatures in Central America and most of the tropical and subtropical zone of South America (Carabine and Lemma, 2014).

According to Magrin, rainfall has increased considerably to the southeast of South America, involving Argentina, Uruguay, Paraguay and part of Brazil, Peru and Bolivia. However, in Central America and Mexico, the effect of climate change could be seen more due to droughts in the region. The climate risk in Latin American countries is mainly due to floods, landslides, droughts, and hurricanes as the most significant hydrometeorological threats (Magrin, 2015).

The effects of climate change will be felt more intensely in developing countries due to the high vulnerability they present to natural disasters, temperature increases and sea level rise (Mordt et al., 2015). Nevertheless, the Latin American countries constitute an essential block in the regulation and stabilisation of the global climate, since considering their development needs, it is possible to adopt clean, efficient and low carbon technologies. In this way, great opportunities arise in the region regarding reducing the use of energy based on fossil fuels on which the highly industrialised countries now depend (Carabine and Lemma, 2014).

One of the most meaningful conclusions of the last report of the IPCC, and at the same time worrisome, is that the effect of climate change leads to a global problem whose eradication is not very simple. Due to historical emissions of carbon dioxide, the stabilisation or reduction of the high greenhouse gases concentration will not be feasible in the coming centuries (IPCC, 2013). Preventing the impacts of current climate change is practically impossible, so it is convenient to prioritise the implementation of strategies to reduce vulnerability to climate change. However, sustainable economic growth is also important in order to prevent future increases in greenhouse gas emissions (Conde-Álvarez and Saldaña-Zorrilla, 2007).

2.4 Trends of Climate Change

As explained above, climate change is a real problem whose effects have been felt tangibly since the end of the last century. Making a parallel with the medical sciences, it could be said that the planet Earth is in a critical state due to the effects that climate change is causing in all environments and ecosystems. Unfortunately, the forecast of the planet is not encouraging considering the trends of climatic phenomena for the next 100 years. Many studies indicate, based on climate scenarios, that all the extreme weather events that are currently observed have a high probability of being attenuated in the long term.

It is probable that, by the end of the 21st-century, the global surface temperature will be higher by 1.5 °C than the corresponding period of 1850-1900 for all scenarios considered by the RPC, except for the RCP 2.6 scenario. Furthermore, it is probable that this temperature will be higher by 2 °C for the RCP 6.0 and RCP 8.5 scenarios, and even more probable that it to be higher by 2 °C for the RCP 4.5 scenario. Global warming will continue to show interannual and decadal variability and will not be uniform across regions. The oceans will continue to warm up during this century, and the heat will penetrate from the surface to the deep layers of the oceans, affecting ocean circulation and the sea life (IPCC, 2014b). Figure 2.3 shows the increase in the expected surface temperature for the RCP scenarios until the end of the 21st-century.

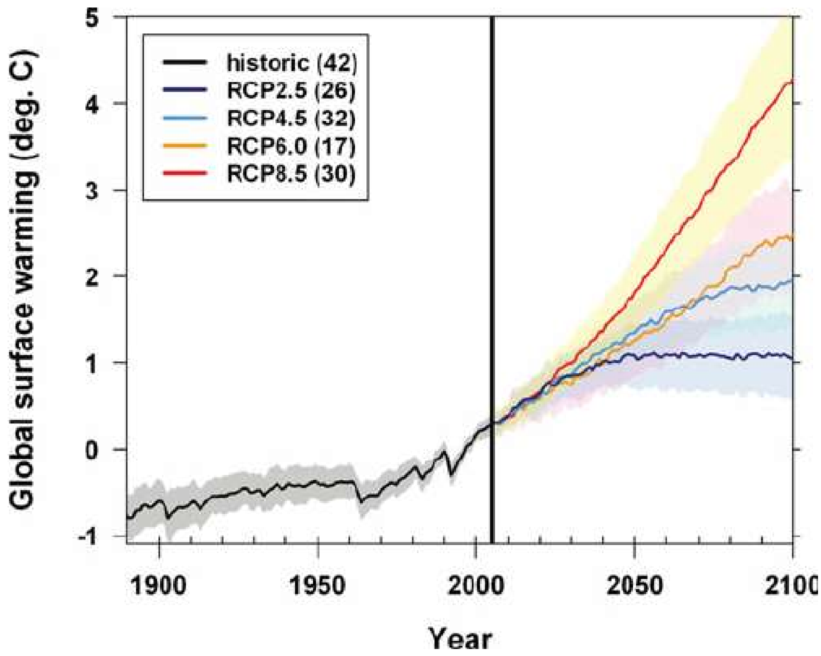


Figure 2.3 Trends of the global surface warming increase in the coming years (Knutti and Sedlacek, 2013).

In a global context, it is very probable that heat waves will be more intense, frequent and extensive, causing a decrease in the cold days. The intensity of rainfall will increase in tropical areas, while the risks of drought will increase for other regions of the planet. Due to warming, glaciers could decrease their volume causing an increase in sea level. This projected reduction is accentuated in the Arctic, where some models predict that for some climate scenarios, in the latter part of the 21st-century, the ice cover of the ocean could completely disappear (Meehl et al., 2007). In the Latin American context, an increase in temperature of 2.5 °C is projected for Central America and warming in South America that would reach 4 °C more than the current mean temperature by the end of this century (Magrin, 2015).

Under these considerations, the main objective worldwide is to mitigate greenhouse gas emissions, which includes the origin of the problem. The goal is to stop the increase in the concentration of greenhouse gases in the atmosphere by the end of this century, which, however, implies an overall increase in temperature between 2.5 °C and 3.5 °C. Otherwise, if the goals are not reached, and if it is not possible to maintain a constant concentration between 500 and 550 ppm, margins of change would arise for all systems so achieving an inflexion point with very low possibilities of adaptation (Samaniego, 2009).

Regarding the concentration and emission of carbon dioxide, this greenhouse gas would remain in the atmosphere more easily in warmer climates. For the climatic scenario RCP 8.5, carbon dioxide concentrations simulated by climate models foresee a concentration that varies between 730 and 1020 ppm by the end of this century, with a standard value above 900 ppm, as shown in Figure 2.4. This value entails more significant climate change, a greater impact of the carbon cycle, and, therefore, a more considerable increase in temperature.

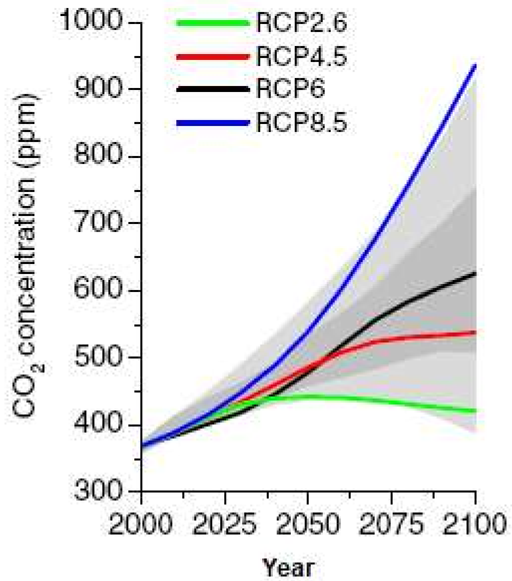


Figure 2.4 Expected CO_2 concentration for the RCPs scenarios (Wayne, 2013).

Likewise, if the current concentration of CO_2 (about 400 ppm) is taken into account, it can be seen in the previous figure that, for the best scenario (RCP 2.6), this concentration would remain close to the current values. However, considering that this scenario requires reductions in greenhouse gas emissions that are quite pretentious concerning reality, currently, the trends for the RCP 2.6 scenario are commonly ignored in the studies. Another area to analyse is the industrial sector. According to the IPCC, climate change will affect the cost of energy and the construction of infrastructure. In construction, higher demands will arise regarding service standards and this sector will have to submit to new regulations related to climate. In addition to this, there will also be a risk of the higher cost of primary resources (IPCC, 2007).

Another parameter that is also very influential to the context of the study, i.e. the carbonation of structures, is relative humidity. However, several studies have shown that the increase will not be significant due to the expected increase in temperature. For this reason, a large amount of IPCC's climate change studies are based on climate models that assume constant the relative humidity.

2.5 Climate Change in the context of Paraguay

Paraguay is a country highly vulnerable to climatic conditions because the productive structure is based on agriculture and livestock. That is, the country has an infrastructure related to international trade which is highly dependent on the flow of navigable rivers. Bearing this in mind, it is possible to conclude that climate change would have significant impacts on the Paraguayan economy and that, if short-term adaptation measures are not implemented, the costs of this impact would be high (CEPAL, 2014).

According to a study performed by the Andean Development Corporation (*CAF*, for its acronyms in Spanish) called "Index of Vulnerability and Adaptation to Climate Change in the Latin America and the Caribbean Region", Paraguay is classified in the category of "extreme risk" considering the climate change impact. Precisely, the country was placed at number 8 in a total of 33 countries in the region, where the analysis has included an assessment of the risk of exposure to climate change concerning human sensitivity and the ability of the country to adapt (CAF, 2014). It is estimated that the climate in Paraguay will be drier and with greater thermal oscillation than the registered trademarks in 2007. In this way, the summers and winters will be warmer. Another effect of climate change is the difficulty of access to water due to prolonged droughts, which will cause an increase in desertification, with soil degradation that will be more favourable to fires (PNUD, 2007).

Intense rainfall, storms, floods caused by extreme weather events are other climatic effects that have affected communities causing physical, economic and suffering to the population (SEAM, 2011). Moreover, it has been detected that the rainfall regime has increased from 400 mm per year in the Chaco Region (north of the country) to more than 1700 mm per year on the Paraná River coast in the Eastern Region, and it was noted that, in general, summer rains occur as large storms (SEAM, 2015). In order to contrast the future effects with current conditions, Figure 2.5 shows the annual variation of temperature and relative humidity for the city of Asunción, capital of Paraguay.

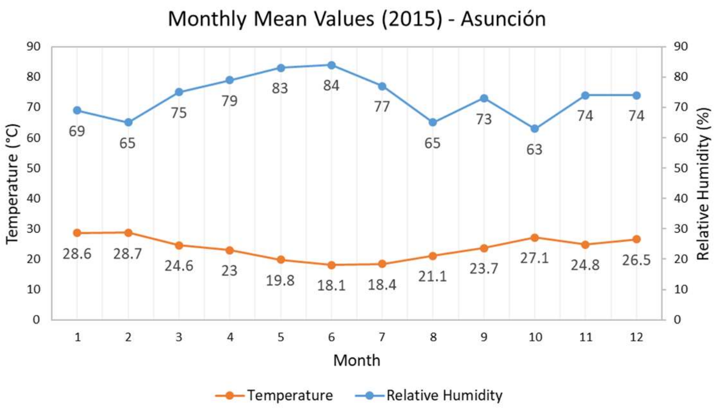


Figure 2.5 Variation of temperature and RH in Asunción (DGEEC, 2015)

Regarding greenhouse gas emissions, Paraguay increased these emissions by 153% in 2011 concerning 1990 records. This situation is a common factor in countries that are developing. However, the purpose of climate change programs for these countries is to promote the search for mechanisms and processes that aid to mitigate emissions, and above all aiming for sustainable development. On the other hand, the increase in temperature could also cause an increase in the relative humidity of the atmosphere, which in turn originates an increase in the occurrence of rainfall (PNUD, 2007; SEAM, 2011, 2015).

The National Commission on Climate Change of Paraguay (CNCC for its acronyms in Spanish) emphasises that the generation of scenarios of climate projection is essential for the development of adaptation strategies to the threats of climate change. These projection scenarios allow identifying the possible impacts by geographical area taking into account a specific sector (CNCC, 2016). Several climatological studies have been developed in Paraguay to know the most vulnerable regions to this global phenomenon. These studies allow the establishment of public policies that mitigate the effect on society and allow the optimisation of resources.

In the Second National Communication on Climate Change in Paraguay, different climate scenarios were analysed in the context of the country. The obtained results showed an increase in the temperature around 2.5 °C for the year 2050 and a significant variation of the precipitations in which a decrease is estimated in the western and northeastern part of the country; and more significant increases towards the north, east and southeast. Also, in all the analysed cases a decrease of the runoff coefficients in the rivers was detected (SEAM, 2011).

The main purpose of this research is to evaluate the effects of climate change on the degradation process of reinforced concrete in Paraguay. Thus, the scenarios RCP 4.5 and RCP 8.5 have been selected as the best and worst scenario respectively. In a recent study about climatic effects in Paraguay, an analysis was developed considering the climatic scenarios RCP 4.5 and RCP 8.5. The outcomes have shown that the whole country will be affected by climate change, highlighting the temperature change as the most significant being expected to increase between 3 °C and 4 °C (Pastén, 2017a, 2017b). Figure 2.6 (a) shows the expected temperature increase for the RCP 4.5 scenario until the middle of this century. In Figure 2.6 (b) the same parameter is shown but for the RCP 8.5 scenario.

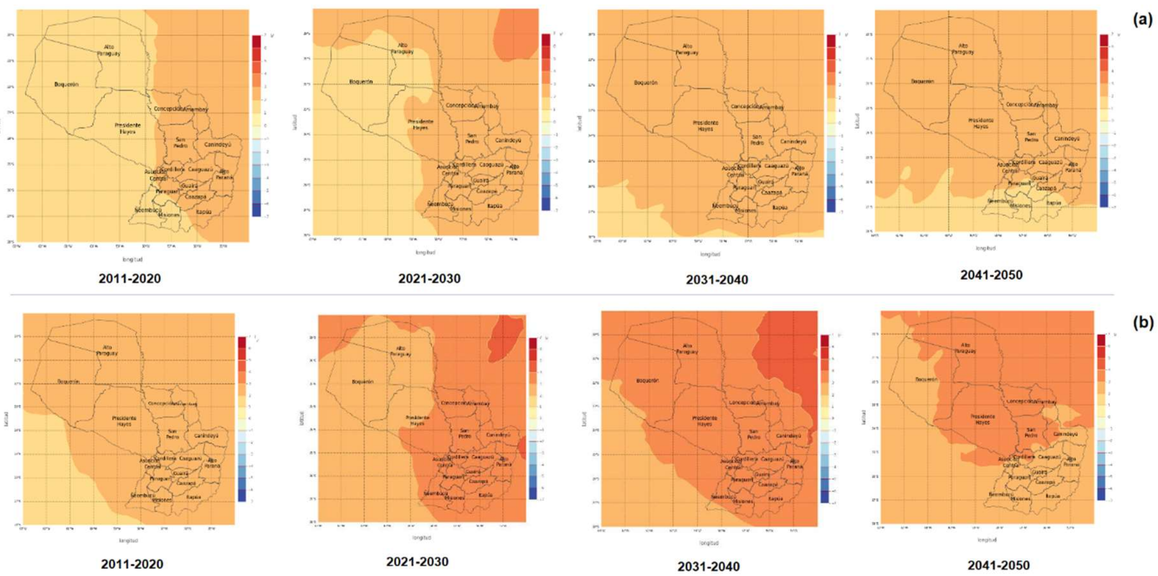


Figure 2.6 Expected temperature increment in Paraguay for the scenarios RCP 4.5 and RCP 8.5 (Pastén, 2017a, 2017b).

Since the country became aware of the phenomenon of climate change, more emphasis has been given to the development of mitigation strategies than adaptation. However, this approach has changed in recent years due to the creation of international financial funds to support adaptation in developing countries. In this way, Paraguay has prepared the CNCC, the last update of which was published in 2016. The objectives of this plan are to reduce vulnerability by fostering adaptation and resilience as well as integrating adaptation to climate change in current and future policies, activities and programs (CNCC, 2016).

The United Nations Development Program, in addition to highlighting the importance of installing the theme of climate change in all sectors, emphasises the importance of promoting the political commitment of the governments on all levels. In this way, it is intended to ensure compliance with the laws relating to climate change by the rulers and society. Furthermore, this program highlights the need to carry out coordinated work between public and private entities, setting common goals in pursuit of sustainable development (PNUD, 2007).

In summary, the temperature and rainfall would increase in almost all the Paraguayan territory, in addition to the occurrence of extreme weather events, with droughts more frequently in the western region from October to March. In contrast, there will be more floods in the region between April and September. Bearing this in mind, it is important to focus investments on the prevention of natural disasters, infrastructure improvements and rapid response measures to the impacts of climate change that currently occur in the country and those that could increase in the future (SEAM, 2015).

2.6 Effects in the Infrastructures

Research has shown that climate change and buildings are interrelated and that design and sustainable construction represent an important factor in the reduction of greenhouse gas emissions. On the other hand, buildings suffer the impact of climate change in different ways and cause different effects, which are linked to the location and climatic characteristics of each country. Several investigations have also been developed to analyse and evaluate different adaptation techniques for these effects in buildings. In this section, a review of some investigations is carried out in order to know the state of the art on this subject.

Several studies agree that climate change will cause new considerations and establish new conditions for the construction industry. Therefore, knowledge about the implications of climate change on infrastructure will be of great importance to the industry in the coming years. The climatic impacts influence the operation and maintenance costs, which implies that the building economics (i.e. life-cycle cost) must be approached considering the climate change. Thus, the study must be addressed within a dynamic analytical framework that explicitly allows for changes in information sets over time (Nordvik and Lisø, 2004). According to Talukdar, professionals in the civil construction industry must adapt future building designs to ensure that they have enough capacity and durability to deal with the consequences of climate change. For this reason, there is a need to review the standards and technical specifications of construction to analyse and consider the possible risks for the structural capacity; as for example, increases of loads of wind, snow, loads of rain, among others (Talukdar, 2013).

In the same way, Chalmers argues that buildings are at great risk of damage due to the expected impacts of climate change. That is, the construction sector needs to adopt policies and take aggressive and sustained measures in the design, construction and operation of buildings. For this, it is necessary to improve resilient infrastructure systems in order to greatly reduce the vulnerability of this sector (Chalmers, 2014). In a generalised way, Table 2.3 summarises the effects that each climate parameter has on the performance and durability of infrastructures.

Concrete is the predominant type of construction used in buildings, bridges, power plants, docks and other infrastructures throughout the world. Due to climate change, the risks of damage induced by carbonation in concrete can increase by more than 16% by the year 2100, which means that one in six structures will suffer additional and costly corrosion damage due to this phenomenon. It has also been found that, as a consequence of climate change, the corrosion rate could increase by 15% if the temperature increases by 2 °C (Stewart et al., 2011, 2012a). Among the changes that can occur in the environment due to the phenomenon of climate change, six effects can be indicated that will affect the durability of reinforced concrete structures. These effects are (Talukdar, 2013): (1) the increase in temperature, (2) the increase in atmospheric pollutants, (3) the variations in the annual amounts of precipitation, (4) the variations in the relative humidity levels, (5) the variations in the duration of the seasons, and (6) the occurrence of more frequent extreme weather events.

Gupta argues that the degradation of buildings caused by climatic effects occurs mostly due to temperature differences between day and night and between seasons; due to the particles transported by strong winds and air pollution. Furthermore, the movement of water in constituent elements of the building by capillarity and the abrasive effects of rainwater, salt and other chemicals present in the water can be considered as causatives of such degradation (Gupta, 2013).

Research developed for concrete infrastructures in China has determined, taking into account the same climatic parameters mentioned above, that the depth of carbonation could increase by 45% in reinforced concrete structures by the year 2100 (Peng and Stewart, 2016). On the other hand, Stewart *et al*. concluded that in some regions of Australia, the risks of damage induced by carbonation can increase even by more than 400% by the year 2100 (Stewart et al., 2011).

Table 2.3 Effects of climate change on infrastructures (Sabbioni et al., 2008; Snow and Prasad, 2011; European Commission, 2013).

Climatic Parameter	Effects in infrastructures
Increase of Temperature	Impacts on the external surfaces of buildings (deterioration of facades). It affects the thermal behaviour and health of the occupants. It can cause internal damage to masonry, stones and ceramics. It can also lead to fatigue and accelerated ageing of the material.
Intense rainfall	Greater intensity of runoff; problems of structural integrity; the overflow of sewers; ground slides; erosion of materials.
Intense and frequent cyclones	Greater tension in the components, coatings and supports of construction materials. Greater demands of wind load. It helps with the penetration of moisture into the pores of the materials.
Floods	Damage caused by water to buildings, contamination by sewage, soil and mud, increased the risk of soil subsidence, undermining foundations; erosion of materials can be expected.
Fire by drought	Total or partial damage by fire, damage by smoke and water.
Storms with hail	Impact damage (mainly roofs, gutters, windows) and subsequent penetration of rain/moisture.
Increase in moisture	Moulds, condensations, reduction of the thermal behaviour of buildings, physical changes in porous materials, crystallisation and dissolution of sales on surfaces, corrosion of metals, irregular humidity cycles that produce cracking/detachment of materials.

Bastidas-Arteaga *et al*. developed a study of the influence of climatic conditions and global warming on the chloride ingress into concrete structures. The results obtained indicate that climate change can produce significant reductions in the service life of concrete structures (Bastidas-Arteaga et al., 2010). Later, other studies showed that global warming could advance the time of failure by 31% or decrease the service life up to 15 years for moderate levels of aggressiveness (Bastidas-Arteaga et al., 2013).

Other research on the analysis of climate risk areas in Norway shows a dramatic expansion of the risk zone because of climate change. It could be verified that of the 615000 buildings that are currently in areas with high risk of degradation, by 2100 this number would increase to 2.4 million of buildings due to the expansion of the risk zone. Regarding the use of climate data for building design, the main challenge is not necessarily that the climate is changing, but rather that the local climate is receiving insufficient attention in the design and construction phases of current construction projects (Almås et al., 2011).

Regarding ceramic masonry structures, the vulnerability of this type of material is critical to several climate exposures. The precipitations and the frosts are the main climatic challenge to be considered in search of masonry structures of high performance. Regardless of the climate impact category considered, contractions and thermal movements dominate in this type of material within the most frequent category of damage. On the other hand, defects related to humidity are generally presented in at least 80% of cases of ceramic masonry (Kvande and Robert, 2009).

The transportation infrastructures would also suffer the impact of climate change. This phenomenon poses costly impacts regarding maintenance, repairs and loss of road connectivity. Developing countries will incur a higher relative cost of the impact of climate change on road infrastructure networks until 2100, but only from the 2020s. However, proactive adaptation measures can significantly reduce these impacts and costs (Schweikert et al., 2014). Although the uncertainty of climate behaviour in the future must be considered, which can increase the economic aspect of the infrastructure lifecycle (Nordvik and Lisø, 2004).

A study on the impact of climate change on the built heritage also proved that this global problem would have physical, social and cultural effects on cultural heritage. Furthermore, in conjunction with the possible socio-economic changes, a great impact on heritage conservation is foreseen (Kurtovic-Folic and Zivaljevic, 2014). Solar radiation and relative humidity are parameters that also affect historic buildings of cultural heritage. The direction of the wind during the rainy seasons tends to raise the humidity in the structure, which accelerates the process of degradation of the material (Martínez-Garrido et al., 2014).

Climatic indexes that allow a quantitative evaluation of material performance or degradation potential can be an important element in the development of adaptation measures. These climatic indexes are useful tools to face the future risks of climate change in different parts of the world. This and other indexes should be applied with quantified relationships established between the climatic impact and the behaviour of the material or performance of the building. They can also be used as a tool to assess changes in functional requirements or degradation rates due to global warming, incorporating data from regional and local climate change scenarios (Lisö et al., 2006).

Climate change and global warming, as well as greenhouse gas emissions, also affect hydraulic infrastructures such as dams, canals, aqueducts and hydroelectric stations. These are affected mainly by extreme events such as torrential rains and floods, drought seasons, high and low humidity, and solar radiation. Therefore, the risk of infrastructure failure due to the complex interaction between materials, climate and environmental factors affected by climate change cannot be ignored (Valdez et al., 2010).

It is evident that, even without the current uncertainties in the science of climate change and the potential impacts of climate change on buildings, the establishment of adequate control mechanisms to deal with these problems is a complex task. The long life of the buildings is perhaps the most palpable problem since most of the buildings built now must continue in service within the next 50-100 years. However, the most efficient way for adaptation is taken the proper actions before these buildings are built, i.e. during the design stage. Therefore, it is crucial to develop policies and strategies that reduce long-term risks for new buildings, encourage early adaptation where possible to existing buildings and, at least, adopt a precautionary approach to the uncertain risks of climate change (Camilleri et al., 2001).

2.7 Summary

Climate change is a problem that emerged and reached a palpable concern since the late twentieth century. It is a phenomenon that could be considered as the greatest threat to humanity since its emergence, considering the notable alterations in the natural behaviour of planet Earth because of anthropogenic activities. To face this problem, there are many negotiations, agreements and projects prosecuted by the main world leaders. However, as has been seen in recent times, different state policies often generate disagreements that make it difficult to make decisions that benefit the "repair" of the planet to this serious problem. For this reason, it is important to recognise that, for this purpose, international commitment and cooperation are necessary to achieve the expected results.

The main objective of the international agreements developed on climate change is not to allow the average global surface temperature to increase by 2 °C by the end of the 21st-century. Does not to meet this goal would greatly increase the risk of shortages in water supply, food, and natural disasters. For this, the proposed strategy consists of reducing greenhouse gas emissions at 15-30% by 2020, and at 60-80% by 2050.

Climate change is changing the economy, health and communities in different ways. Scientists warn that if measures are not taken substantially now, the results are likely to be disastrous. If the Earth warms, the immediate effects would be the rise in sea level, which would cause flooding. In other regions, the droughts would eliminate lakes and rivers, which would affect food production, the existence of fauna and flora. On the other hand, natural disasters such as tornadoes and hurricanes would intensify in intensity and frequency.

Perhaps one of the most concerning conclusions regarding the climatic phenomenon is given by the fact that the carbon dioxide accumulated in the earth's atmosphere cannot be reduced to acceptable levels until within a few centuries. This conclusion is made even considering that greenhouse gas emissions were entirely and immediately reduced. Then, this situation suggests that the best strategy to deal with this problem involves adaptation measures rather than mitigation measures. From the engineering and constructions approach, this strategy must be proposed dynamically considering the accelerated change in the expected climate according to the IPCC scenarios.

Many studies have been developed regarding the adaptation of infrastructures to climatic demands. However, there is still a long way to go to achieve a correct understanding and awareness of the problem that climate change implies in the durability of structures. It is necessary to recognise that this new problem must be approached from a multidisciplinary point of view so that technicians and construction professionals should look for new tools of knowledge that allow them to understand and solve problems. Therefore, the problem of climate change currently forces professionals to acquire more specialised knowledge that may not have been previously considered.

Maintenance and repair of existing structures, as well as new constructions, present the challenge of dealing with the uncertainty of climatic phenomena that in the last century did not require further consideration. In recent years, it could be verified that the materials present a particular vulnerability to aggressive environments, affecting the service life of the structures, affecting the economic aspect of construction projects. Nowadays it is necessary to consider aspects of the project such as maintenance, inspection and sustainable rehabilitation dynamically, considering the variability in the assumptions referring to the environment in which the infrastructures are constituted.

Although civil engineering historically has covered its knowledge based on physical and mechanical stresses to establish the durability conditions of a structure, the effect of climate change has forced to expand knowledge in a deeper way towards the environmental and chemical stresses that influence directly on the integrity of structures and infrastructures worldwide. The main objective of this chapter was to establish the basic notions of climate change, not only in social and economic aspects but also in the durability of the structures. Moreover, in Chapter 4 of this research, a comprehensive analysis of the concrete structures degradation by corrosion is addressed considering the last climate scenarios of the IPCC.

CHAPTER 3

Degradation of Reinforced Concrete

Chapter 3

3 DEGRADATION OF REINFORCED CONCRETE

3.1 Introduction

Concrete is a composite material which can support compressive stresses. However, it is vulnerable to cracking due to other types of mechanical stresses such as bending, tensile, torsion, shear, among others. Due to this feature, there is a need to work with a composite material, i.e. the reinforced concrete. This material combines the properties of concrete with the properties of the structural steel to fulfil the demands required in the construction of civil infrastructures such as bridges, buildings, skyscrapers, tunnels, etc. (Aguirre and Gutiérrez, 2013). Reinforced concrete (RC) is a durable material which was globally adapted as a constructive element from the beginning of the 20^{th}-century with the clear idea that, with proper structural design, the infrastructure could perform satisfactorily during its service life. However, this notion was disproved over time (Vassie, 1991; Daly, 2000). Under a historical perspective, 1945 was the year of a shift in the building materials application where most of the structural elements applied until then (steel and timber) was replaced for the use of reinforced concrete (Dias, 2013). The idea of introducing steel as reinforcement of concrete structural elements arose precisely in the mid-nineteenth century when in 1867, Joseph Monier patented the technique that later would allow the versatility of reinforced concrete worldwide (Hudson, 1972).

The join between concrete and reinforcing steel is a mutually beneficial link since the layer of concrete cover between the steel surface and the external environment extends its service life by acting as a barrier against the necessary or conducive substances to corrosion. On the other hand, the chemical environment within the concrete pores does not promote corrosion (under optimal conditions) thus providing additional protection (Dyer, 2014). Within the structural design, specific parameters are considered, such as the resistance of the material, the loads applied, the morphology of the elements, among others. These parameters define the principles of quality and performance characteristics of the designed structure. Nonetheless, the structural performance is directly related to the surrounding environment of the building, which plays an essential role in the definition of the degradation mechanisms and their corresponding effects. Thus, such interaction between the environmental conditions and the structure determines the durability patterns of the building.

Concerning the durability of this type of structure, Muñoz and Mendoza define it as the capacity of reinforced concrete structures to keep their physical and chemical conditions unchanged during their service life when they are subject to the degradation of their material by different effects of loads and solicitations, which are foreseen in its structural design (Muñoz and Mendoza, 2012). As an example, durability is one of the criteria on which the need to limit the crack opening is based. To this end, the maximum crack opening values are established in different standards according to the environmental exposure class and depending on the type of concrete used (reinforced or prestressed).

In the literature, it is possible to find a set of references through organisations and projects that provide standards and valuable information on the durability characteristics of concrete structures. The Committee Euro-International of Betón (*CEB*) and the International Federation for Structural Concrete (Fédération Internationale du Béton, *fib*) have developed numerous studies related to the topic addressed in this section (CEB, 1989; fib, 1999), as well as the well-known *DuraCrete Project* that emerged at the end of the last century (DuraCrete, 1998, 2000). In these works, it was demonstrated that it is not enough the consideration of parameters related to materials and execution of a project to ensure the structural durability of a building; instead, it is also necessary to consider the environmental conditions to which the structure will be exposed during its service life. On the other hand, this conception of durability should not be taken into account only in the design stages but also during the evaluation of its conservation state during its life cycle, applying a correct modelling of the degradation process and the accurate prediction of the future performance of the building.

Several factors may be considered as the causes of the degradation of reinforced concrete materials throughout their service life. Generally, these are usually classified as physical causes (cracking and wear) or chemical causes (carbonation and sulphation), which are affected by both existing conditions during construction and environmental conditions. At the time of construction, it is essential to consider the aspect of the ambient temperature, which should be between the values of 5 °C and 32 °C (Ahmad, 2006).

In a statistical study carried out in several European countries, it was possible to determine that the structural elements most affected by some pathology are those exposed to bending stresses, where the outstanding manifestation corresponds to the cracking which is present in 59.2% of the cases analysed. Regarding the period of appearance of the cracks, in 75.9% of the cases, these manifested in the first ten years of the structure's service life. The degradation mechanism of the more than 17000 cases of concrete structures surveyed have varied about the constructive and environmental characteristics of each country, although cracking, humidity and differential settlements were the most prominent anomalies in general (Vieitez and Ramirez, 1984).

One of the most significant pathologies that characterise concrete structures is given by the presence of cracks and fissures that directly affect the functionality and durability of the building. Similarly to the above, there are several causes of this anomaly. On the one hand, cracks of chemical origin, related to the hydration of the cement and/or the oxidation of the reinforcement, while that of physical origin is caused by changes of expansion and contraction of the element, which exceed the limit values of resistance leading to the cracking of the material (Toirac, 2004).

It is important to bear in mind that there is no material that when applied or used as a constructive material in a structure may be considered as intrinsically durable. This is because their properties and characteristics tend to vary according to the interaction between the microstructure of the material and the environment. This interaction affects specific mechanical alterations such as the capacity and performance of the material, causing what is known as degradation. Currently, it is possible to associate the low durability of the structures with the lack of knowledge regarding the features of the material related to its service life (Roque and Moreno, 2005). The understanding of the quality of construction material or, at least, a range of quality defines the comprehension of the future evolution of its degradation (Sarja et al., 2005).

Although the emphasis concerning the degradation mechanism of concrete in this research is oriented towards the reinforcement corrosion, this chapter presents a description of the durability factors that affect the concrete structures in general. Therefore, the main degradation mechanisms related to the interaction mentioned above between the structure and the environment are described. Herein it is considered the factors that affect the advance of the deterioration throughout the service life of structures and the critical phase of the conservation state. It should be mentioned that the description is merely a review of the concepts, and for further details, it can be consulted the cited references.

3.2 Durability of Concrete Structures: Influential factors

The concept of durability in buildings comprises a paramount and determining factor which is directly related to the degradation patterns of a structural or non-structural element, and which is capturing the consideration and concern of construction professional’s whit great frequency increasingly. According to ISO 15686-1, durability is defined as the capability of a building or its parts to perform its required function over a specified period of time under the influence of the agents anticipated in service (ISO 15686-1, 2000).

Another concept that is often confused with durability is the concept of a building's service life. The service life of a building can be understood as the quantification of durability, that is, by using this concept the importance of designing and constructing projects can be established. Likewise, the service life of the buildings can be guaranteed and extended by taking into account the durability criteria of the constitutive material (Da Silva, 2001). The factors that affect the service life can vary, not only from one building to another but even within a given building. For example, the quality of the roof structure in a building may vary if different subcontractors were responsible for them. Also, different construction elements can receive different degrees of maintenance, depending on their accessibility and capacity of inspection (Dias, 2013).

At the end of the 19th-century, buildings were considered as long-term structures in professional practice, and the long-term consequences were not automatically taken into consideration. If the idea of the life cycle of buildings is old, the consideration of the end of the service life of a building is historically quite new and remains mostly abstract yet (Kohler and Hassler, 2002). It was not until the 1990s that the first works concerning the durability and service life of the construction elements of buildings began to be developed worldwide. From that moment on, the different initiatives and research carried out in this field acquired relevant importance and a predominant approach towards the prediction of the service life in the construction sector (Ortega, 2012).

Considering the complexity of the nature of environmental effects on concrete structures, it is known that their performance cannot be improved only in the design of the characteristics of the materials. Therefore, it is necessary to involve the architectural elements, execution processes, structural design, and inspection and maintenance procedures, including preventive maintenance. Therefore, it is important that the stakeholders have the necessary knowledge related to the degradation processes of the materials and the parameters that govern them. This last requirement comprises a precondition for the ability to make correct decisions at the precise time to attain the required durability (CEB, 1989).

Carrying out frequent inspections and maintenance actions are essential for proper design and management of structures under the time-dependent deterioration process. It is paramount the application of life-cycle design and management concepts in selecting the material and structural attributes to minimise the total life-cycle cost and extend the durability of structures. Among the main drivers of economic growth and sustainable development of the countries are the civil infrastructure systems, which are the backbone of modern society. Therefore, consolidating and improving criteria, methods and procedures must be a strategic priority to protect, maintain and enhance the safety, durability, reliability and recovery capacity of critical infrastructure systems in conditions of uncertainty (Biondini and Frangopol, 2016). The detailed design of the capabilities of reinforced concrete structures is only the beginning of the process that ensures its durability. As mentioned above, several factors contribute to the structure durability whose combination significantly affects the structural performance of the building during its service life. This performance of a structure throughout its service life is depicted schematically in Figure 3.1.

The stage of execution of a structure is one of the most critical phases of a project. The shortcoming in the durability of the structure is often given by the workforce careless who cannot fail in the correct compaction of the material, the curing and the proper thickness of the cover. These are just some of the factors that considerably affect the quality of the structure and its behaviour under the stresses and the surrounding environment. Considering the environmental effects, the degradation mechanisms are often favoured under certain environmental conditions, which disturbs both the appearance and the functionality of the structure.

A clear example of the incidence of certain factors in the degradation is the formation of cracks. Cracks allow the quick advance of the degradation process causing the decrease of safety and reliability of the structure for the building's users, as well as the reduction of resistance and the loss of functionality (Quillin, 2001). Regarding this perception, a research carried out to evaluate the state of conservation of social buildings showed that the technicians were more concerned about the technical consequences of the defects. Meanwhile, the residents of the building were more sensitive to living conditions and the comfort requirements (Rodrigues et al., 2011).

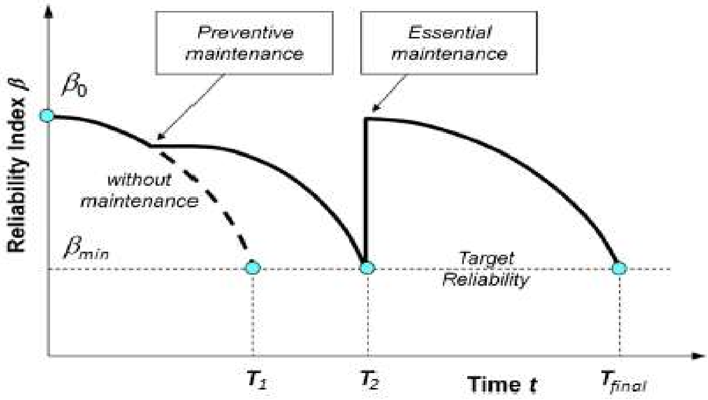


Figure 3.1 Performance of RC structures over its service life (Biondini and Frangopol, 2016).

Although degradation is often caused by anomalies that involve effects that do not go beyond those of an aesthetic nature, if this degradation is not treated on time, it can lead to a structural safety case. According to the standard of the Code on Structural Concrete (EHE, for its Spanish acronyms), in the article n° 5, it defines the structural safety as the reduction to acceptable limits of the risk referred to the structure concerning to an inadequate mechanical behaviour in front of the foreseeable actions and influences to which it may be subject during its construction and expected use, considering its entire service life (EHE-08, 2008). There are many factors (environmental conditions, quality of maintenance, the frequency of inspections, etc.) that generate impact within the structural safety of a building. However, it becomes a difficult task to accurately estimate the influence that each of these factors has on the performance of a building. Therefore, it is important to develop degradation models for the reliability analysis of a structure, which considers the potential correlation of degradation by loads (Wang et al., 2017).

Another influential parameter in the service life and degradation of structures is the term of reliability, which has been depicted in Figure 3.1 by the notation β. Reliability is defined as the probability that an element or infrastructure will perform its expected function for a specific period of time, under specified conditions. In contrast to safety, the reliability is measurable, that is, quantifiable. Therefore, the lack of reliability implies that a service condition, under a certain probability, will not be fulfilled satisfactorily, for instance, the condition by which a structure will not collapse or reinforcing bars do not rust prematurely (Schneider, 1997).

The degradation patterns often show a phenomenon of entropy between the degradation mechanisms. This phenomenon occurs as a result of these patterns are often disturbed by certain failure conditions, such as the presence of small cracks distributed on the surface allowing the penetration of the deterioration agents. In other words, it is often possible to observe different degradation patterns in similar facades, or even, the same facade with varying patterns of degradation in some regions of the same. This may be related to the nature of the failure that affects each part of the facade, the extent of the degradation and the severity of the degradation (Gaspar and de Brito, 2005).

To reduce the undesirable results that may affect the durability of a structure and the structural performance, it is meaningful to perform monitoring measurements in the execution stages of construction and during its service life. These controls, which must be considered during the design phase and implemented during the execution phase, could guarantee the quality of the structure. That is, the inspections and monitoring carried out periodically will allow detecting the defects in the early stages of their development and thus achieve the appropriate treatment to ensure durability.

A methodology generally used to determine the degradation and durability of a construction system are the state diagrams. The degradation process can be represented by a sequence of stages of increased deterioration, which finally lead to the failure of the system. A state diagram represents a simple failure repair process for this case, where if the maintenance is not carried out in a degradation stage, the system goes through a next more deteriorated state level than the previous one, where later or early failure status is reached (Welte, 2009).

The study of the building's durability must be addressed from the conception of the project itself. In the design stage must be defined the degree of environmental aggressiveness to which the structure will be exposed, as well as an exposure class for each element that comprises it. European regulations indicate the need to prioritise the durability of buildings, ensuring the safety of people and the protection of the environment. For this, buildings must meet specific requirements such as to be safe and functional; it must be able to withstand fire actions; meet minimum requirements of hygiene, health and protection of the environment (Muñoz and Mendoza, 2012).

3.3 Degradation Mechanisms

Although reinforced concrete structures have greater durability compared to other types of structures, they are not exempt from being affected by several mechanisms that govern their deterioration process, which is commonly classified as physical processes and chemical reactions. Among the physical processes that affect the durability of concrete include surface wear, cracking in pores, and exposure to temperature extremes such as during frost action or fire. The chemical effects include leaching of the cement paste by acidic solutions, and expansive reactions involving sulphate attack, alkali-aggregate reaction, and corrosion of the reinforcement (Mehta and Monteiro, 2006). These degradation agents are responsible for causing considerable damage to the structure, or also in the reinforcements through corrosion processes, a failure that later triggers in anomalies such as cracking and spalling of concrete cover. These degradation mechanisms are described in detail in other several studies found in the literature (Bentur et al., 1997; Torfs and Van Grieken, 1997; Balaras et al., 2005; Broomfield, 2007; Wang et al., 2017).

Regarding the environmental effects on the durability of the construction elements, it is necessary to recognise that anthropogenic activities have a direct impact on the process, increasing the concentration levels of harmful gases for the "health" of the building. According to Balaras *et al.*, the pollution caused by humans has considerably increased the degradation rate of buildings. Corrosion and erosion of building materials and coatings are mainly due to the acid in the environment, especially from SO_2, humidity, rain, dust and smoke (Balaras et al., 2005). Another similar interpretation on the degradation mechanisms of buildings declares that the structures begin to deteriorate immediately after their construction. The degradation of the components of a building is a normal consequence of the ageing process, having a combination of factors that influence this process, such as the quality of the building, climatic conditions, lack of maintenance, etc. (Sarja et al., 2005).

The degradation mechanisms explain the way in which a constructive element decreases its performance throughout its service life. Nonetheless, to represent this process in a mathematical perspective, it is common to resort to the so-called degradation models of the construction elements. Usually, these models begin with a deterministic description of the degradation processes using differential equations or a system of differential equations. Subsequently, randomness can be introduced into the model using probabilistic distributions to describe the variability of parameters such as rates, constants or properties of the material (Yánez et al., 2003). It is recognised in most countries the needs for more reliable models of the composition and dynamic of the building sector. However, there is insufficient statistical data upon which to base them (Kohler and Hassler, 2002). Despite this problem, several models are currently being developed regarding the degradation mechanism of the elements of a building (Soh et al., 2003; J. Mullard and Stewart, 2009; Talukdar et al., 2012a; Li et al., 2016; Xu and Chen, 2016)

Even though in the literature it is possible to find a diversity of mathematical models that describe the degradation mechanisms of structures, being classified according to their typologies such as statistical degradation models, RILEM TC130 CSL models, and reference structure models. Statistical degradation models are based on physical and chemical laws of thermodynamics, which include parameters that must be determined with specific laboratories or field studies. RILEM models are based on parameters which are available from a combination of concrete designs and structural design. Lastly, the reference structure models are based on a statistical treatment of the degradation process and the condition of a part of real structures taken as a reference, which is in similar situations and has similar durability properties with the whole structure. This last category can be combined with the Markov Chain method in the classification and statistical control of the structural conditions (Sarja et al., 2005).

In the 1930s, the "*Transport and Road Research Laboratory (TRRL)*" was created in England. This organisation has been developing a series of research and works related to construction methods since its inception. Currently, this organisation has been operating privately for 20 years, although it continues in the same line of research. In 1980, the TRRL developed a report concerning the methods for conducting tests to assess corrosion. In such a report it was mentioned that both in the United States and Canada, the corrosion in reinforcement was one of the mechanisms most relevant cause in the degradation of bridges. Likewise, a warning is made in the report about the corrosion problem that could affect the infrastructure of the United Kingdom in the future (Vassie, 1980, 1991).

According to Gaspar & De Brito, the mathematical function of degradation can be represented by two variables as is shown in Figure 3.2: the loss of performance as the dependent variable and time as an independent variable. In turn, this function can be linear (e.g. the action of the wind) or discrete (e.g. the mechanical actions). Regardless, it is convenient to point out that, for the real service considerations, the loss of performance occurs due to a combination of both hypotheses (Gaspar and de Brito, 2005).

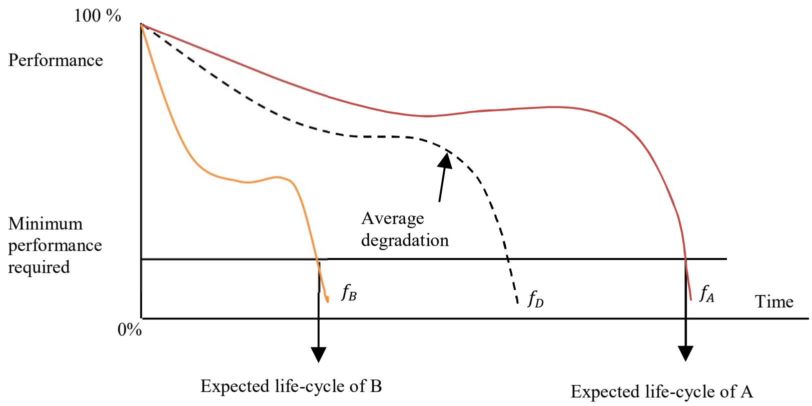


Figure 3.2 Degradation function for a constructive element of a building concerning its expected life-cycle (Gaspar and de Brito, 2005).

If the degradation is caused by several mechanisms, the Weibull distribution can be a good choice to design a degradation system and the inspection and maintenance strategies. Degradation systems and inspection and maintenance strategies are two parallel and separate processes. These processes are connected to each other through new information about the condition of the construction system, which is obtained through inspections or after the detection of an apparent failure. In these terms, inspection intervals are rather deterministic than exponentially distributed. If the inspection intervals are not modelled as deterministic numbers, the gamma distribution or Weibull distribution may be an appropriate choice (Welte, 2009). Nonetheless, the normal distribution is frequently used in studies of engineering to describe random variables with a continuous probability distribution whose real distribution is unknown (Xu et al., 2015).

Below is a brief description of the various more considerable degradation mechanisms in reinforced concrete, to later cover more specifically the corrosion of reinforcements. Furthermore, the parameters that influence corrosion, as well as the identification of the critical structural states that govern its structural analysis, are considered.

3.3.1 Sulphates Attack

The phenomenon of sulphate attack occurs due to the ingress of dissolved sulphate ions into concrete, which subsequently experiences reactions with the hardened cement. Depending on the exposure conditions, some different reactions are presented in the structure, whose influence on concrete properties results either from expansion and cracking or loss of strength and integrity (Dyer, 2014). The main chemical equations that govern this degradation mechanism are the following (Ahmad, 2006):

$$Ca(OH)_2 + Na_2SO_4.10H_2O \rightarrow CaSO_4.2H_2O + 2NaOH + 8H_2O \tag{3.1}$$

$$\begin{aligned} 3CaO.Al_2O_3.12H_2O + 3(Na_2SO_4.10H_2O) \\ \rightarrow 3CaO.Al_2O_3.3CaSO_4.31H_2O + 2Al(OH)_3 + 6NaOH + 17H_2O \end{aligned} \tag{3.2}$$

There are some factors that most significantly influence the ability of concrete to resist degradation by sulphates attack: the concentration of sulphate ions in the solution, the permeability properties of the material, the level of the water table and its seasonal variation, the flow of groundwater and soil porosity and the composition of the cement paste. Therefore, the measures necessary to avoid the sulphate attack depend mainly on the cations associated with the sulphate of the environment in which the structure is developed, in addition to other environmental factors (Mehta and Monteiro, 2006; Dyer, 2014).

To counteract this degradation mechanism in the concrete structures, it is recommended to use denser concrete with the capacity to resist sulphates such as Portland cements type I and IV, pozzolanic cements, etc. (Porrero et al., 2009). Another way of protection would be to use waterproof cement, applying protective coatings of the bituminous type on the surface of the concrete (Mullheron, 2000).

3.3.2 Abrasion and Crystallisation

In addition to the anomalies caused by chemical reactions that can affect the concrete, such as the sulphate attack exposed above, there are two processes that commonly occur in the structures and that are related to cement paste and aggregates. These processes are the abrasion and the crystallisation of the concrete surface. In general, appropriate curing practices are proven to promote the abrasion resistance of concrete significantly. Moreover, the abrasion resistance of the concrete structure is highly dependent on the microhardness and pore structure of the surface zone (Ardalan et al., 2017).

The abrasion is a phenomenon that describes the process of disintegration of cement paste on the surface caused by aggressive agents that are carried by the movement of water, which is often seen in channelling elements. The damage index is determined from the flow of water where a higher value reflects a progressive attack towards the interior of the structure. On the other hand, crystallisation is a process in which the reaction occurs within the pores of the cement paste through the ingress of these aggressive agents. As a consequence of crystallisation, cracks in the concrete surface generated by internal stresses generally is observed, accelerating the degradation process (Porrero et al., 2009).

3.3.3 Temperature variations - Freeze/Thaw Cycle

A typical degradation mechanism in concrete structures located in cold regions is the variation of temperature generated by the freeze/thaw cycles, which involves significant economic importance for the maintenance management of these structures. This phenomenon consists of physical damage caused by the iteration of the freezing and thawing of the water contained inside the concrete pores, which was formed due to the air occluded generated during the curing stage. Consequently, the damages that occur by the action of frost depend mainly on the degree of water saturation into the pore system of the concrete (Ahmad, 2006; Mehta and Monteiro, 2006; Porrero et al., 2009).

This degradation mechanism is associated with cold regions where the mean temperature is around 0 °C (freezing temperature of the water). When the water contained in the pores freezes, it expands generating internal tensions. Whether this process occurs repeatedly, then the formation of micro-cracks in the concrete is caused. During a freezing cycle, the water expands its volume by around 9%, causing expansive pressures that produce a gradual scaling, cracking and subsequently, the spalling of concrete cover (White et al., 1992).

There are two ways to handle this degradation mechanism in concrete. On the one hand, considering that the water contained in the humidity is the one that, when it freezes, causes internal tensions, it is advisable to keep the structure impermeable. A dry concrete usually is not affected by freezing. Nevertheless, it is not possible to ensure that a structure is not exposed to humidity or cold climates throughout the whole service life. Therefore, preventive protection of the structure with adequate drainage of exposed concrete surfaces is required (Mullheron, 2000).

Another solution to this problem is the commonly applied technique of voluntary incorporation of occluded air into the concrete mix by special additives. This additive may generate a series of pores during the setting and curing of concrete, reaching between 3% and 7% of the volume of the concrete. This occluded air allows the increase of the resistance capacity of the concrete in front of the internal tensions through to the generalised distribution of the pores inside the mixture (Porrero et al., 2009).

3.3.4 Alkali-Silica Reaction

The proper understanding of the expansion process as a consequence of the alkali-silica reaction is essential for the evaluation of the susceptibility of a concrete structure to degradation. Also, it allows adequate planning and implementation of preventive measures. A chemical reaction involving alkali and hydroxyls ions from Portland cement paste and certain reactive siliceous minerals can lead to the expansion and cracking of concrete. These reactive siliceous minerals are often present in the aggregate of the concrete. Because of the reaction, to the loss of strength and elastic modulus is produced. Alkali-silica reactions cause great problems since their degradation rate is relatively slower than other mechanisms and the first symptoms of cracking of the structure may take several years to appear. Hence, this mechanism makes it difficult to take actions to stop the anomaly in a stage in which the damage presented is still not severe (Prezzi et al., 1997; Mehta and Monteiro, 2006; Dyer, 2014).

Several studies have been carried out in order to describe this procedure and to formulate techniques that allow acting in a timely manner. In 1997, Prezzi et al. developed a theoretical model to explain the volume change behaviour of mortar blocks containing a reactive aggregate. In their study, they determined that the expansion of the gels formed by the reaction is caused by the electrical forces of double-layer repulsion (Prezzi et al., 1997). Then, Bažant & Steffens have developed a mathematical model that allows establishing a quantitative prediction of the process of this mechanism. In its study, differential equations are formulated numerically by combining the kinetics of the chemical reaction with the associated diffusion process and the fracture mechanism of the damage process (Bažant and Steffens, 2000). Recently, more detailed studies have been carried out where it has been determined that the effect of alkali dosing is of great importance. Changes in weight, compressive strength, pore structure and chemistry of the pore solution were investigated from a new material known as alkali-activated slag. This new material has shown relatively better performance against alkali-silica reaction concerning the traditional Portland cement structure (Shi et al., 2017).

3.3.5 Carbonation and Chloride penetration

The carbonation and the chlorides penetration in concrete are often two mechanisms that resemble each other, but that differ according to the reaction process that defines them. Carbonation is a process that consists of a primarily superficial degradation of the concrete. In this process, the calcium hydroxide coming from the hydration of the cement reacts with the atmospheric carbon dioxide forming calcium carbonate. As a result of this process, a retraction is generated under certain humidity conditions which generate micro-cracks, which facilitate the ingress of carbon dioxide and other contaminating materials into the structure (Porrero et al., 2009). On the other hand, this reaction leads to the pH decrease in the concrete, which when dropping down to values lower than 9, the concrete remains completely contaminated by carbonation. Moreover, once the carbonation front reaches the reinforcement, the passive protective layer disappears initiating the corrosion process (Concrete Society, 2000).

According to *fib*, the corrosion induced by the chlorides penetration is one of the degradation mechanisms that have greater severity and amplitude in reinforced concrete structures worldwide (fib, 1999). Due to its chemical composition, this process is generally governed by the presence of salts, which is the main reason for which it is a common anomaly in marine environments. However, in contrast to carbonation, chloride-induced degradation can occur both by the interaction of the structure with the environment and by the action of the chlorides within the concrete. The degradation by chlorides is often given by the addition of calcium chloride accelerators (widely used until the mid-1970s), use of seawater for the mixing of concrete, and the use of contaminated aggregates (Broomfield, 2007). Both carbonation and chloride penetration are discussed in more detail in the following sections of this chapter.

3.4 The Corrosion in Concrete Structures

Regarding reinforced concrete structures, it is usual to associate corrosion as the common cause of its degradation. Corrosion has been the subject of scientific study for more than 150 years. It is a phenomenon of natural origin commonly defined as the deterioration of a material (usually a metal), or its properties, due to a reaction with its environment. Corrosion has a huge economic and environmental impact on practically all aspects of the global infrastructure, namely roads, bridges and buildings, oil and gas systems, chemical processing and water and wastewater systems. As already mentioned, it is estimated that the annual cost of corrosion in infrastructure exceeds USD 1.8 trillion worldwide, which translates into 3% to 4% of the Gross Domestic Product (GDP) of industrialised countries.

Meanwhile, in Western Europe alone, repair activities for corrosion-induced damage comprise a cost of 5 billion euros annually. In other studies, it is reported that the expenditures associated with the maintenance and repair of RC structures damaged by corrosion are estimated at around USD 100 billion per annum worldwide. Nonetheless, studies estimate that 25% to 30% of annual corrosion costs could be saved if optimal practices in the treatment of corrosion are employed (Schmitt et al., 2009; Ekolu, 2016; Taffese and Sistonen, 2017; Xi and Yang, 2017).

There are several causes of degradation in reinforced concrete materials throughout their service life. Due to the high alkalinity of the cement paste, the main constituent material of the concrete, it is possible to protect the reinforcement against corrosion. This cement paste surrounds the rebar and isolates it from the harmful agents of the surrounding environment. In this way, the hydroxide or oxide layer is stabilised, and the corrosion rate process is reduced (Ahmad, 2006).

The corrosion process within reinforced concrete structures can be easily analysed and understood from the chemical point of view. In concrete, corrosion begins with the rupture of the protective layer of steel due to the loss of the alkalinity of the cement paste, which is caused by the penetration of chlorides and gases that react with water moisture. When the reinforcement initiates the corrosion process, it produces iron electrons on the surface which react with water and gases through a cathodic reaction. These reactions form an alkaline layer (rust) that seeks to protect the rebar from the contamination of the cement paste. If this contaminated paste is not repaired, then the reactions are repeated several times, increasing the volume of the alkaline layer, which translates into cracks and spalling of the cover, gradually degrading the structure. Further details regarding the mathematical expression of this chemical reaction of corrosion may be found in (Broomfield, 2007). In Figure 3.3, the corrosion process is depicted schematically.

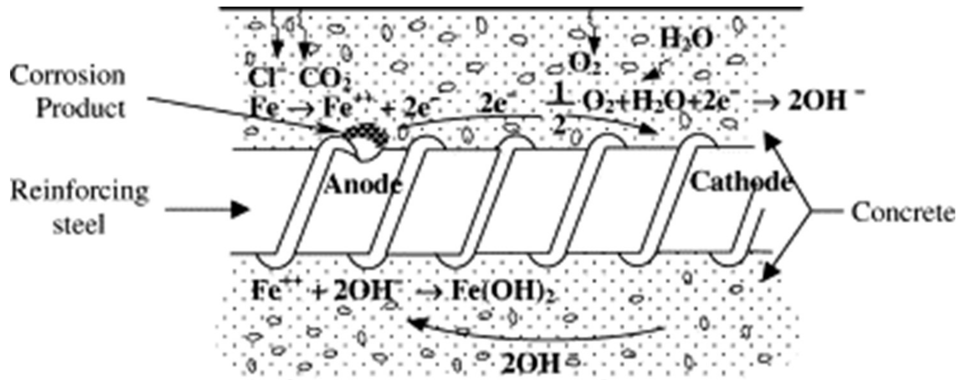


Figure 3.3 The corrosion process in RC structures (Ahmad, 2003).

Numerous materials, not just reinforced concrete, have the susceptibility to corrode when exposed to harmful agents and environments. However, the common factor among these elements is the presence of metals in their composition. So, more generically, it is common to name the process as metallic corrosion. Considering this generality, Dyer describes the corrosion process as a basic oxidation reaction involving molecules of some metal (M) with the oxygen molecules (O), expressing it more simply with the following equation (Dyer, 2014):

$$2M + O_2 \rightarrow 2MO \tag{3.3}$$

The state of energy of metal in the electrolytic solution is influenced on the one hand by the pH of the solution, and on the other hand, by the particular characteristics of the metal. Consequently, Marcell Pourbaix developed a simple way to represent these reactions in a diagram known as the Pourbaix diagram, in which the pH and the electrochemical potential are taken as coordinates of the diagram, as shown in Figure 3.4. Thus, the diagram defines the areas of a ferrous material as a function of corrosion as follows: passivation zone, immunity zone and corrosion zone (del Valle et al., 2001).

In essence, every structure composed of reinforced concrete remains vulnerable to corrosion during its whole service life. The first visible manifestation is the surface discolouration of the structure in a reddish-brown tone representing the iron ions detached from the reinforcement. The increase in volume given by the rust formation on the surface of the rebar generates expansive stresses, which when exceeding the admissible tension of the concrete cause its cracking. In turn, this causes the reduction of the strength of the structure that, in critical cases, could trigger a failure (Ahmad, 2006).

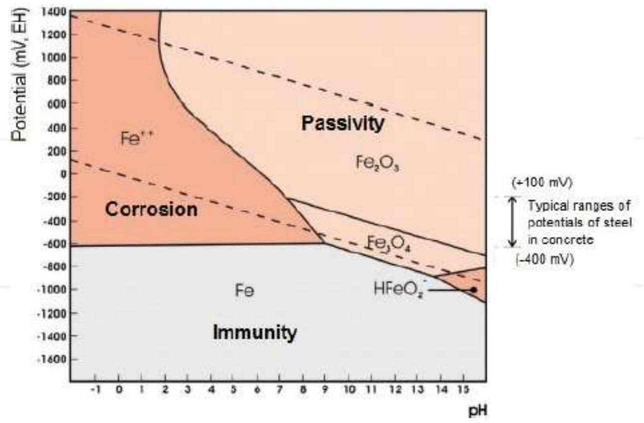


Figure 3.4 Pourbaix diagram for corrosion (Rivetti et al., 2018).

3.4.1 Carbonation

Carbonation is the chemical reaction between the hydration products of the cement in the concrete and the carbon dioxide (CO_2) of the atmosphere that leads to a decrease in the alkalinity of the concrete, leaving the steel reinforcement vulnerable to corrosion. These chemical reactions that occur during the carbonation process can be fragmented into two stages. First, the dissolution of CO_2 in water. Then, the reaction of the dissolved product with the products formed by the hydration of the cement paste inside the concrete structure (Dyer, 2014). One of the major concerns regarding this degradation mechanism comes from the tangible increase in the volume of CO_2 in the environment. According to the National Oceanic & Atmospheric Administration (NOAA) and the Earth System Research Laboratory (ESRL), the concentration of CO_2 has reached the value of 411.24 particles per million (ppm) by May 2018 (NOAA-ESRL, 2018).

Under a more chemical approach, Ahmad describes the carbonation process as the conversion of carbon dioxide into carbonic acid (H_2CO_3) in the presence of moisture (H_2O), as can be seen in Equation 3.4. This chemical then reacts with the calcium hydroxide ($Ca(OH)_2$) to finally form the calcium carbonate ($CaCO_3$), which is shown in Equation 3.5. As already mentioned above, this reaction causes the reduction of alkalinity due to the removal of the hydroxyl ions, which facilitates the initiation of corrosion in the reinforcement. The reactions that govern during the carbonation process can be formulated as follows (Ahmad, 2006):

$$CO_2 + H_2O \rightarrow H_2CO_3 \tag{3.4}$$
$$H_2CO_3 + Ca(OH)_2 \rightarrow CaCO_3 + 2H_2O \tag{3.5}$$

On the other hand, within the research developed by Ta *et al.*, carbonation is described as a process that can be considered in three stages as shown in Figure 3.5. The first zone, which corresponds to the external surface of the structural element, is considered to be completely carbonated having a pH approximately equal to 9 or less, with a total loss of the alkalinity of the cement. Next to the first zone, there is a transition zone called the carbonation front, which covers a gap that ranges from the maximum carbonation index to the minimum index. Finally, in the internal zone of the element is the not contaminated sector by carbonation where the pH is still maintained above 13 (Ta et al., 2016).

The carbonation process is governed by three parameters related to the cover, the structure and the alkalinity in the pores of the cement paste. Carbonation occurs more quickly if the cover thickness is very low which decreases the time of corrosion onset. On the other hand, despite the cover thickness to be higher, if the structure of the pores is open, it will allow a faster diffusion of the carbon dioxide in the interior of the structure. Finally, if the cement content within the concrete structure is low, the water-cement ratio is high and the curing is not properly developed, then the alkalinity of the structure will not be high enough to provide the passive layer on the reinforcement (Broomfield, 2007).

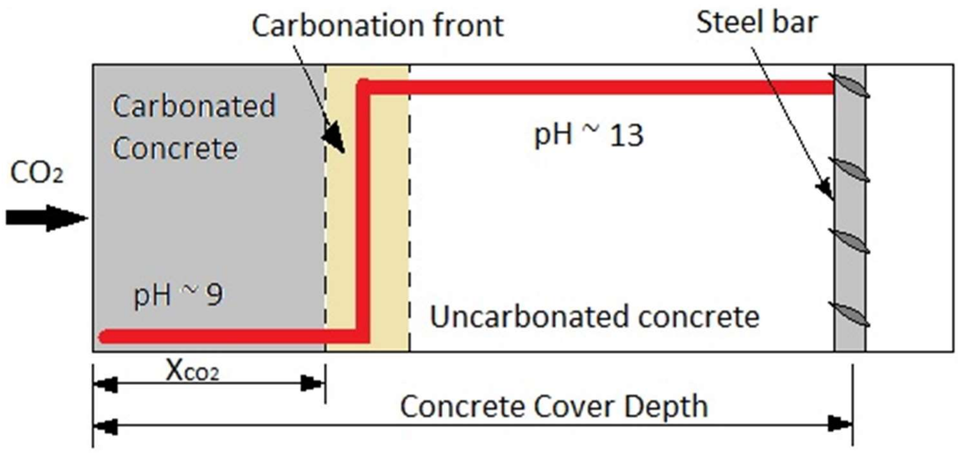


Figure 3.5 Degradation zones of a carbonated concrete structure (Ta et al., 2016)

The speed advance of the carbonation front is a very useful value once it is obtained, although expressing this value as a function of depth and time is not an easy task due to the non-linear nature of this relationship. However, considering that the carbonation index depends on the diffusion rate of the CO_2 in the concrete, like any other phenomenon governed by the diffusion of gases, it is possible to obtain a representative graph of the depth through the square root of time. Then, the gradient of this straight line formed by the equation provides a value in millimetres per year$^{0.5}$ or its equivalent, which was known under several names previously and which is now known as the carbonation coefficient (Dyer, 2014)

The study of corrosion has been developed in the last decades based on the formulation of mathematical and theoretical models that describe the process. In general, the starting point of several of these models is determined by the schematic diagram proposed by Tuutti in 1982, which relates the depth of corrosion with the life-cycle of a structure. Figure 3.6 shows the scheme proposed by Tuutti to represent corrosion degradation:

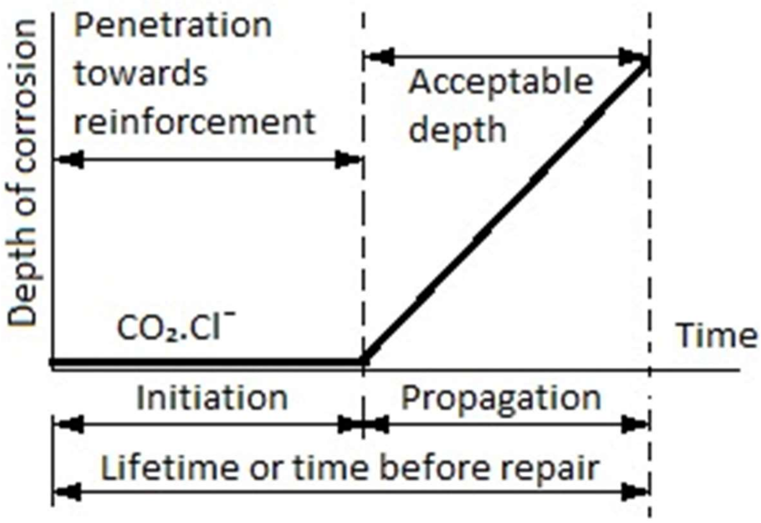


Figure 3.6 Corrosion sequence in RC structures (Tuutti, 1982).

The life-cycle of a structure can be considered from the viewpoint of corrosion as a period of two stages: initiation and propagation of corrosion. The initiation stage comprises the time that the harmful gases involved in corrosion take to get from the external surface of the concrete to the location of the reinforcing rebar. On the other hand, the propagation stage comprises the time in which the corrosion is accentuated until reaching the safety limit state, for which if repair measures are not taken, the end of the structure life-cycle is presented (Tuutti, 1982).

There are several mathematical models related to carbonation-induced corrosion found in the literature. Saetta and Vitaliani have developed a two-dimensional model in which the effects of moisture in a multidimensional aspect, the heat transfer and the transport of CO_2 through the concrete are demonstrated (Saetta and Vitaliani, 2005). Already at that time, there was the notion of developing new designs for each concrete mix that guarantee the reliability of the structure during the life-cycle taking into account the environmental effects.

Subsequently, slightly more complex theoretical models were developed to predict the effect of carbonation on structures. Models have been developed that describe the same parameters considered in the previous paragraph, combining it with balance equations and diffusion laws. These types of models are solved by an efficient numerical method using the concept of finite elements and numerical integration techniques. Therefore, the numerical result allows proposing a set of representative values for the corrosion risk of RC structures during its service life (Steffens et al., 2002; Park, 2008).

From the point of view of structural integrity, first Yoon *et al.* and then Stewart and Peng have developed a reliability-based approach for predicting the probability of the onset of corrosion and damage (caused by severe cracking) for reinforced concrete structures. These studies apply a projection of 100 years for the analysis of the service life of the structures based on climate change reports from the IPCC, which provides the possible scenarios for each climate variable. The required cover depth of the structure is estimated using reliable application methods and stochastic concepts that consider the micro-climatic conditions. These studies seek to provide an adaptive design for structural degradation against the effects of climate change. As a result, it was found that it would not be very useful to increase the cover thickness from the economic point of view. That is, preventive maintenance in the structure would be more economically efficient to address the problem of degradation under the climate change effects (Yoon et al., 2007; Stewart and Peng, 2010).

As has been seen in recent years, when studying the carbonation process, it is common to link the analysis considering the aspects of climate change. This is because carbon dioxide concentrations are directly related to this phenomenon. For this reason, a numerical carbonation model was developed by Talukdar *et al.* for the evaluation of the climate change effect on the RC structures. These studies show how this phenomenon negatively affects the performance and durability of buildings. The model successfully includes the effects of various properties (porosity, humidity, temperature, the atmospheric concentration of CO_2, and rates of chemical reactions) on the corrosion progression induced by carbonation in these structures (Talukdar et al., 2012a; Talukdar and Banthia, 2013). The model developed by Talukdar *et al*. (Talukdar et al., 2012a; Talukdar et al., 2012b) was adopted and is validated in this research work to generate the degradation curves of structures under the conditions (climatic and construction) concerning Paraguay. For this reason, this degradation model is comprehensively developed in Chapter 4 of this thesis.

More recently, the studies focus in a much more complex way, developing models that include parameters that represent reality more accurately and proposing new inspection methodologies for structures affected by corrosion. A recent study develops a new meta-model to predict the carbonation depth in concrete considering the natural condition of the process and the parameters available for new structures. These parameters include the design of the concrete mix, size of aggregates, type of cement, the initial period of curing, ambient temperature, relative humidity, CO_2 concentration, among others. Thus, this model allowed to determine that the carbonation index obtained with the accelerated process (i.e. in laboratory tests) is lower than the index in the natural process (Ta et al., 2016).

Regarding the detection of the carbonation depth in the structure, another investigation suggests a new method to replace the use of phenolphthalein for the determination of the carbonation front. The use of an innocuous solution based on curcumin (Curcuma powder) has been proposed, which provides a red indicator in areas where alkalinity exists and yellow in the carbonated areas. The dissolution of curcumin can be used with the same reliability as phenolphthalein in the detection of carbonation depth, with the advantage that it is safe and presents no health risks for technicians (Chinchón-Payá et al., 2016).

3.4.2 Chloride penetration

The presence of chloride ion in concrete is one of the greatest threats to steel reinforcement and the potential for corrosion. These ions can be immersed as contaminants in materials that are constituents of the cement paste or can penetrate the concrete through various transport processes from the environment that surrounds it. Concerning the latter, soluble chloride is commonly found from two sources: seawater and de-icing salts on concrete highways such as bridges and viaducts. The mechanisms of chloride ingress can occur through three processes: concentration of gradients (diffusion), a pressure gradient that causes the flow of solutions through the pores, and capillary action (Dyer, 2014).

A mixture of concrete that contains in composition the chloride ions is usually given by the use of saline sands or special additives. Even though the concrete can tolerate the presence of chlorides in its composition, the use of these additives is currently no longer allowed in RC structures and pre-tensioned structures due to its highly corrosive nature. Although certain levels of chloride inside the concrete are admissible, it is important to consider the surrounding environment to evaluate these levels. In this way, it is considered that in dry environments, structures with a low water-cement ratio admit up to a value of 4% chloride concentration. On the other hand, in more humid environments, corrosion can be facilitated in concretes with a chloride percentage of 0.5% (Porrero et al., 2009). Most design codes and regulations include guidelines and recommendations regarding tolerable levels of chloride concentration in concrete. Likewise, analyses that were carried out on structures with Portland cement have shown lower chloride concentration, which guarantees that it not to be a cause of corrosion onset (Bentur et al., 1997).

From the point of view of exposure, an essential condition for chlorides attack is the presence of moisture and the presence of salts, which is common in coastal environments. The regulations in force define the coastal zone as the region comprised up to five kilometres inland from the sea coast, where specific measures are necessary to protect the structure from this degradation mechanism (Fernandez, 2015). The chloride that comes from marine environments is a potent deterioration agent which can penetrate even the high-quality concrete until reaching the reinforcement, jeopardising its durability due to corrosion (Chess and Broomfield, 2014).

From the chemical perspective, BS 812-123:1999 establishes that when the chloride ions are positioned on the surface of the reinforcement, they seek to break the passive layer of such a surface and thus allow the corrosion propagation. This process involves the formation of more complex chlorides combined with iron ions from the passive layer, which is represented by the following reaction (BS:812-123, 1999):

$$Fe(OH)_2 + 6Cl^- \rightarrow FeCl_6^{-3} + 2OH^- + e^- \tag{3.6}$$

From the product of this reaction is obtained the compound $FeCl_6^{-3}$, which is soluble in the pores of the concrete and is removed from the passive layer, which compromises the protection of the reinforcement. In this way, the loss of the passive layer tends to occur at localized points on the surface of the reinforcement, causing pitting corrosion, which progresses until a hole has been created in it. Chloride ions upon contact with the reinforcement initiating a reaction that becomes a self-catalytic process, which is repeated several times until the protective layer is completely destroyed (Ahmad, 2006).

As with carbonation, the entry of chlorides into concrete has captured the attention of several researchers who seek to describe and understand the process. Several studies have been carried out to analyse the degradation of structures by chemical attacks such as sodium, magnesium, chloride and carbonate ingress, detecting the formation of Friedel salts (a substance formed from calcium mono-chlorinated aluminate hydrate) caused by the chloride (Brown and Doerr, 2000). The corrosion onset time was also studied, developing a prediction model of the chloride penetration rate in concrete, expressing it as a function of time in four different ways: constant, linear accumulation, square root accumulation, and square root accumulation with an initial set of surface chloride that comprise a more refined model (Ann et al., 2009).

The process of the chloride diffusion through the concrete is usually estimated analytically, being a simplified approach that generally presents shortcomings such as not considering the entry of chlorides by convection given in areas of spray and waves. To address these limitations, a study has presented a more comprehensive model where all these parameters and uncertainties are considered through the use of random variables. The equations of the degradation model are solved by coupling finite element systems. Thus, it is demonstrated the importance of including the influence of the random nature of environmental actions (Bastidas-Arteaga et al., 2011).

Currently, models for the estimation of chloride penetration in concrete continue to be developed, looking for alternatives to mitigate degradation by this mechanism. Thus, probabilistic models have been formulated which estimate the depassivation time of the reinforcement for the prevention of corrosion using limit state functions. These models have found that for a given failure probability of 10% (recommended by most regulations), the probabilistic times of depassivation obtained are lower than the deterministic times for the different transport models commonly considered in the literature (de Vera et al., 2017).

In summary, there is a certain similarity between the processes of degradation by carbonation and by chlorides. Therefore, several studies analyse the influence of both processes on the degradation of structures. The results of these investigations have shown that carbonation significantly influences chloride ingress patterns by accelerating ion diffusion rates. However, the carbonation rate in certain concretes (e.g. cement with fly ash) is reduced by the presence of chlorides (Liu et al., 2016). More recently, a probabilistic model has been developed for the initiation of corrosion considering the transport of CO_2, chloride ions, heat and humidity, obtaining a log-normal distribution function. This combination of factors has shown a higher probability of corrosion onset than those models that consider it independently (Zhu et al., 2016b). Actually, studies have shown that the initiation time of the reinforcement corrosion is reduced about 40% by the combined action of carbonation and chloride ingress in concrete (Zhu et al., 2016a).

3.5 Parameters that influence Carbonation-induced Corrosion

There are many parameters that can influence the initiation and propagation of corrosion. However, these parameters can be clearly classified into two groups. On the one hand, those parameters proper to the materials constituting the structure, and on the other hand, the external agents are given by environmental parameters in general. In this section, some of these parameters are briefly described, such as the water/cement ratio, the cover thickness, the content and type of cement, the curing and compaction during its execution and the environmental parameters.

Carbonation is facilitated when the quality of the concrete is not adequate. Lower cover thicknesses, high water/cement ratio, small amounts of cement and wet and dry cycles are conditions that favour the rate of carbonation corrosion. On the other hand, a high reserve of calcium hydroxide in the cement paste, proper compaction, adequate curing process of the concrete and the addition of pozzolanic fly ash delay the process of carbonation-induced degradation (Ribeiro and Cunha, 2014a).

By evaluating the durability of RC structures, it is important to consider these parameters both individually and the interaction of this set of parameters in the performance of the structure. The identification of these parameters is a paramount tool to establish a study on the degradation mechanism of structures. The influence of each of these parameters from a mathematical formulation is developed in Chapter 4 of this research, where it is addressed the mathematical model adopted for the development of degradation curves.

3.5.1 Water/Cement Ratio

To reduce the amount of water and maintain minimum workability of the cement paste, it is necessary to add cement to the concrete mix. In this way, the water/cement (w/c) ratio is the main factor that controls the strength of the concrete (Nilson, 2001). Therefore, excessive water in the concrete mix decreases strength, increases drying shrinkage, increases porosity, increases creep, and reduces the abrasion resistance of concrete. In other words, if the w/c ratio is not controlled, then the concrete will not be durable in many exposure conditions (Von Fay, 2015). Results of studies show clearly that the corrosion current density is reduced when the w/c ratio of the concrete is decreased from 0.7 to 0.4. Thus, if other parameters of the concrete mix are constant, the change in concrete quality associated with the decrease in w/c ratio influences greater the corrosion current density value than an increase of cover thickness (from 5 to 10 cm) for concrete of the same quality (Balabanic et al., 1996).

It has been found that if the water/cement ratio is increased from 0.4 to 0.8, the diffusion coefficients increase more than ten times (Houst and Wittmann, 1994). Furthermore, the water/cement ratio is associated with the porosity of the material. Studies have shown that the microstructural characteristics are very different among cement pastes that have water/cement ratio that ranges between 0.35 and 0.40. However, for values higher than this, the porosity increases strongly, facilitating the diffusion of gases (CO_2) toward the interior of the concrete (Chaussadent et al., 2000). Therefore, concrete structures may be designed to have a specific carbonation resistance based entirely on the water-cement ratio (Basheer et al., 1999). Recently, a study has established expressions for estimating the accelerated carbonation resistance of concrete as a function of its water/cement ratio through a set of abacuses. It was found that such formulations are a helpful tool for the service life prediction of concrete structures (Ferrer et al., 2016).

3.5.2 Cover Thickness

The cover thickness of a concrete structure is usually determined from the considerations and solicitations established in the design. For an improvement in the durability of this structure, the thickness of the concrete cover is considered one of the most important parameters in the structural design, knowing that the initiation of reinforcement corrosion is a critical factor that determines the service life of a structure. Therefore, the thickness of the coating must necessarily satisfy the following equation (Pan et al., 2015):

$$C(x = x_0, y = d_c, t = T_s) \leq C_r \tag{3.7}$$

Where x_0 is the spatial location of the exposed surface, $C(x, y, t)$ is the content of chlorides or carbonates in the position (x, y) at the time t, d_c is the cover thickness, T_s is the expected life-cycle time, and C_r is the critical content of chloride or carbonate in the structure. As can be seen, the same approach and formulation can be equally applicable to the process of carbonation and chlorides in the concrete.

Carbonation of the cover jeopardises the durability of a structure by reducing the protective capacity of the concrete to reinforcement. A safe thickness comprises the minimal thickness of the concrete covers, which will protect reinforcement. A necessary condition for a sustainable approach to the relationship between the amount of CO_2 sequestration by concrete and the durability of a reinforced concrete structure is given by an appropriate choice of concrete cover thickness (Czarnecki and Woyciechowski, 2012). In a study developed by Sisomphon and Franke, the most appropriate cover thicknesses are shown according to the type of cement used for the concrete structure (Sisomphon and Franke, 2007).

3.5.3 Type of Binder

The cement type, as well as its content, usually plays an important role in ensuring the durability of the structure from the point of view of carbonation-induced corrosion. Nowadays it is possible to find a series of cement in the construction market in addition to the best-known type, the Portland cement. A common practice in construction is the combination of Portland cement with some of the other existing types of cement such as cement with fly ash, cement with silica fume, pozzolanic, among others.

According to Khan & Lynsdale, carbonation-induced can be increased in its propagation values in concrete that uses cement with fly ash, e.g. for every 10% increase; the carbonation depth rises approximately 0.3 mm. However, in cases where the concrete has between 10% and 20% of silica cement, it is possible to decrease the carbonation depth concerning the values obtained with Portland cement only (Khan and Lynsdale, 2002). Meanwhile, it should be mentioned that in the case of fly ash cement concrete, the carbonation depth is higher than Portland cement concrete only if the water/binder ratio is higher than 0.55 (Czarnecki and Woyciechowski, 2012). For this reason, it is important to know the environment of the structure in order to design an appropriate combination of cement use that guarantees its durability, decreasing its permeability and increasing its protection against harmful substances (McPolin et al., 2005).

The pozzolanic mixtures have a lower carbonation resistance, therefore have also a shorter induction period for carbonation. Hence, the induction period for carbonation of pozzolanic mixtures, particularly in the case of CEM III series, is considerably shorter than of CEM I concretes with the same curing (Sisomphon and Franke, 2007). A study about carbonation in concrete with different supplementary cementitious materials (SCM) has found that mixes with similar clinker replacement level but with different types of mineral additives manifested remarkable variation in carbonation resistance. Moreover, the carbonation rate was found to increase with an increase in clinker replacement level with SCM (Shah and Bishnoi, 2018).

3.5.4 Concrete execution

The moment of execution of the concrete structure is a very important parameter that influences the performance of the structure during its service life. In addition to establishing control over the type of cement, the water/cement ratio for the mixing and the attention needed to fulfil the cover thickness, it is necessary to take care concerning the curing stage in the concrete elaboration. Water may evaporate where concrete surfaces are exposed to the atmosphere, leaving the quantity of free water insufficient for complete cement reaction. The ambient temperature, concrete temperature, relative humidity, wind speed and exposed surface area are some factors that influence the evaporation rate of the water. Therefore, the concrete curing process affects both the mechanical properties and the permeation characteristics of the material, particularly on the surface. Considering that all aspects of durability are determined by one or both properties, the durable concrete must be properly cured (Dyer, 2014).

Concerning concrete structures containing normal Portland cement, a minimum period of 7 days is generally recommended for the curing process. On the other hand, for concrete mixtures comprising either a blended Portland cement or a mineral admixture, extended curing period is desirable to guarantee strength contribution from the pozzolanic reaction (Mehta and Monteiro, 2006). Another parameter to be considered is the vibration of the fresh concrete. When the concrete is recently poured into the formwork, air bubbles can occupy between 5 and 20 % of the total volume. Vibration allows fluidifying the mortar component of the mix so that internal friction is reduced and packing of coarse aggregate takes place. Vibration must be applied uniformly to the whole concrete mass as otherwise some parts of it would not be entirely compacted while others might be segregated due to over-vibration (Neville, 2011). Practical guidance for the proper execution of concrete may be found in (CEB, 1989; fib, 1999; ACI, 2002, 2011)

3.5.5 Environmental factor

Environmental exposure is also considered a determining factor in the degradation by carbonation in concrete structures. Many studies have determined that depending on whether a structural element is sheltered or unsheltered from the weathering, the carbonation rate varies significantly. In a research where it has been investigated the carbonation of existing concrete building structures located in China, the influence of the concrete exposure has been demonstrated. For the concrete core samples without surface coating, the indoor natural carbonation depth and rate were found to be greater than the outdoor ones due to the different level of CO_2 concentration (Li et al., 2018)

The environmental factor was always a parameter of concern in the study of the durability of the structures. However, in the last decades, this parameter has become stronger due to the global concern that causes the effect of climate change. Considering the focus of this research, the environmental parameters for the carbonation-induced degradation commonly considered are the relative humidity, the temperature and the concentration of CO_2 in the atmosphere.

On the one hand, the diffusion of carbon dioxide within the pores of the structure is influenced by the external relative humidity. As is seen in Figure 3.7, whether the value of this parameter is too high or too low, it usually does not influence the carbonation in the structure. For this reason, several investigations coincide in the standardisation of a relative humidity value between 50% and 70%, with which the transport of carbon dioxide to the interior of the structure is increased. Similarly, the temperature plays an important role in the carbonation depth, for which it has been found that the optimum gap for this process is between 20 °C and 40 °C. This is due to the nature of the diffusion phenomenon and the influence of temperature on the diffusion coefficients (Papadakis et al., 1991a, 1992; Figuereido and Meira, 2013).

Regarding the concentration of carbon dioxide in the atmosphere, many authors use the value of 0.03% to estimate their influence on the structures. However, as has already been seen, this parameter tends to increase rapidly since the beginning of this century as a direct effect of global climate change. Also, the values of the concentration of CO_2 vary according to the region and the type of exposure to which the structure is immersed. Even so, studies have shown that the speed of carbonation front and the carbonation depth depend fundamentally on the concrete quality and relative humidity, without noticing significant variations between the different measurements of concentration in several locations (Yoris et al., 2010).

Concerning the concentration of carbon dioxide, it is expected that an additional 16% of all concrete surfaces by the year 2100 will be damaged and will require expensive maintenance or repair work. Thus, it was found that the risks of damage induced by carbonation will increase to 20-40% for certain emission climatic scenarios provided by the IPCC. Likewise, the additional damage risks for chloride-induced corrosion are only 3% during the same period due to the increase in temperature, but without considering the change of ocean acidity in the marine exposure (Stewart et al., 2012b).

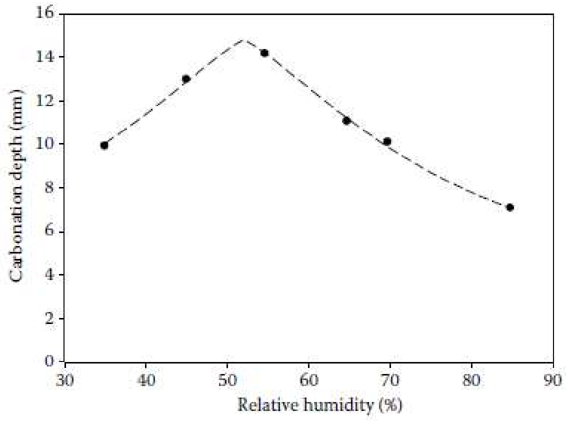


Figure 3.7 Carbonation depths concerning relative humidity (Papadakis et al., 1991a).

Considering the above regarding climate change, numerous studies and projects have been developing since the beginning of this new millennium. These investigations are aimed at predicting the performance of structures against global warming and the effects of greenhouse gases. Parameters previously mentioned such as humidity, temperature and concentration are directly influenced by the effects of climate change. According to the observed records, it is possible to see that there has been an increase in the global mean temperature around 0.6 °C since the beginning of the 20th-century and that this increase is associated with more intense warming in the minimum daily temperatures compared to the maximum. Furthermore, global precipitation has also increased over the same period, which has increased the relative humidity (J. Easterling et al., 2000).

In the coming years, knowledge regarding the implications of climate change in the environment of buildings will become one of the most important aspects in the industry. The economic factor of buildings about climate change should be treated with a dynamic analytical framework that allows explicit consideration of changes in the information established over time (Nordvik and Lisø, 2004).

3.6 Inspection Techniques and Maintenance

Once a technical analysis on the degradation of a structure is developed, the next point to consider is the maintenance planning of such structure, which depends directly on the results obtained in the inspections that are carried out during the service life. Generally, the conception of the maintenance of a structure is given from a proper need of the constructive system, which is to avoid that it fails during its service.

Before maintenance, inspections are applied to monitor the state in which a construction element is found and to preventively repair or replace components to ensure reliable system functionality (Kallen and Noortwijk, 2006). For any maintenance activity, an inspection activity is always necessary. It has been shown that about 80% of the preventive maintenance activities were carried out due to failures previously reported or identified during the inspection period (Wang and Carr, 2011).

According to British Standard BS:3811, maintenance operations can be defined as the combination of any action taken to restore an element to an acceptable condition. The maintenance then seeks to preserve the proper conditions of the infrastructure as well as to improve the conditions of the elements as necessary (BS:3811, 1984). Maintenance operations cover most of the life cycle of a building. However, the maintenance of these structures becomes uneconomic after the end of the service life. Therefore, it is necessary to know the economic service lifetime for each type of infrastructure analysed. For instance, in monumental buildings and bridges, a period between 100 and 200 years of economic service life is usually considered (Panchdhari, 2003).

Both the degradation mechanism and the maintenance tasks of a construction system are governed by some uncertainty. Therefore, the activities involved in maintenance actions often provide results with a certain degree of controversy among themselves. In other words, maintenance clearly affects the reliability of the components and the system. If the number of interventions is low, this can result in an excessive number of costly failures and poor system performance and, therefore, reliability degrades. Otherwise, if it is performed too often, reliability may improve, but the maintenance cost will increase drastically. Hence, in a profitable scheme, a trade-off between both expenses must be achieved (Endrenyi et al., 2001).

This section describes the techniques commonly applied to the inspection and maintenance of concrete structures with carbonation-induced corrosion risk. This review is carried out because in Chapter 5 of this research analysis is made regarding the effectiveness and cost of applying these techniques through the optimisation of maintenance.

3.6.1 Inspection Techniques

The inspection of the construction systems arises as a need to know the state of conservation of a structure or because of an external action that forces to know eventual signs of damage or system failure. There are specific inspection techniques to be applied to each case and for each type of material inspected. In this section, it is presented the techniques applied both to know the "*state of health*" of the carbonated concrete as well as to detect an eventual state of corrosion of the reinforcement.

A general classification of the inspection techniques comprises two categories: destructive techniques and non-destructive techniques (NDT). The main purpose of NDT is to determine the quality and integrity of materials, components or assemblies without modifying the ability to perform their assigned functions. Among several ways of evaluating material integrity in concrete structures, NDT is increasingly gaining acceptance. Usually, engineers have some problem in understanding the variability both in the construction materials and geometry of the structure and what the NDT specialist can and cannot quantify (McCann and Forde, 2001). However, its acceptance is being hindered by the low awareness regarding NDT methods that are attributable to a deficiency in a proper understanding of construction materials and NDT methods themselves (Helal et al., 2015).

Corrosion of reinforcement in concrete is a complex process for which individual experience must be involved in its assessment. Also, any technique used to determine corrosion status should include two additional methods: carbonation depth measurement and chloride ion profiles (Carino, 1999). This section addresses the more traditional NDT applied in the inspection of corrosion in RC structures.

3.6.1.1 Phenolphthalein Test

Phenolphthalein is a chemical compound that reacts according to the pH level of a material. For use as an indicator, phenolphthalein must be dissolved with an appropriate solvent, such as, for example, isopropyl alcohol (isopropanol), in a 1% solution. The test consists of applying the indicator on the sample surface of the concrete, which produces a dark pink colouration when it is in the presence of a basic medium or high pH (> 9.5). This test is only recommended as an estimator of the depth of the carbonation front since to have an accurate confirmation of this value the technique of optical or electronic microscopy is recommended (Vidaud and Vidaud, 2012).

Phenolphthalein test may be performed either by breaking away a fresh surface or by coring and splitting or cutting the core in the laboratory. However, care must be taken that no contamination of the surface takes place from dust and the phenolphthalein sprayed surface must be freshly exposed, or it may be carbonated before testing. This test gives the maximum, average and standard deviation of the carbonation depth of the sample (Broomfield, 2007).

Despite the popularity of the phenolphthalein test, as it is a chemical of recognised toxicity, the need arose to look for another method that is less risky. To this end, studies have recently been carried out on the use of a harmless solution based on curcumin (i.e. the herbaceous root of the ginger family), which acts as an indicator with the same reliability of phenolphthalein but with the advantage that it is safe and does not present health risks (Chinchón-Payá et al., 2016). The discolouration of the concrete surface sprayed with this solution with a purity of curcumin above 95% remains reddish (pH~12) or yellow (pH $\leq$ 9).

3.6.1.2 Thermogravimetric Analysis

Thermogravimetric analysis (TGA) is a technique in which the weight of a substance is monitored as a function of temperature or time. Likewise, it is essential that the sample specimen stay in a regulated temperature program as well as in a controlled atmosphere (Talukdar, 2013). Although the measurement of carbonation by phenolphthalein can be used as a good indicator of the corrosion onset, the technique of inspection by thermogravimetric analysis has the advantage of giving a more accurate profile of the carbonation front location. Studies have shown that between the carbonation depth measured by the phenolphthalein test (X_P) and the measurement made with TGA (X_{TGA}), the relationship X_P/X_{TGA} is approximately equal to 2.0 (Chang and Chen, 2006).

TGA is thus a convenient technique to determine concrete carbonation because it quantifies the calcium carbonates and portlandite contents in a sample of mortar obtained from the concrete mortar and reduced into powder. It is recommended that TGA is supplemented with any chemical analysis to give more accurate quantitative profiles. However, when calcareous aggregates are present in the concrete mix, the direct evaluation of carbonates by TGA is not possible anymore (Villain et al., 2007).

3.6.1.3 Gammadensimetry

In civil engineering, the gammadensimetry is an NDT method employed to measure the density of materials. For instance, by auscultating bituminous or hydraulic concrete cores taken from the pavement, or by directly inserting a probe in the layers of the roadway to be checked. Moreover, gammadensimetry is quite suitable to determine the drying, porosity and carbonation profile. Proper knowledge of the internal hydric state of the tested sample enables to examine the results of other tests regarding the influence of internal moisture on the studied phenomena, mainly carbonation. This makes the use of gammadensimetry a powerful method to quantify the carbonation progression by monitoring its progression over time. Nevertheless, this method cannot be used on cores from concrete structures to obtain carbonation profiles. In contrast to the previous method, the TGA profiles are very similar to that obtained by gammadensimetry. However, gammadensimetry enters all CO_2 penetrated in concrete, which has reacted, or which is adsorbed at the surface of the concrete pores. Meanwhile, TGA quantifies only CO_2 chemically bound to calcium carbonates (Villain and Thiery, 2006).

The main advantage of this method is that it makes it possible to measure very accurately the total penetrated CO_2, without a specific preparation of the specimens. Likewise, it has shown to be a useful tool to monitor laboratory accelerated carbonation tests. However, a main shortcoming of the gammadensimetry methods relies upon that an auscultation of uncarbonated sample before degradation is required. Therefore, it is cannot efficiently and accurately quantify the CO_2 content in a core of an aged concrete structure (Villain et al., 2007).

3.6.1.4 Electric Resistivity Test

Concerning the determination of the intensity of the initiated corrosion process, the electrical resistivity of concrete is an important parameter. If a high electrical resistivity is detected in the concrete material, the corrosion process will be slow regarding concrete with low resistivity in which the current can easily pass between anode and cathode areas (Song and Saraswathy, 2007). The measurement of the resistivity allows, in a certain way, to indicate the corrosion rate, being a useful complement to the inspection techniques based on the potential measurement (Carino, 1999). It is a method applied for several years which provides the risk of corrosion of the rebar based on the measurement of the electrical resistivity. This method is standardised by the American Society for Testing and Materials (ASTM), which in principle had its application in soils, but was quickly adapted for use in concrete (ASTM-G27, 2012).

The method mainly consists of inducing the passage of current between electrodes by means of an alternating current source. In this way, the potential difference between the electrodes is measured, which gives the value of the resistivity through the application of Wenner's formula. The interpretation of the results is made from a corrosion risk classification formulated by the CEB 192:1998 as can be seen in the following table (Da Silva, 2001):

Table 3.1 Interpretation of resistivity test results.

Resistivity (Ω.cm)	Corrosion Risk
< 5000	Very High
5000 a 10000	High
10000 a 20000	Low/Moderate
> 20000	Low

The principal deficiency associated with this method is that of attaining good electrical contact between the electrodes and the concrete structure. Moreover, it is usually necessary to drill small holes to ensure effective contact (McCann and Forde, 2001). Furthermore, it is important to consider the effects that interfere with the results of this test. Among them, it can be mentioned the effects of polarisation, type of cement, the porosity of the concrete, the resistance of contact between the electrodes and the surface of the concrete, among others.

3.6.1.5 Potential Measurements

Potential measurements are a commonly used NDT method which is relatively easy and quick to apply. The method can provide an indication of localised corrosion initiation by voltage measurements between a reference cell attached to the concrete surface and the reinforcement. Therefore, the measurement of the potential provides an evaluation of the probability that there is active corrosion in the structure. It should be made clear that the method does not provide information regarding the corrosion rate (Carino, 1999).

Preparing potential contour maps on the concrete surface may be useful to identify zones of varying degrees of corrosion risk and these are especially suitable for assessing maintenance and repair requirements (McCann and Forde, 2001). In the interpretation of the results obtained by this test, it can be considered that measurements below -350 mV suggest a 90% probability of corrosion.

Meanwhile, values above -200 mV are associated with a probability of corrosion equal to 10% (Broomfield, 2007).

The quality of the measurement can be affected by the humidity of the concrete and the contact of the reference electrode with the reinforcement. The main limitation of the method is that, although it provides an indicator of the presence of corrosion, it is not able to represent the corrosion rate (Vassie, 1991). This shortcoming is given because its values indicate the balance between the cathodic and anodic reactions, without offering quantitative information (Ribeiro and Cunha, 2014b).

3.6.1.6 Acoustic Emissions

The acoustic emission (AE) techniques have a high sensitivity to assess the ongoing corrosion process. It has remarkable feasibility as a technique for monitoring corrosion in RC structures. Hence, these capabilities make the AE technique an efficient not destructive method in the detection of corrosion occurring in real-time, giving it an advantage over other NDT methods (Ahmad et al., 2015).

Studies were reported that AE techniques could provide an earlier warning than electrochemical methods. Indeed, both the corrosion onset in reinforcement and the nucleation of cracks in concrete due to the expansion of corrosion products are successfully detected by AE (Kawasaki et al., 2010). Likewise, other studies have found that the index of damage derived from the AE technique obtained during the first stage of damage was found to be useful as an indicator for assessing the extent of corrosion damage of RC beam specimens at initial loadings (Ahmad et al., 2017).

Nevertheless, some care must be taken during the application since the performance of this technique is often poor in surfaces with rough patterns. Hence, it is necessary to ensure proper contact with the material for which may be required the use of a coupling gel between the transducers and the structure (Song and Saraswathy, 2007).

3.6.1.7 Linear Polarisation Resistance

The Linear Polarization Resistance (LPR) technique has been developed to attain an accurate assessment of the condition of RC structures. The technique is rapid and non-intrusive, needing only a connection to the rebar. The data gives a worthy insight into the immediate corrosion rate of the steel reinforcement, giving more specific information than a simple potential survey (Song and Saraswathy, 2007). Moreover, the linear polarisation can be used to estimate the section loss rate of the corrosion reinforcement (Carino, 1999).

This technique comprises a hand-held electrode that is held at the concrete surface and used to apply an external current that acts to modify the current in the reinforcement by a small quantity. Table 3.2 shows the interpretation of the results given by the LPR test using the corrosion current density, i.e. the corrosion current per unit surface area of reinforcement (Dyer, 2014).

Table 3.2 Interpretation of resistivity test results.

Corrosion current density ($\mu A/cm^2$)	Corrosion rate
< 0.1	Negligible
0.1 a 0.5	Weak
0.5 a 1.0	Moderate
> 1.0	High

Nevertheless, these measurements are influenced by environmental parameters such as the temperature and relative humidity (RH), so the conditions of measurement will affect the interpretation of the boundaries defined in the table. Thus, the LPR technique has two significant shortcomings. On the one hand, this technique detects the instantaneous corrosion rate which can fluctuate with temperature, RH and other factors. On the other hand, with this technique is usually made assumptions about the area of measurement that leads to errors of 10-100 in the estimated area of measurement, especially at low corrosion rates, unless the sensor controlled system is used (Broomfield, 2007).

3.6.2 Maintenance Techniques

Many technical systems, such as civil infrastructure, are exposed to degradation as a result of their daily use and the age of the structure itself. Therefore, these systems are usually subject to preventive maintenance policies to avoid failure (Welte, 2009). As already mentioned, the maintenance of a building is essential to guarantee its service life. However, as with the inspection technique, these maintenance actions must be carried out according to the needs of the building, the expected symptomatology, and the purpose to be achieved with the activity.

Repairs can be either structural or non-structural. Structural repair involves the removal and replacement of concrete in an element that is subsequently required to support loads. The non-structural repair, on the other hand, corresponds to actions such as filling holes or cut-outs from concrete where the repair material will not be subjected to loads and will not be in contact with reinforcement – i.e., the repair material is just an aesthetic intervention. Regardless of whether a structural repair is treated or not, the intervention must provide adequate protection against corrosion reinforcement (Dyer, 2014).

The following describes some methods commonly employed for the repair and maintenance of concrete structures in which the inspection has detected symptoms of corrosion initiation, or the propagation of carbonation-induced corrosion.

3.6.2.1 Mechanical scarification

This technique consists of removing the concrete cover that was contaminated using a machine. Special care must be taken with the depth to which the concrete removal is made so as not to damage the reinforcement, the installations that could exist in the concrete and the head of the machine. This technique is no longer widely applied in the United Kingdom and much of Europe, although in North America it is still used in the repair of concrete bridges. It is usually preferable that the scarification of the damaged concrete is made using hydro-pressure machines which remove the material with a uniform depth avoiding possible unwanted damages (Broomfield, 2007).

In recent years, the scarification of the surface damaged by hydro pressure has had a high acceptance for the maintenance of damaged concrete structures. The main feature of this technique is not generating excessive vibrations in the structure that can produce additional cracks that worsen the condition of the structure.

3.6.2.2 Cleaning of exposed reinforcements

The cleaning of the rebar is done to remove the rust or any other unwanted material that contaminates the reinforcement. It is extremely important to perform this cleaning to stop the corrosion of the reinforcement. They are usually carried out according to three methods: sandblasting, brushing, and hydro-demolition. The first method consists of the abrasive impact of sand particles by compressed air; the brushing is done with wire bristles with a pneumatic or hydraulic operation; and the method by hydro-demolition is similar to the first method where the sand particles are propelled with high-pressure water to create a cleaning abrasive liquid (Vorster et al., 1992).

3.6.2.3 Epoxy bond bridge

The epoxy is a substance that provides a barrier coating to the steel and, thus, prevents the corrosion initiation caused by the depassivation layer near to the rebar. While many structures with epoxy coated steel have worked well, there are other cases with less favourable results. These latter cases have been attributed to defects in the coating, most likely due to defects introduced during construction, or to the absorption of moisture by the epoxy which leads to swelling and separation of the steel. Therefore, it is generally concluded that good quality epoxy coatings will increase the time to corrosion onset but, once started, the corrosion rate will be approximately the same as that of uncoated steel reinforcement (Sagüés, 1991; Weyers et al., 1998).

3.6.2.4 Cathodic protection

Cathodic protection consists of changing the potential of the steel to more negative values. This change in potential can be obtained by connecting an external anode to the steel and printing a continuous electrical current through the reinforcement using a rectified power supply. The expanded titanium mesh activated with a surface coating of titanium oxide is the anode most used in practice. This external anode is mounted on the surface of the concrete. The positive terminal of the low voltage direct current source is connected to the mesh and the negative terminal is connected to the steel bars (Cheaitani, 2000). It should be mentioned that to establish whether a cathodic protection system has the wanted effect, it is usually necessary to also provide a structure with a monitoring system (Dyer, 2014).

There are three factors that must be considered when controlling a cathodic protection system. First, there must be enough current to overwhelm the anodic reactions and stop or reduce the corrosion rate. Then, the current must stay as low as possible to reduce the acidification around the anode and the attack on the anode for those that are consumed by the anodic reactions. Lastly, the steel should not exceed the hydrogen evolution potential. In addition, this method has some limitations such as an increase in the dead load on the structure, a change in the profile that can reduce clearances, and the overlay can be difficult to apply in restricted areas or with complicated geometries (Broomfield, 2007).

3.6.2.5 Realkalization

This method consists of an electrochemical technique that is applied temporarily to promote the re-alkalization of contaminated concrete. The realkalization comprises the application of a continuous electrical current between the rebar of the concrete and an external metallic mesh, encapsulated in an electrolyte. The production of OH^- ions induces, at the level of the reinforcements, the increase in pH in values above 13.5. Simultaneously, these ions in an electrolytic solution migrate by electro-osmosis effect from outside to inside the concrete. The intensity of current used varies between 0.5 to 2 A/m^2 of the concrete surface whose treatment can last days or weeks, depending on the carbonation depth, cover thickness, concrete quality, the intensity of the current and its distribution (Lourenço and Caldas de Souza, 2014b).

After applying this treatment, the pH is considerably higher in most parts of the concrete and highest around the reinforcement, principally due to the production of hydroxide ions in this zone (Dyer, 2014). In laboratory tests, the patent authors have shown that it is extremely difficult if not impossible for a treated specimen with this technique to carbonate again (Broomfield, 2007).

3.6.2.6 Reconstruction of the cover

The reconstruction method must deliver the selected repair material to the prepared substrate with predictable results. The properties of the repair materials generally specified are the compressive strength, the bond strength, the shear strength and the properties that influence volume changes, such as shrinkage by drying, modulus of elasticity and thermal expansion coefficient. The repair material must completely encapsulate the exposed reinforcing steel, achieve a satisfactory bond with the substrate and fill the prepared cavity without segregating. If these requirements are not met, the repair will not fulfil its purpose (Nemati, 2006).

3.6.2.7 Membranes and Coatings

The coating of the concrete surface with an adherent and impermeable membrane is one of the most used methods in the protection against corrosion. Its main objective is to minimise the entry of harmful agents such as water, chloride ions, oxygen or carbon dioxide. Among the most used materials are organic paints, such as paints based on epoxy resin, acrylic, polyurethane, vinyl, and bituminous. The high-density concrete waterproof coverings are also used, as well as Portland cement polymer mortar (Lourenço and Caldas de Souza, 2014a).

It is important that the surface of the concrete that will receive the treatment is free of oily or fatty materials that may damage the adhesion of the membrane. There are anti-carbonation coatings which should be applied after carbonation repairs to prevent further carbon dioxide ingress. Moreover, it should be considered that the total or near total sealing of concrete can cause problems. If this coating seals all the pores of the concrete, the water trapped in the pores then when the atmospheric humidity drops or the temperature increases huge forces can be exerted against the barrier coating causing blistering and coating failure (Broomfield, 2007).

3.7 Carbonation in Latin America

The focus of this research is aimed at providing a useful tool for the maintenance of concrete structures in Paraguay. As it will be seen later, carbonation is one of the most recurrent problems concerning the degradation of these structures in the country. Therefore, to contextualise the phenomenon of carbonation in the region corresponding to Paraguay, this chapter includes a section describing the problem of carbonation-induced corrosion in Latin America. Some studies developed in different countries of the region are presented in the following paragraphs.

One of the pioneers in the study of the forecast of the service life of the structures and the durability of the same in the Latin American region was Paulo Helene, who at the end of the last century wrote a series of books regarding the methods for estimating the life-cycle of structures (Yugovich, 2007). As has been demonstrated earlier in this chapter, carbonation is a global problem that affects concrete structures. Its immediate effect, the corrosion of the reinforcement, is directly related to the estimation of the service life of these structures.

Although worldwide the study of carbonation has been widely studied, in some countries of Latin America there is still little progress in the study on this subject. This research aims to investigate the degradation of buildings in Paraguay, with emphasis on the degradation of RC structures by carbonation-induced corrosion. However, the literature is insufficient in relation to this field in the country, so this section develops a review of the studies conducted on this subject in South America. For this purpose, studies carried out in Argentina, Uruguay, Colombia, Chile and Venezuela will be taken as a reference, as well as some case studies in Paraguay.

In Argentina, some case studies have been made to diagnose the state of the structures with respect to carbonation corrosion. In the city of Santa Fe, specimens were prepared and then exposed to the weathering to analyse the influence of urban pollution on the degradation of structures. The samples were exposed for 36 months and it was determined that the penetration of the carbonation front tend to increase slightly in those that are protected from wetting of rainfall. In some samples were applied acrylic paints in which practically the carbonation was null. The phenolphthalein tests at the end of the test yielded values between 0 and 11mm thickness of the carbonated front. Likewise, in the city of San Juan, reinforced concrete constructions built in areas with high underground water have denoted widespread corrosion by carbonation in their reinforcement (Ocampo et al., 2009; Yoris et al., 2010).

Also in Argentina, an investigation was carried out on the problems of corrosion observed in buildings executed between 1930 and 1960 in reinforced concrete that integrate modern heritage (Traversa, 2011). Measurements made in different environments of the province of Buenos Aires using a carbon dioxide meter have shown values of 410 ppm in urban areas and 378 ppm in rural areas, considerably high values that facilitate the carbonation process in the structures. It is worth mentioning that these concentrations depend on the vehicle fleet, the population density, the existence of industrial areas, and the climatic conditions related to the wind. The structural typology studied in this research includes infrastructures typical of modern heritage such as housing complexes, public buildings, viaducts and bridges. In the survey carried out, 56% of the works showed signs of corrosion. This percentage includes 15% of infrastructures that show advanced signs of corrosion.

In 2006, a sector of the reinforced concrete slab of a business centre built in 1955 collapsed in Venezuela. This event provided an exhaustive investigation to determine the cause of the failure in the structure. It was possible to determine that the longitudinal rebar that was exposed due to the collapse of the slab showed an advanced state of corrosion with a cross-sectional loss. The generalised humidity in different structural elements caused the formation of efflorescence in the building. In addition, it was possible to confirm the loss of bearing capacity due to excessive corrosion of the reinforcement. Hence, the main cause of the collapse was the moisture coming from the filtrations and the carbonation of the concrete. Measurements of the carbonation depth were made whose values ranged from 2.45 cm to 3.33 cm, which indicated that the carbonation front already reached the area of the reinforcement since the cover thickness was just 1.5 cm. Evaluating the porosity (20%), which exceeded the maximum values allowed by the standards (15%), it was also possible to verify a poor quality of the structure (Dikdan et al., 2008).

On the other hand, several studies have been carried out in Chile in the city of Santiago and in the city of Valdivia. In these investigations, it was possible to observe that the carbonation depths are higher in the structures located in the city of Santiago than in the other cities. This presumably due to the urban and industrial environment present in the capital of Chile. The most probable reasons for this difference are the seasonal variation of relative humidity that allows a higher concentration of CO_2 in the city of Santiago. In Chile, like much of Latin America, there is no real awareness of the problem of the durability of RC structures. In an analysis of several infrastructures (bridges and buildings), the results corresponding to the city of Santiago and its surroundings showed maximum values of carbonation depth of up to 8 cm for a structure of 95 years of lifespan (Rojas, 2006).

In the same way, Barrera *et al*. have performed an analysis on samples taken from 11 buildings of different ages in the city of Santiago de Chile, confirming in all of them the presence of calcium carbonate in different proportions (Barrera et al., 2008). A meteorological study concerning relative humidity has attained to predict that carbonation could become a real danger for buildings in the city. The geographical location of the city facilitates the highest concentration of carbon dioxide due to the scarce winds in the region. In this study, the oldest building analysed (69 years) presented an average carbonation advance of 5.85 cm, and the youngest one (33 years), an average value of 2.8 cm. The greatest depth of carbonation (9.1 cm) occurred in a 56-year-old building, which represents very high values that match previous research. On the other hand, in the city of Valdivia, a structure built in 1950 was analysed, which presented relatively lower levels of carbonation than in Santiago, with an average value of 1.5 cm. This is due to the high relative humidity value in the city (> 80%)

that saturates the pores of the concrete and does not allow the diffusion of CO_2 into the structure (Monroy, 2007).

The analysis of Chilean standards regarding the corrosion problem of concrete reinforcement leads to the conclusion that there should be a training of the personnel assigned to control the compliance of these. Another problem is due to the lack of awareness and commitment towards the durability of the structures. This occurs because the construction companies lose contact with the building once it is completed and do not usually take part in the anomalies that arise since the legal term of responsibility in Chile is only five years (Carvajal, 2002).

In Uruguay, the situation does not differ much from the above. A study carried out on a building built a little less than 40 years ago in the city of Punta del Este detected anomalies related to the absence of concrete cover caused by spalling, apparent increase in the rebar diameter caused by corrosion and even partial lack of the reinforcement. It was detected that all the exterior surfaces exposed to the air were carbonated, being able to determine a carbonation rate equal to 6.71^{-5} mm/year (Valleta and Martinez, 2009).

On the other hand, in Colombia, studies on carbonation in concrete structures were also found. The notable increase in air pollution in the city of Bogotá has generated concern in recent decades both for the quality of life and for the durability of the structures. This pollution is mainly due to the accelerated economic and industrial growth that demands greater use of energy and the use of fossil fuels (Gaitán et al., 2007). According to a study carried out in the towns of Usaquén, Teusaquillo, Engativá and Fontibón on several concrete infrastructures, such as bridges and viaducts, a high level of corrosion by carbonation in the structures were found. Over a total of 20 structures, in 95% of cases the structures showed signs of corrosion by carbonation, highlighting that in the remaining cases, the protection provided by waterproof paints or coating protections has prevented the extension of the carbonated front into the interior of the structure (Prada, 2014; Bolívar and Cañón, 2015).

Finally, with respect to Paraguay, perhaps the situation does not differ significantly with respect to the other Latin American countries. The uncertainty of this statement is given due to the lack of scientific research on the subject and the difficulty in accessing those investigations that do not yet exist in digital format. Even so, it was possible to obtain some case studies of the region, which, combined with the empirical knowledge about the constructive method of the country by the author of this research and some interviews with professionals of the country, made possible the formulation of the problem for Paraguay.

Most of the reinforced concrete structures in Paraguay date from the 70s and 80s of the last century, a time of significant economic growth generated from the construction of the well-known Itaipu hydroelectric dam. No systematic work has yet been carried out on these structures for the study of their service conditions, especially considering the aspect of durability. In Paraguay, corrosion caused by carbonation is the predominant mechanism in the degradation of reinforced concrete structures. (Yugovich, 2007). This predominance is because chloride problems are practically null in the region due to its landlocked condition, that is, a country without a sea coast.

In some buildings analysed in Asunción, the capital of Paraguay, the corrosion of the exterior, as well as the interior structures, was verified in a generalised way. The causes of this corrosion, in addition to carbonation, were many times due to constructive negligence and failures in waterproofing. The low cover thickness on the structures, the lack of maintenance and the humidity of the precipitations considerably facilitate the advance of carbonation in buildings that, even in some cases, involved buildings with only 25 years of service. With the phenolphthalein tests carried out it was possible to confirm a carbonation depth of 20 mm and a corresponding carbonation rate of 4 mm/year$^{0.5}$. In cases of prefabricated concrete structures, it was possible to verify that in 5 years of service the structure presented high levels of carbonation due to a state of micro-cracking on the surface (Yugovich, 2007).

On the other hand, in a work carried out for the inspection of 61 concrete bridges located along the most important highway in Paraguay, more efficient performance of concrete structures has been observed. The result of the survey showed that 84% of the bridges analysed are in good condition from the point of view of corrosion, considering that it refers to bridges that exceed 50 years of lifespan. However, many of the bridges that presented carbonation and corrosion in the rebars corresponded to a relatively new group of structures (Gavilán and Baruja, 2001).

Although the levels of air pollution in Asunción do not reach the levels that perhaps occur in Bogotá, the emission of harmful gases for the health of people and for the durability of buildings increases every year along with the economic growth of the region. A study developed by the Partnership for Clean Fuels and Vehicles (PCFV) has shown that the environmental condition of the country is not promising regarding the emission of harmful gases. Thus, it is estimated that motorised sources are particularly important in contributing to the pollution of urban areas since the vehicle fleet is made up of old transportation as a distinctive feature. This last factor is also combined with a poor territorial arrangement that locates the industrial zone immersed within the urban area of the capital (PCFV, 2011).

The electrochemical corrosion of the reinforcement, induced by a carbonation process, constitutes one of the main causes of deterioration of the reinforced concrete structures, especially in those localities where the groundwater level of water are high (Solorza et al., 2003). Under this consideration, the problematic of the phreatic levels directly affect the situation in the city of Encarnación in Paraguay. The city is influenced by the hydroelectric dam built downstream of the Paraná River, which raised the level of the reservoir for the beginning of 2011, increasing the phreatic water levels accordingly. No significant studies have been carried out so far to determine the influence of this situation on the potential corrosion of structural reinforcements. For this reason, systematic studies will be necessary to evaluate the state and durability of these structures, proposing the appropriate maintenance techniques for them (Salazar, 2009). A more detailed case study regarding the degradation of concrete structures in Paraguay will be presented below.

3.7.1 Carbonation in Structures of Paraguay

Carbonation is one of the most frequent phenomenon that leads to degradation problems in structures located in Paraguay. The index of relative humidity and the tropical temperature makes a propitious place for the carbonation to propagate in the structures and jeopardize its durability due to the corrosion of the reinforcement. In this section, an analysis of the results obtained from several real cases of inspections in structures carried out by a company in Paraguay is presented. *Gavilán & Asociados* is a company that has been dedicated for several years exclusively to the maintenance and repair of structures not only in Asunción but much of the country. The author of this research had the opportunity to conduct a series of interviews with the owner of the company through which it was possible to obtain a database and photographic records of tests and inspections carried out in more than 30 different concrete structures located in the city of Asunción.

It was possible to obtain records of 206 carbonation tests carried out on 38 different concrete structures (apartments, school, parking, public building) located in the urban area of Asunción corresponding to, most of them, multi-storey buildings (between 3 to 7 stories high). Moreover, some of these buildings were unfinished structures. Unfinished structures are a typical case not only in Asunción but throughout the country. These cases correspond to private constructions of apartments or dwellings. Structures are projected and then start built but, due to lack of budget or problems with the project, these sometimes remain unfinished for several years.

The year of construction of these structures was between 1986 and 1991, although some of them were built in the current century. On the other hand, the interventions were made between 2013 and 2016. According to the engineers' expertise of the company, most of these concrete structures are made with Ordinary Portland Cement. In Paraguay, indeed, it was not usual to use pozzolanic concrete in constructions until the end of the last century, where there has been a sudden growth in infrastructures in the country, especially in the capital, city of Asunción. An overview of some of these structures is shown in Figure 3.8.

Figure 3.8 Some structures of the case study

In these inspections, measurements of concrete strength, cover thickness and carbonation depths were performed through the phenolphthalein test. Through the statistical analysis of the real carbonation data, a mean value for the cover thickness equal to 18.9 mm with a standard deviation equal to 8.04 mm was found for the structures considered. On the other hand, the carbonation depth mean value and its standard deviation were equivalent to 16.5 mm and 7.25 mm respectively. Furthermore, it is convenient to emphasise that the statistical mode among these tests, i.e. the number with the highest frequency, were a cover thickness of 10 mm and a carbonation depth value of 15 mm. The last result suggests that in several cases, the depth of carbonation is at a glance greater than the concrete cover.

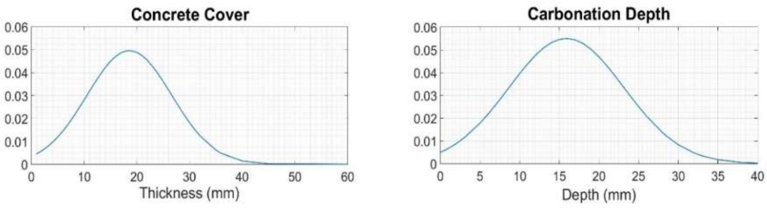


Figure 3.9 Normal distribution of test results.

In addition, these measurement results have been classified according to the structural elements of the building. After this classification, it has been seen that the slabs were the most compromised elements from the point of view of the corrosion initiation by carbonation. This is deduced since the mean cover thickness was practically equal to the mean of the carbonation depth in the structures. This classification by structural element is summarized in Table 3.3.

The database provided by the company reflects the poor quality of concrete structures in the city of Asunción. It is interesting to mention that, among the results, cover thickness measurements were even found with a value of 1 mm in several cases. This poor condition is because the control during the execution of the structures is not entirely rigorous, leading to not respecting the thicknesses calculated in the project. In fact, another study was found that in 41% of cases analysed no quality control is implemented during the placing of concrete on the construction site and only in 14% of cases was proper quality control applied (Gonzalez et al., 2008). Therefore, if the minimum concrete cover thickness required by several codes throughout the world are taken into account, for example, a value around 20-25 mm (ACI 318, 1995; BS:8110-1, 1997; European Standard, 2004; Standards Association of Australia, 2009), then it can be said that structures in Asunción have concrete cover below this minimum value.

Table 3.3 Summary of results obtained from the intervention of concrete buildings in Asunción

Structural element	N° of data	Cover Thickness (mm)		Carbonation depths (mm)	
		Mean	Standard Deviation	Mean	Standard Deviation
Beam	87	18.4	9.61	12.9	7.92
Column	88	19.2	10.78	17.6	10.07
Slab	31	12.2	5.52	12.0	7.86

Through the phenolphthalein test, it has been possible to verify positive cases and negative cases regarding the carbonation depth. Figure 3.10 shows some of these results. The worst cases, Figure 3.10(b), showed completely carbonated structures, even with the appearance of visible corrosion. Some of these structures had a cement plaster, but without positive effects against carbonation degradation. On the other hand, there were cases where the quality of the concrete cover was excellent, Figure 3.10(a), avoiding carbonation-induced degradation.

(a) (b)

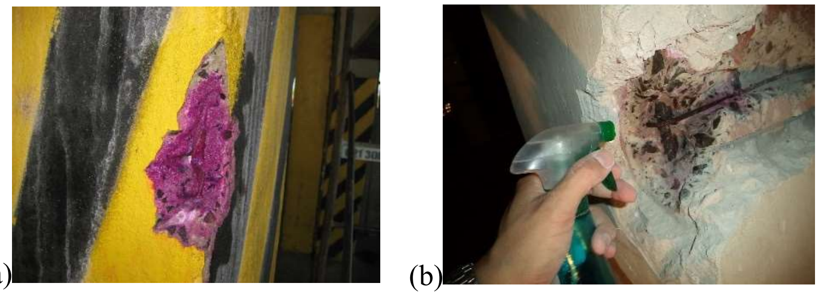


Figure 3.10 Evaluation of Carbonation depth through phenolphthalein tests.

The thickness of the cover of these structures was one of the determining factors when evaluating the degradation condition. As is known, a good cover is essential to ensure the durability of the reinforcement, and therefore, that of the structure. Many of the structures analysed in the case study shown very low values concerning the cover thickness. As it has been seen, the statistical mode of the data set was only 10 mm for the cover thickness, which can be seen in Figure 3.11 (a). On the other hand, a cover thickness that fulfils with international regulations does not guarantee the durability of the structure either. As can be seen in Figure 3.11(b), in a structure that had a 20 mm cover, it was found that the reinforcement was completely corroded. This is because a good cover thickness must be complemented by a good quality (low porosity) of the concrete.

(a) (b)

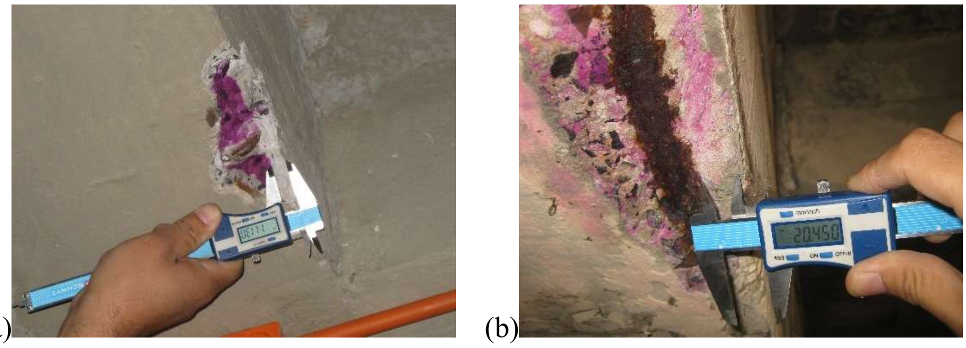


Figure 3.11 Measurement of cover thickness in the structures.

Considering the carbonation tests results, an analysis was performed in order to represent the risk of carbonation-induced corrosion of these structures and the correlation between the concrete cover and carbonation depths. It was possible to classify three conditions of degradation for the analysed structures as is shown in Figure 3.12: with corrosion risks (CR), without risks of corrosion (NCR), and the condition of imminent initiation of corrosion (IC). For this purpose, a structure without corrosion risk (NCR) has been considered when the cover of the structure is not yet carbonated or, at least, the carbonation front is not in a critical zone that may induce corrosion by carbonation. This consideration is specifically referred to that an immediate carbonation-induced corrosion is not expected. However, as has been studied in the literature, corrosion could be induced by other mechanisms and this study has focused only on carbonation.

Furthermore, the condition referring to structures with corrosion risk (CR) includes those where the carbonation front had exceeded the cover thickness. This implies that the reinforcement is wholly embedded in a carbonated concrete, leaving it vulnerable to corrosion due to the suppression of the passive protection layer. This condition of degradation is the most concerning from the viewpoint of maintenance costs and the functional capacity of the structure. The last condition referred to imminent initiation of corrosion (IC) considers what has been established in Yoon et al. (2007), in which the corrosion begins when the carbonation front is located at least within 5 mm of the rebar surface, being necessary an immediate intervention.

The term *imminent* is applied because, although the corrosion onset can be expected for the considered carbonation depth, several factors influence the accurate formulation of initiation time. For this reason, an imminent corrosion onset is considered herein when carbonation front is within 5 mm of the rebar. In summary, if a concrete element with a cover thickness of 25 mm is considered, the degradation condition is established regarding the carbonation depth (C_d). So, the structure has corrosion risk if $C_d \geq 25\ mm$; there is no corrosion risk if $0 \leq C_d < 20\ mm$; and imminent corrosion risk if $20 \leq C_d < 25\ mm$.

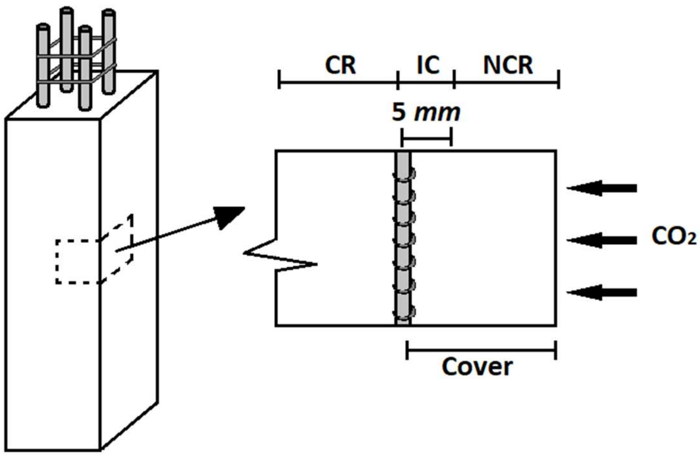


Figure 3.12 Scheme established for corrosion risk analysis.

Then, this analysis has shown that almost half of the cases analysed (49.07%) present a considerable risk of corrosion or imminent corrosion initiation caused by carbonation. That means the carbonated thickness in the concrete is higher than the cover thickness, and that the durability of the structure is in a critical condition. Thereby, it can be said that almost 50% of the buildings analysed in the urban area of Asunción, whose were built no more than 30 years ago, can be considered as structures at the beginning of the last stage (corrosion propagation) of its corrosion degradation. Such degradation conditions correspond to the time in which the interventions were carried out in the structures of the case study. Hence, the structures without corrosion risk could also be considered under carbonation-induced corrosion risk in the next years if appropriate maintenance actions are not performed. Over the 206 real carbonation data, the result of the analysis is depicted in Figure 3.13.

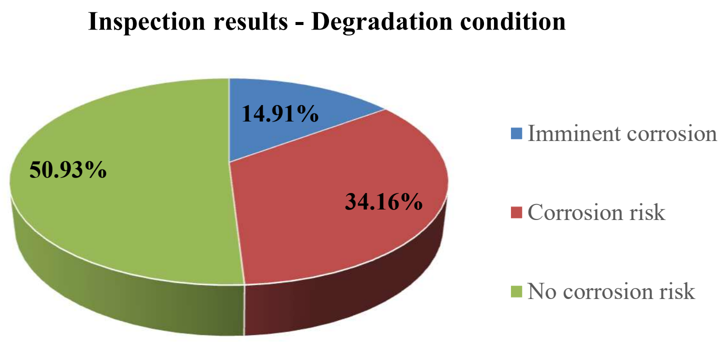


Figure 3.13 Degradation conditions of structures based on inspections results.

One of the most concerning cases detected with these real carbonation tests was a residential building located in the urban area of Asunción. The case is referred to a two-storey unfinished residential building where four carbonation and strength tests were made in columns and beams. The structural elements tested have shown good cover thickness values between 27 and 45 mm, and strength values around 30 MPa. Nevertheless, what was worrying was that it had very high values of carbonation depths (between 14 and 25 mm) considering that the structure had only five years of lifespan at the inspection time. In this sense, not just the older structures were the ones with high carbonation-induced corrosion risk.

In essence, through the case study shown in this section, it was possible to illustrate that carbonation is a common problem in the concrete structures of Paraguay. The lack of control during the execution of the works often triggers in structures without the adequate conditions that guarantee their durability. Therefore, according to the study developed in this section, the cover thickness is referred to as the most influential parameter to reduce the time to corrosion initiation. In this way, a rigorous control is recommended during the elaboration of these structures that guarantee, not only the adequate thickness but a good cover quality through a low porosity.

3.8 Summary

Since the beginning of the 20th-century, the use of reinforced concrete as the predominant material in the construction of infrastructures has allowed construction professionals two significant advantages: the prolongation of the service life of the structures and the versatility of a material that enables satisfying the most challenging construction designs. However, at the end of the same century, it was found that the durability of these structures is compromised by a series of factors and degradation agents.

In this chapter, it was developed the mechanisms and processes of degradation of concrete structures emphasising the anomaly of corrosion by carbonation. The degradation mechanisms of RC structures go beyond those previously presented, since they can cover physical and mechanical processes such as fatigue failure, ultimate limit state of resistance, wear, etc. Therefore, it should be remarked that in this chapter was described only the influential factors in the degradation of the structure by corrosion of the reinforcement. For a more comprehensive study, it is necessary to cover all the other degradation mechanisms to guarantee the durability of the structures. However, this includes a complex and extensive investigation that goes beyond the focus of this research.

Regarding corrosion, it has been shown through the bibliographic references that the processes that affect with greater consideration in this anomaly are carbonation and chlorides attack. Both anomalies are commonly confused with each other. However, they are clearly differentiated according to their process of degradation. On the one hand, the carbonation symptoms show a decrease in the alkalinity of the cement paste in the concrete reflected in the reduction of pH towards acids levels, while the chlorides attack does not cause this effect in the structure. On the other hand, the carbonation causes generalised corrosion in the reinforcement whereas the chlorides attack is given by pitting corrosion in certain parts of the rebar.

Although there are several investigations related to the corrosion of concrete reinforcements in countries such as China, the United States, Australia and European countries, at the Latin American level it was possible to confirm a lack of available research that can describe the same problems under the specific circumstances of the region. In the specific case of Paraguay, as it is a developing country, the accessibility to information and the low number of it does not allow to describe the state of degradation of the infrastructures scientifically. Even so, it was possible to make a parallel analysis regarding the studies carried out in other countries of the region.

It is worth mentioning that the construction industry in Paraguay suffers from the inconveniences associated with carrying out the work in the absence of a complete construction regulation as there is in Brazil or Argentina. In other words, Paraguay does not have its own construction standards and so engineers and architects must base its calculations on other international standards that usually are not adapted to the context of the country. Therefore, many of the constructions in the country are carried out without the supervision of a trained professional who can guarantee good practices that will contribute considerably for the durability of the structures. Nevertheless, in the last decade, the economic growth in the country's capital and in its main cities allowed the execution of works under standards, safety and quality requirements of the structures. This was caused due to the propulsion of foreign investors who often demand more control over their projects.

The results of the case study in Paraguay presented on this chapter reveal that quality control during the construction process is of utmost importance to ensure a minimum concrete cover, which is one of the most critical factors on the initiation of corrosion time. Another point of interest is the need for maintenance and rehabilitation of structures and infrastructure. The optimised planning of maintenance in infrastructures provides the extension of the service life of buildings and allow for saving money in expensive retrofitting. Furthermore, structures and infrastructures in Paraguay are not appropriately designed and executed according to the consideration of environmental effects. The lack of preventive and corrective maintenance compromises the durability of these structures. These aspects are highlighted, not only through the literature review but also through the on-site tests carried out on structures in Paraguay.

Another emerging requirement within the construction industry in Paraguay is the application of *post-sale* theory in this market. That is, design the structures from the conception of the project considering an adequate plan of inspection and maintenance that ensures the durability and functionality of the structures, prolonging their service life. In this chapter, it was shown that the processes and mechanisms of degradation of structures could be expressed regarding time. If attention is rewarded to the development of these anomalies, it will be possible to establish the precise times for the application of preventive interventions in the infrastructures. This research deals with this matter, which seeks to propose optimal planning of inspection and maintenance actions according to the mathematical models that describe the physical and chemical phenomena of degradation.

It is necessary to look out when defining the degradation mechanisms of a structure so that scheduled maintenance seeks to eliminate the causes of the anomaly from its conception. Also, and as it is in the case of Paraguay, it is not reliable that the diagnosis of the degradation of the structures be made considering only the process of carbonation, arguing that it is a country that does not have maritime zones. This is because, although the chlorides attack is given by the presence of salts in the concrete, in previous decades it was common in practice to use curing accelerator additives with chloride content, which implies that many of the structures currently in service in Paraguay could suffer from this mechanism. However, despite these exceptions, this research is focused on the most recurrent degradation problem that is carbonation in concrete structures.

Regarding the inspection carried out on the bridges presented in a case study, an important factor to note is that a large part of the bridges in Paraguay with a lifespan of more than 50 years present a considerably higher performance compared to other relatively younger bridges. This is because good practices in the preparation and implementation of concrete have decreased in recent decades. Also, in this chapter, the influence of parameters such as the cover thickness and the compactness of concrete in the vulnerability to degradation has been demonstrated. However, this referred study was carried out on bridges located predominantly in rural areas, which requires particular attention to the results considering the level of environmental aggressiveness that occurs in this type of area concerning urban areas.

Finally, it is important to be clear about the mechanism of degradation of concrete structures when corrosion is treated. The aspects that must be considered include the economic implications of repairing the damage, the need to a more comprehensive knowledge towards an anomaly, the importance of the use of a specific material in all types of structures, and the attenuation of corrosion agents due to the effects of climate change. Furthermore, the need for mathematical models that allow simulating the mechanism of corrosion degradation in a way closer to reality.

This chapter has been developed with the purpose of identifying the most common degradation mechanisms RC structures. Among these mechanisms, corrosion degradation has been discussed in depth since it is the main topic of this investigation. Furthermore, considering that the study is oriented to the structures of Paraguay, the analysis mainly covers the corrosion by carbonation since, as it has been seen in this chapter through the case study, it is the main degradation mechanism in these structures. The results of the case study addressed in this chapter, also, will be useful to validate the reliability of the numerical carbonation model that is exposed in the following chapter.

CHAPTER 4

Modelling of Carbonation-induced Corrosion

CHAPTER 4

4 MODELLING OF CARBONATION-INDUCED CORROSION

4.1 Introduction

This chapter was prepared with the purpose of presenting a review of the models that have been developed by different researchers around the world. The main objective of this research is to develop a useful support guide for the elaboration of maintenance strategies in the structures. However, to achieve this goal, it is essential that the degradation mechanism in the structure be appropriately modelled. In the literature, it is possible to find several models that have been developed to describe the degradation process through mathematical and chemical formulations. At the end of this chapter, one of the numerical models described will be used as a reference to formulate the degradation curves for the RC structures of Paraguay corresponding to the context of this investigation. For this reason, an analysis is carried out on the parameters and variables considered in the chosen carbonation model, to validate them according to specific case studies carried out in Paraguay, as shown in the previous chapter.

In recent decades, the modelling of the degradation process of concrete structures by several mechanisms has been one of the main concerns in countries such as Australia, China, the United States, Canada and several countries in Europe. For the case of the structural degradation caused by corrosion, Kyösti Tuutti has been one of the first researchers to establish the bases of knowledge representing the service life of the structures under the corrosion perspective in two stages: initiation stage of corrosion and propagation stage (Tuutti, 1982). Many investigations have been developed since then on the prediction of the service life of the structures. However, the applied methods cannot be considered as an exact science due to the role of uncertainty within the variables found in the study, which leads to the prediction of service life as a multidisciplinary process.

The methods for the prediction of the service life can be categorised generically into three broad groups: deterministic methods, probabilistic methods and engineering methods. The deterministic methods focus on an analysis of the parameters that influence the degradation of the elements, the mechanisms that intervene in the process and the determination of performance through degradation functions. Probabilistic methods, on the other hand, consider degradation as a stochastic process, in which the probability of degradation is formulated for each element during a specific period of time. Lastly, the engineering method seeks complementation between the first two methods, to identify the degradation phenomena analytically, using the maintenance planning as a control mechanism (Cecconi, 2002; Daniotti, 2003).

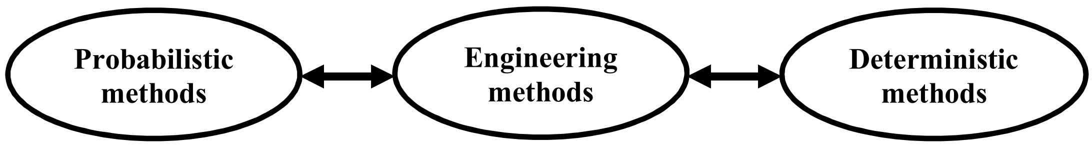


Figure 4.1 Relationship among the models for the prediction of service life (Hovde and Moser, 2004).

Due to the stochastic nature of the corrosion-induced degradation in concrete structures, the probabilistic method is the most appropriate and most used method for this type of analysis. In turn, within the probabilistic method, the Markov model is perhaps the most used by researchers for the prediction of service life, which focuses on a particular type of material and a corresponding exposure environment. Among the fields of application of this method, it can be mentioned the prediction of the service life of the concrete structures, the component elements of the building envelope, pavements, among others. This method is based on the hypothesis, which defines the degradation model from a limited number of conditions. Likewise, the method assumes that degradation is a stochastic process governed by random variables (Noortwijk and Frangopol, 2004).

In 1998, Lounis *et al*. were able to demonstrate that the probabilistic approach used until then only to establish the safety parameters in the structural calculation, could also be applied in the prediction of the durability of the structures. In this way, it was stated that to ensure a proper prediction of the service life, it was necessary to maintain the probability of failure of the system within a reasonable time interval that is below a specific limit value, which in turn depends on the consequences of such a failure (Lounis et al., 1998).

Then, considering the degradation process addressed in this research, it is possible to define the corrosion initiation period as the time required within the degradation process for the depassivation of the reinforcement due to carbonation or chlorides attack. Once all the concrete cover is carbonated, or the chloride content exceeds its critical amount, then the steel corrosion process begins in the structure. Then, from this moment begins the process known as propagation period, which includes the corrosion onset until the end of the service life indicated by the cracking of the cover. The accumulation of oxide in the rebar generates expansive stresses that fissure the cover. Consequently, when the critical stress value is exceeded, its detachment occurs. Finally, the collapse or failure of the structural element occurs when the bonding between concrete and steel is lost, or when the loss of rebar section decreases the strength capacity of the structure (COIN Project, 2008).

It should be noted that the study of the corrosion process of concrete has been studied since the middle of the last century. In 1965, the *German Research Institute of Cement Related Industry* performed an investigation on existing concrete buildings measuring the carbonation depth (Steffens et al., 2002). Carbonation, as a meaningful degradation mechanism in the durability of concrete structures, has led to the formulation of mathematical models that allow establishing the acceptable limits regarding the damage degree of a structure. Thus, degradation models have been improved considerably since the first schematic model established by Tuutti. Currently, the numerical models seek to enhance their ability to predict and represent reality by combining a set of parameters involved in the chemical, physical and mechanical corrosion of structures. Nonetheless, there are still discrepancies between these models regarding the values adopted for certain parameters, the threshold of damage established, the validation mechanisms of the models, and the conditions selected for the formulations.

Regarding the model proposed by Tuutti, the first stage of degradation has practically not been modified by the subsequent models conceptualised under the same scheme. However, the propagation stage has been comprehensively studied, and several researchers have proposed new alternatives for modelling the degradation. Several studies carried out in recent years divide the propagation stage into several sub-stages, for which several models have been formulated that estimate the service life of a structure. These sub-stages that comprise the propagation period could be considered according to four limit states as shown in Figure 4.2: (1) corrosion onset due to depassivation of steel, (2) initiation of cracks formation, (3) spalling of the concrete cover, and finally, (4) collapse of the structure due to bonding failure or sectional loss of the reinforcement (*fib*, 2006).

The duration of the propagation stage depends mainly on the corrosion rate, whose nature is very complex and depends on several factors. For this reason, estimating the duration of the propagation process using reliable models is usually a difficult task. This is because the incorporation of relevant factors that are adapted to the limit states usually has a subjective character. For instance, for the repair of a structure, the limit state corresponding to the failure or collapse of the structure cannot be adopted as a threshold damage degree for repair actions, mainly due to reasons related to safety.

Then, for the maintenance and repair planning of the structure, a limit state before the formation of cracks should be considered, when it is still possible to recognise the failure through non-destructive methods (NDMs).

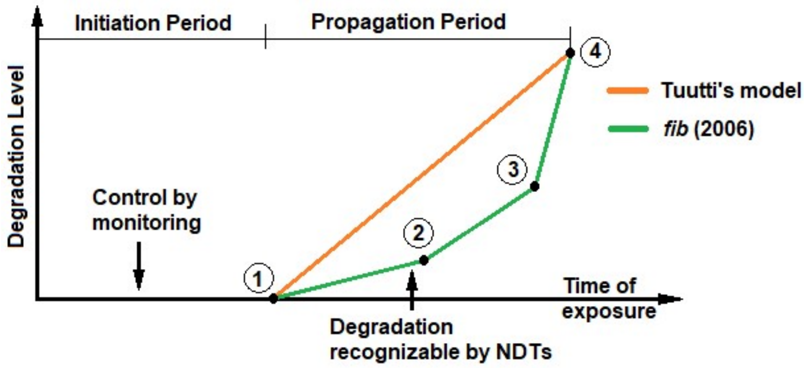


Figure 4.2 Stages and sub-stages in the service life of corroded concrete structures (Tuutti, 1982; fib, 2006).

When choosing a degradation model for the analysis of a structure, it is essential to consider that many of these models available for the prediction of the service life adopt only an indicator of damage for which they are validated. That is, it is important to understand the situation for which a certain type of model was designed, as well as the parameters and validation mechanisms adopted. In other words, the choice of a numerical model for analysis is a fundamental stage in the study of the buildings degradation.

As mentioned, and as will be explained in the sections of this chapter, the study of the degradation of buildings based on the corrosion of the reinforcements has already been entirely developed worldwide. However, the studies that take into consideration the degradation of the structure according to the effects of climate change are still limited. As discussed in Chapter 2, climate change is a phenomenon that involves several sciences that go beyond the merely climatological. In the case of engineering infrastructures in general, the phenomenon of climate change has a direct impact on the durability of the structure and considering that it is a relatively "new topic", many infrastructures will need to be adapted to these new requirements in the coming years.

The main advantage of models that consider the climate change effects on the prediction of service life is that they have a more accurate representation of reality than previous models. The combination of several environmental parameters within the model, considering the dynamics of these values given by climate models, will allow being able to program the maintenance and repair activities of the structures correctly. Nevertheless, due to the stochastic nature of climatic phenomena, working with these types of models is not an easy task. Many mathematical models seek to describe the performance of buildings against degradation through simulations or predictions of corrosion propagation stages. Likewise, degradation models can be grouped into three broad categories: analytical, numerical or empirical (Köliö et al., 2014). Table 4.1 presents this classification and its corresponding subdivisions.

In this classification, the numerical models comprise a series of mathematical equations that give an approximate solution of the parameters as a function of time through numerical simulations. The empirical models are based on the consideration of the relationship between the corrosion rate and the basic parameters of the concrete. They are usually developed using a database of laboratory experiments that isolate the parameters that influence corrosion. Finally, the analytical models consider the fact that the crack accelerates typically the penetration of harmful agents to the concrete promoting the corrosion and later the detachment of material, i.e. the spalling of cover (Raupach, 1996; Bjegovic et al., 2006; Warkus et al., 2006; Köliö et al., 2014).

Table 4.1 Classification of corrosion propagation models.

Category	Modelling method	Description
Numerical Models	Boundary Elements Method (BEM)	The objective is to satisfy the differential equations based on corrosion potential constant, current density, density-potential relationship (Redaelli et al., 2006).
	Finite Elements Method (FEM)	It provides the ability to vary the properties of the concrete volume easily. The numerical size of the model can be significant and, therefore, expensive. Conditions may include temperature, chloride content, relative humidity, concrete resistivity and electrochemical behaviour (Isgor and Razaqpur, 2006; Redaelli et al., 2006).
	Network of resistors and transmission lines	The relationship between the corrosion potential, the resistance of the corrosion systems and the electric current of the macrocell can be calculated on the basis of simplified electrical circuits (Raupach, 1996).
Empirical Models	Fuzzy Logic Models	It has been applied for the evaluation of corrosion-induced degradation and to estimate the loss in the cross-sectional area of steel (Bjegovic et al., 2006).
	Delphic Models	It is a complex model generally used when the corrosion rate is estimated based on experiences of past years.
	Models based on electrical resistivity and/or oxygen diffusion	In this model, it is assumed that the electrical resistance and oxygen diffusion of the concrete is the main control parameters for the corrosion process (Scott, 2004).
Analytical Models	Cracked Elements	The models are based on mathematical equations where concrete is modelled using the thick-walled cylinder approach. These models should be calibrated based on the results of the experimental tests (Bhargava et al., 2006).
	Non-cracked Elements	

In turn, each of these methods for modelling the degradation of structures has its advantages and disadvantages. For example, the numerical boundary element method has the advantage over the finite element method because for its application requires a smaller number of elements and the two-dimensional elements can be used for three-dimensional simulation of the problem. Another advantage of the method is that the processing cost and mesh generation are considerably reduced. However, the disadvantage of the method is that it requires that the conductivity must be considered as a constant, whose results must necessarily be treated in discrete points (Warkus et al., 2006). Regarding the network method of resistors and transmission lines, its main shortcoming is that the geometry of the corrosion cells is not considered within the model. That is, the geometry of the galvanic cell determines the current, which significantly affects the behaviour of corrosion (Jia et al., 2006).

On the other hand, the empirical methods for corrosion modelling have the disadvantage that the variables considered are studied in isolation regarding other parameters. This, in turn, influences the process of degradation, which is the reason for these models are generally limited to a set of specific conditions under which they are developed. For this reason, the use of these models must be carried out carefully to avoid erroneous estimations regarding the service life of the structure. For example, a structural failure that jeopardises the safety of its occupants due to an overestimation of the service life.

Lastly, a limitation of the analytical method is given by the inability to consider the nonlinear behaviour of the concrete, which is given when the radial cracks begin from the inner surface of the cylinder. In the same way, another handicap consists in the assumption of flat stress formation, since under conditions of the problem, the formation of flat deformation would be more suitable (Tepfers, 1979; Chernin et al., 2010). Following, the content of this chapter includes a review of several types of numerical models that have been developed with a description of the methodologies and parameters considered in these models. Subsequently, one of these models will be chosen, which will be used for the development of the degradation curves. Then, this chosen model will be presented within a section in this chapter describing the mathematical formulations used to analyse both the initiation and propagation of corrosion process. Finally, the results obtained with this model will be analysed through the correlation of the data obtained concerning real carbonation data in Paraguayan structures.

4.2 Modelling the Carbonation-induced Corrosion

The importance of reinforced concrete as a constituent material of several infrastructures has motivated the study of the degradation process of this material. As the knowledge on the corrosion phenomena of reinforcement in concrete has become more and more relevant during recent years, different degradation models have been performed to enable scientists and engineers to describe the effects of corrosion quantitatively (Raupach, 2006).

Modelling the corrosion process is not a straightforward task since the main objective of these models is to represent reality reliably so that, with the estimation of the service life, corrective measures can be taken to guarantee durability, safety and functionality of a structure. Many numerical, empirical or analytical models for the study of degradation can be found in the literature. In this section a reference is made only to the degradation models formulated for carbonation-induced corrosion, leaving the other mechanisms mentioned in the previous chapter exempt.

Table 4.2 presents an overview of some of these models found in the literature in which the method and the result of the investigation are described. This review is presented in chronological order to appreciate how the study has been improved through an increasingly accurate approach to reality. Although some authors did not elaborate a degradation model, their contribution helped to demonstrate the reliability or not of some developed models, so they were also included in this review.

Table 4.2 Progression of modelling of carbonation-induced corrosion.

AUTHOR	METHOD	OUTCOMES
(Tuutti, 1982)	Schematic model based on the flow of penetrating substances in the concrete cover.	The service life is divided into an initiation stage and a propagation stage of corrosion.
(Molina *et al*., 1993)	A numerical model based on the finite element technique with a smeared-crack approach.	The effect on the degradation of the crack-width rate caused by the oxide was evaluated quantitatively. A final validation has not been possible.

AUTHOR	METHOD	OUTCOMES
(Lindvall, 1998)	A quantification of parameters is performed for the model developed by the DuraCrete project.	It comprised one of the first attempt to make a probabilistic performance based on the durability design of concrete structures.
(Alonso *et al*., 1998)	Quantification of the relationship between the amount of corrosion and the cover cracking was developed considering the influence of some parameters in the generation and propagation of the crack-width.	For radius losses of the cross-section of the rebar, approximately 15-50 μm of oxide is necessary to generate the first visible crack (width <0.1 mm). There is a linear relationship between the growth of the crack-width and the rebar radius losses during the propagation of the crack.
(Steffens *et al*., 2002)	Using equations of equilibrium and laws of diffusion, a theoretical model was formulated, which is solved by the numerical method using finite elements and numerical integration techniques.	The wet parts of structures due to rainfall significantly affect the development over time of the depth of carbonation. The model was successfully validated through the results of experimental tests.
(Liang and Lin, 2003)	A one-dimensional mathematical model was developed that uses a linear partial differential equation based on the principle of mass balance and the connective-dispersive equations.	The model can chemically and physically describe the process of carbonation in concrete. The solution transport process can be expressed more accurately than the analytical solution used with Fick's second law.
(Bary and Sellier, 2004)	The model is based on macroscopic mass balance equations. These equations are discretized in time and space in the one-dimensional case.	Progressive filling of the pores in the carbonated zone was demonstrated, which has a direct impact on humidity, CO_2 concentration and calcium transfer properties.
(Saetta and Vitaliani, 2004, 2005)	A numerical mathematical model with a stochastic approach was developed. The variability of the parameters in the differential equations was evaluated.	The validation was made with real cases. One of the environmental parameters that most affect the carbonation rate is the concentration of CO_2 in the air.
(Yoon et al., 2007)	Mathematical equations based on the first Fick's law of diffusion have been used to determine the coefficient of diffusion of CO_2 in concrete under microclimatic conditions.	It was found that carbonation-induced corrosion begins once the carbonation front is located at least within 5 mm of the rebar surface.
(El Maaddawy and Soudki, 2007)	A mathematical model is developed based on the relationship between the loss of steel mass and the internal radial pressure.	The onset of cracking is defined as the time for the corrosion products to fill a porous zone before it begins to induce an expansive pressure on the cover.

AUTHOR	METHOD	OUTCOMES
(Park, 2008)	Carbonation model for the diffusion reaction in concrete based on the finite element method. Estimate the carbonation depth through basic differential equations.	The effect of the cover on the reduction of carbonation can be represented with high precision, with an estimated calcium hydroxide diffusion coefficient equal to $1.10^{-12}\ m^2/s$.
(Bahador, 2008)	A simulation model to measure the saturation degree in concrete for different moments and depths under a wide range of environmental conditions.	The carbonation model allows predicting the carbonation depth in natural and accelerated conditions accurately.
(Marques and Costa, 2010)	Safety factors and probabilistic approaches were used to estimate the lifespan of concrete structures.	Corrosion begins when along the reinforcement there is a significant difference in electrical potential with enough presence of oxygen and moisture.
(Kwon and Song, 2010)	A numerical technique for the study of the carbonation behaviour using the algorithm of neural networks and modelling of carbonation, considering the changing effect on porosity.	It provides a straightforward estimation of the diffusion coefficient and the reasonable prediction of the carbonation depth. It was demonstrated that the decrease in total porosity under carbonation is 80-86% regarding the initial condition.
(Chernin and Val, 2011)	A nonlinear finite element model is developed for the prediction of corrosion-induced cover cracking.	Modelling the expansive behaviour of the corrosion products by applying internal pressure or radial displacement leads to an incorrect assessment of the stress-strain state in the concrete.
(Stewart et al., 2011)	A probabilistic approach based on reliability that predicts the probability of corrosion onset and damages due to severe cracks in concrete structures.	It was found that the damage risks induced by carbonation could increase by more than 400% by the year 2100 in some regions of Australia due to the climate change effects.
(Lollini et al., 2012)	The role of design parameters is analysed in models developed by *fib* for existing structures subjected to carbonation.	The concrete cover thickness had a significant effect on the output of the service life model.
(Wang et al., 2012)	Time-based probabilistic analysis to evaluate the probability of corrosion onset and corrosion damage in existing concrete infrastructures in Australia, considering changes in climatic variables.	It has been shown that corrosion rates could increase by more than 15% if the temperature increases by 2 °C. If the climate change effects have not been considered, existing concrete structures could deteriorate more quickly than anticipated.

AUTHOR	METHOD	OUTCOMES
(Talukdar et al., 2012a; Talukdar et al., 2012b)	A deterministic one-dimensional numerical diffusion model for a gaseous medium through a porous substrate based on Fick's Second Law has been developed. The model considers the variation in time of environmental parameters given by the climate change effects.	Applying an urban simulation in Canada, it was shown that climate change would affect the progress of carbonation in structures that could be up to 45% higher in 100 years.
(Talukdar and Banthia, 2013)	A service life model was developed to assess the climate change effects on the structures of various cities around the world.	Corrosion is influenced mainly by the temperature, saturation and resistivity of the concrete, and the cover thickness. In cities where carbonation rates are high, a lifespan reduction of 15-20 years could be expected.
(Bastidas-Arteaga *et al.*, 2013)	Methodology to estimate the effect of increased CO_2 concentrations and global warming on the durability and structural safety of concrete structures.	Global warming could reduce the estimated time to failure of the structure above 31% or decrease the life-cycle above 15 years for moderate levels of environmental aggressiveness.
(Marques *et al.*, 2013)	Two performance-based methods (safety factor and probabilistic method) were used to model long-term performance in carbonated concrete.	For cement with 50% fly ash content, the durability periods are within the minimum required by the standards, although still with lower values than with Portland cement.
(Décatoire *et al.*, 2014)	The predictive capacity of the degradation model proposed by the DuraCrete project was analysed from simulated measurements with a finite element method.	The kinetics of the DuraCrete model is not satisfactory enough to have confidence in long-term predictions. The models will have to be changed considering the changing environmental conditions.
(Silva *et al.*, 2014)	A statistical model that can estimate the carbonation rate through a set of conditioning factors based on multiple linear regression analysis was performed.	With a value above the 70%, the relative humidity is less critical since the humidity of the environment is enough to slow down the carbonation process.
(Köliö et al., 2014)	Applying the DuraCrete model, the degradation in facades and balconies in Finland was analysed by means of statistical simulation methods.	The model presents an overestimation in the prediction of the propagation stage in all the structures studied for the Finnish climatic conditions.
(de Larrard *et al.*, 2014)	Finite element carbonation model with a probabilistic and reliability approach which considers the	It has been shown that the parameter with the most significant influence on structural safety corresponds to the concrete cover thickness.

AUTHOR	METHOD	OUTCOMES
	uncertainties inherent to the degradation process.	
(Czarnecki and Woyciechowski, 2015)	A mathematical model based on the hyperbolic model of carbonation that considers the results of the research carried out in both accelerated and natural conditions.	The gradual lowering of the CO_2 diffusion ratio and the rate of carbonation tending asymptotically to zero as a function of time has been found.
(Phung *et al.*, 2016)	The one-dimensional model under CO_2 pressure conditions based on a macroscopic mass balance for CO_2 and humidity.	The model can predict the change in microstructures and transport properties. The limestone load promotes the absorption of CO_2, but reduces porosity and transport properties.
(Talakokula *et al.*, 2016)	The corrosion evaluation is based on the mechanical impedance parameters of electromechanical coupling of a piezoelectric lead zirconate titanate (PZT) ceramic patch bonded to the surface of the rebar. The equivalent stiffness parameter and the equivalent mass parameter were considered.	The first stage of carbonation penetration through the cover has been determined as well as the second stage of corrosion onset. The method is non-destructive, simple and completely autonomous.
(Peng and Stewart, 2016)	Time-dependent analysis based on Monte Carlo simulation. It comprises the uncertainty of the climatic projections, the degradation process, the materials properties and the predictive models.	The average carbonation depths by 2100 could increase by 45% for concrete structures in China due to climate change, which means that it could cause additional damage to 7-20% caused by carbonation.
(Lloro *et al.*, 2016)	Inspections have been carried out on concrete bridges in Argentina to validate the DuraCrete model for carbonate structures through a semi-probabilistic approach.	For concrete elements unsheltered from weathering, the actual carbonation depth and the results of the model showed a significant difference, being the latter being clearly overestimated.
(Jiao *et al.,* 2016)	An improved model of carbonation prediction was developed based on reliability theory for the formulation of the time-dependent reliability of concrete structures.	The probability of accumulated failure due to carbonation could increase in the future according to the carbon emission scenarios. To do this, the strength of the concrete and the thickness of the concrete cover must be increased.
(Faustino *et al.*, 2017)	Modelling based on carbonation tests and modelling based on air permeability tests has been compared using safety factors	A convergence between the models was demonstrated, which means that both current models could be an alternative to each other.

AUTHOR	METHOD	OUTCOMES
(Tang et al., 2018)	Using least-square polynomial regression and based on theoretical analysis and experiment results, a model to predict the carbonation depth of recycled aggregate concrete (RAC) under axial stress was established.	The experimental data demonstrate that the carbonation depth increases with the carbonation age and that the carbonation resistance of RAC is lower than that of natural aggregate concrete under the same experimental conditions.
(Suvash et al., 2018)	An empirical model was designed using an automated neural network search (ANS) to investigate the effect of concrete mix compositions, weathering effect and exposure time on carbonation depth in concrete.	The model has shown reasonably accuracy and robustness. Also, it was found that the carbonation process can be controlled by choosing the right composition of concrete mix.
(Ekolu, 2018)	An experimental carbonation model based on square-root function of time for carbonation, growth rate function for compressive strength, a parabolic function for carbonation dependence on humidity and inverse power function for carbonation conductance has been performed.	The model has been validated through 163 datasets taken from a 10-year carbonation study of existing RC structures. It is shown that the proposed model is comparably accurate and involves mainly basic tests with no significant anticipated costs.

Although each of these models has its advantages and disadvantages, they have been able to make a significant impact on the progress of research in the field of the corrosion-induced degradation of concrete structures. It is not a simple task to formulate the behaviour of a structure and its degradation mechanism mathematically, primarily when the variables involved representing certain randomness that makes it difficult to predict their performance. However, these studies that have been developing are attaining to represent, with increasing reliability, the performance of the structures under degradation considerations.

This chapter was developed with the main objective of establishing state of the art on the degradation models developed within the study of corrosion as a degradation mechanism of concrete structures. In turn, the purpose of the research is to develop a maintenance methodology, for which it is necessary first to establish a degradation model of the structure based on a mathematical model, For this reason, the mathematical model adopted for the development of the degradation curves of concrete structures was the one developed by Talukdar *et al.* in 2012 (Talukdar et al., 2012a; Talukdar et al., 2012b).

Despite the model has a certain complexity in its formulation, it has the advantage of considering many parameters and variables that determine the degradation mechanism. Furthermore, it was applied and validated for different climate conditions in several countries which means that the carbonation model has considerable reliability and versatility. Even so, it is necessary to validate the degradation curve obtained in the model concerning real data of the carbonation advance in structures of Paraguay. Finally, the determining factor that led to the choice of the aforementioned model was that it directly considers the effect of climate change on the degradation of the structure. The mathematical formulations of the model not only include the interference of climate parameters in the degradation process but also consider the variation of these parameters in the future as a consequence of climate change.

4.3 Carbonation model adopted

In this section, it is presented the mathematical formulations corresponding to the degradation model adopted to develop the maintenance strategies of this thesis. This model considers the potential consequences of global climate change on concrete infrastructures, establishing as its main objective the determination of the probability of increasing the degradation risk due to carbonation-induced corrosion. It is a deterministic one-dimensional numerical diffusion model for a gaseous medium through a porous substrate based on Fick's Second Law. The model has been validated based on experimental results obtained in an accelerated carbonation chamber. Also, this model was applied to the environmental conditions of cities with different locations around the world (London, New York, Mumbai, Sydney, Toronto, Vancouver) to determine which of them represented the greater vulnerability to the problem considered.

This model comprised a significant advance in the investigation of carbonation-induced corrosion in RC structures, both considering a series of parameters and variables simultaneously, as well as considering the effect of climate change, which is a matter of global concern of the last years. In this way, studies developed with this carbonation model have achieved to demonstrate that global climate change will increase the progress of degradation by carbonation in the coming decades.

The carbonation model is structured by the principles of degradation formulated by Tuutti in 1982, which covers the first stage of the corrosion initiation and the second stage of propagation. Perhaps, the most complex analysis comprises the first stage, since it considers a high number of random variables such as porosity, the effect of temperature, relative humidity and the concentration and diffusion of the gases involved in the process, namely the carbon dioxide and calcium hydroxide $Ca(OH)_2$. For the second stage, instead, the time taken for the formation of an amount of oxide greater than that allowed to generate the crack of the cover is considered. An overview of the structure of the carbonation model is depicted in the figure below.

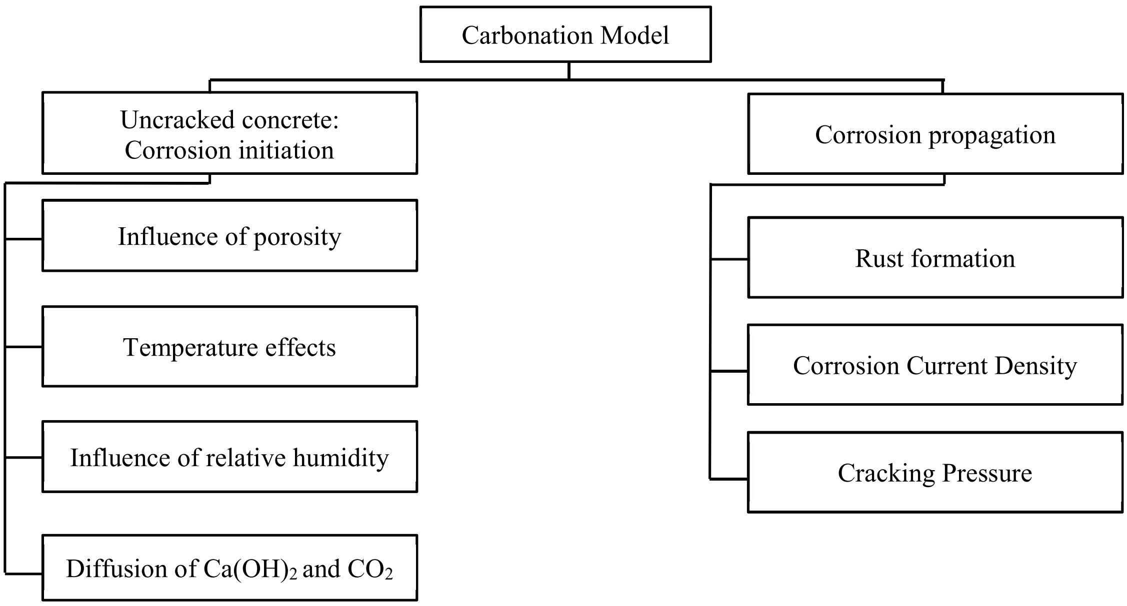


Figure 4.3 Flowchart of the carbonation model applied in this research (Talukdar, 2013).

Furthermore, it is worth mentioning that within the recommendations for future work formulated in the research developed by Talukdar (Talukdar, 2013), it is proposed to use a similar approach of this model to determine the optimal maintenance schedule to lower costs associated with increased carbonation-induced damage due to global climate change for cities throughout the world. This proposal motivated the development of this research, transferring the proposed methodology to the climatic and constructive context in Paraguay.

Although within the research developed by Talukdar the model was formulated under the consideration of uncracked concrete structures, an attempt was also made to determine the capability of the model for the prediction of carbonation advance in cracked structures. However, it was shown that using the concept of effective diffusion coefficient was not permissible for concretes containing structural cracks. Also, it is important to mention that the model was validated for concrete made with non-pozzolanic cement (Ordinary Cement Portland) without consideration of the load situation to which the existing structures are subject. Thus, the main feature of this carbonation model is focused on the environmental perspective, for which the model has been experimentally validated under conditions of multiple, simultaneous, time-varying concentrations of CO_2, temperature, and humidity.

4.3.1 Corrosion Initiation Stage

This stage comprises the first part of the degradation process of a structure due to the corrosion of its reinforcement. In this stage is considered the time that elapses so that the pH of the concrete mass decreases to a value that causes depassivation of the reinforcement, which is generally considered as pH ≤ 8.0. This phenomenon occurs due to a chemical process that mainly involves a series of parameters such as the concentration of gases, porosity, humidity and temperature. For the proper understanding of the application of the mathematical model, the author has proposed a flowchart that facilitates the use of the different proposed equations, as shown in Figure 4.4.

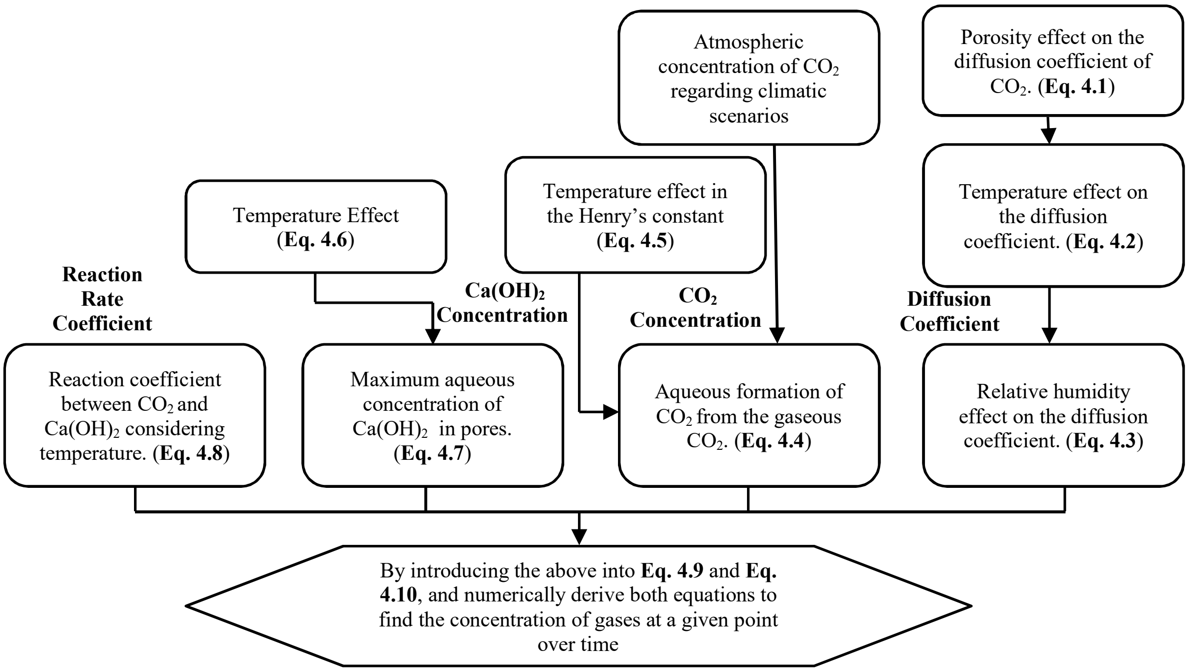


Figure 4.4 Flowchart for the concentrations determination of CO_2 and $Ca(OH)_2$ as a function of time (Talukdar, 2013)

For the application of this model, it is necessary to previously define some input values that will allow the degradation curve obtained to be representative of each specific situation, from the constructive as well as the climatic point of view. In this way, certain values will depend on a specific geographic location for each case, such as the relative humidity of the environment, the concentration of carbon dioxide in the atmosphere, the cover thickness of the structural element, cement content, water/cement ratio, among other variables. The input values adopted for the context considered in this investigation will be presented later. Also, the values that are used within the degradation model such as enthalpy, reference temperature, diffusion activation energy, among others, are summarized in Table 4.3, which is at the end of this section.

4.3.1.1 Influence of Porosity in the diffusion process

The diffusion of gases through the pores of a material is a complex process that can be classified according to three different transport mechanisms. Firstly, the Knudsen diffusion takes place if the pore dimensions are smaller than the mean free path of the gas molecules where the coefficients of diffusivity and permeability are the same. On the other hand, the molecular diffusion occurs in large pores, where the mean free path of the gas molecules is smaller than the diameter of the pores, thus giving a collision between molecules. In this diffusion, it is assumed that the absolute pressure is the same in all parts of the pore. Lastly, surface diffusion can also occur at the same time as the previous ones. If the pore surfaces are active, the gas molecules can diffuse along the walls of the pores through a succession of absorption-desorption reactions from one active site to another. However, in larger pores, the surface diffusion becomes negligible (Houst and Wittmann, 1994).

The pores distribution within the concrete affects the diffusion rate of gases into the material. As seen in the previous chapter, the formation of pores within the structures depends mainly on the way in which the concrete is placed during its elaboration, that is, the amount of mixing water, the process of curing, the water/cement ratio, etc. Likewise, it has been seen that the porosity of the cover is fundamental to avoid the early corrosion initiation since only with a suitable cover thickness, the protection of the reinforcement is not guaranteed.

Another aspect to consider is that studies have shown that carbonation reduces the porosity of concrete over time. It has been shown that the total porosity of the concrete under the influence of carbonation can decrease above 80% compared to its original state (Kwon and Song, 2010). To consider this, the carbonation model takes into account an empirical expression (Equation 4.1) for estimating the effective diffusion of carbon dioxide under the porosity influence (Papadakis et al., 1991b):

$$D_{CO_2} = A_D \left(\frac{V_p}{\frac{c}{\rho_c} + \frac{w}{\rho_w}} \right)^{\alpha_D} \tag{4.1}$$

Where A_D and α_D are empirical parameters derived experimentally, V_p is the pore volume in the cement paste (m^3), c is the cement content (Kg), w is the water content (Kg), ρ_c is the absolute density of the cement (Kg/m^3), and ρ_w is the water density (Kg/m^3).

4.3.1.2 Temperature Effect

Temperature, as the porosity, has a direct impact on the carbon dioxide diffusion inside the material. Small changes in ambient temperatures do not significantly affect carbonation, but high temperatures increase the carbonation rate. Therefore, for arid climates, the influence of temperature becomes more relevant. However, in temperate climates, the carbonation rate would be lower in winter season due to a combination of factors including low relative humidity and low temperatures which generally delay chemical reaction activity in the outdoor weathering. Typically, regions subject to elevated temperatures such as 30-40 °C would be expected to exhibit a relatively increased carbonation rate compared to regions of lower average temperatures such as 18-25 °C (Ekolu, 2018).

Therefore, an increase in the ambient temperature induces the acceleration of the diffusion process due to the molecular activation that causes this increment. Thus, the effect of temperature on diffusion may be expressed by the Arrhenius relation according to the following expression:

$$D(T) = D_{ref} e^{\left[\frac{Q}{R_1}\left(\frac{1}{T_{ref}} - \frac{1}{T}\right)\right]} \tag{4.2}$$

Where Q is the diffusion activation energy (J/mol. K), R_1 is the gas constant (J/mol. K), D_{ref} is the diffusion coefficient takes as a reference (m^2/s), T_{ref} is the reference temperature in Kelvin degrees (K), and T is the temperature of interest (K).

4.3.1.3 Effect of relative humidity (RH)

The relative humidity of the air determines the moisture that is contained in the pores of the concrete. The relative humidity changes with variation in the environmental conditions. Thus, a cyclic wetting and drying exposure could lead to the fluctuation in RH of the concrete. Under such conditions, carbonation phenomenon stops when the concrete is saturated and only continues when the concrete dries out enough to allow the ingress of CO_2. Likewise, if the concrete is dry enough to avoid the sufficient amount of moisture in the pores, the chemical reaction could be not presented.

Therefore, the carbonation achieves the maximum level for RH values between 50 and 70%. If the RH is above 70%, carbonation slows down caused by the slower diffusion rate of CO_2 through the saturated pores. At RH values below 50%, there is insufficient moisture in the pores to enable carbonation reactions to take place in the structure (Russell et al., 2001). Then, the diffusion of carbon dioxide considering the humidity can be expressed as:

$$D \propto (1 - RH)^m \qquad \textit{for} \qquad RH > 50\% \tag{4.3}$$

Where *D* is the effective diffusivity of the carbon dioxide into the concrete, *RH* is the relative humidity expressed as a fraction, and *m* is a humidity constant.

4.3.1.4 Diffusion of $Ca(OH)_2$ and CO_2

For the formulation of the diffusion considering the effects of the variables that interfere in the carbonation reaction, the model considers a set of parameters that provide the model with higher reliability for the prediction of the service life. These parameters are the temperature effect on Henry's constant, the dissolution of carbon dioxide in the water of the pores, the temperature effect on the solubility of $Ca(OH)_2$, the concentration of $Ca(OH)_2$ in the water of the pores and the reaction rate coefficient.

Once the carbon dioxide ingress in the concrete, it dissolves in the water forming an aqueous CO_2 solution whose concentration is governed by Henry's Law according to the expression:

$$CO_{2(aq)} = HR_2TCO_{2(g)} \tag{4.4}$$

Where H is the Henry's constant ($mol/m^3.atm$), R_2 is the gas constant ($m^3.atm/K.mol$), T is the temperature (K), and $CO_{2(g)}$ is the atmospheric concentration of carbon dioxide (mol/m^3). In turn, the temperature has an effect within this Henry's constant that must be considered in the formulation of the diffusion. That is, the Henry's constant may change with the temperature. Thus, this effect may be determined similar to the temperature effects in the diffusion coefficient by means of the expression:

$$H(T) = H_{ref} e^{\left[\Delta\left(\frac{1}{T_{ref}} - \frac{1}{T}\right)\right]} \tag{4.5}$$

Where H_{ref} is the reference Henry's constant (mol/m^3.atm), and Δ is the enthalpy constant (K). There is also a maximum limit regarding how much $Ca(OH)_2$ can react considering the effect of temperature. In the model is proposed the following equation to takes into account such condition:

$$K_{sp} = (0.0125\ x\ 10^9)e^{-0.019T} \tag{4.6}$$

Where K_{sp} represents the solubility product of calcium hydroxide (mmol^3/L^3) for the temperature of interest *T*. In this way, considering the basic solubility equilibrium formula, the aqueous concentration of $Ca(OH)_2$ in the pores of the cement can be determined through the following expression:

$$[Ca(OH)_{2(aq)}] = \left(\frac{K_{sp}}{4}\right)^{\frac{1}{3}} \tag{4.7}$$

The chemical reaction between the carbon dioxide determined in Eq. (4.4) and the calcium hydroxide obtained from the above equation is a process that occurs through a temperature-dependent process. To consider this effect, it is essential to consider in the model a reaction rate between both compounds. This reaction rate is important since the product of this reaction is the calcium carbonate that breaks down the alkalinity of the cement paste. Determine the speed of this reaction allows knowing the rate at which depassivation occurs in the concrete. Therefore, this rate constant may be expressed according to a second order relationship as follow:

$$k_c = \beta_r e^{\left(\frac{-U}{R_2 T}\right)} \tag{4.8}$$

Where k_c is the reaction rate constant between the CO_2 and the $Ca(OH)_2$ for a temperature of interest (m^3/mol/s), *U* is the reaction activation energy (J/mol K), β_r is the pre-exponential factor (m^3/mol/s), *R₂* is the gas constant (m^3.atm/K.mol), and *T* is the temperature (K).

4.3.1.5 Formulation of the model for corrosion initiation: Concentration of CO_2 and $Ca(OH)_2$

Finally, the relationships of the parameters and variables formulated above are combined in a differential equation with which it is possible to obtain the general molar balance for carbon dioxide in the aqueous pores of the concrete for a given location and time. In this way, if the diffusion is independent of the position, in the numerical model the following equation is proposed to determine the aqueous CO_2 concentration over time:

$$\frac{\delta}{\delta t}[CO_{2(aq)}] = D\frac{\delta^2}{\delta x^2}[CO_{2(g)}]HR_2T - k_c[CO_{2(aq)}][Ca(OH)_{2(aq)}] \tag{4.9}$$

Considering the domain:

$$CO_{2(g)}(x,t) \qquad 0 \le x \le L \ ; \ 0 \le t < \infty$$

Under the boundary conditions:

$$CO_{2(g)}(x,0) = 0 \qquad \text{For } x > 0$$

$$CO_{2(g)}(0,t) = CO_{2(atm)}(t) \qquad \text{For } t > 0$$

$$\frac{d}{dx}CO_{2(g)}(L,t) = 0 \qquad \text{Zero-flux boundary}$$

Among the previous conditions, it is important to note that the second conditions are time-dependent. This allows knowing the concentration of carbon dioxide for each instant of time given at a specific location within the concrete element. In the same way as the previous equation, it is possible to determine the molar balance for the general aqueous concentration of calcium hydroxide in the pores of the concrete. Determining this concentration is important for the carbonation model since if there is no $Ca(OH)_2$, the carbon dioxide does not react to form the calcium carbonate. The proposed equation to obtain such concentration for a given time and position is the following:

$$\frac{\delta}{\delta t}\left[Ca(OH)_{2(aq)}\right] = D\frac{\delta^2}{\delta x^2}\left[Ca(OH)_{2(aq)}\right] - k\left[CO_{2(aq)}\right]\left[Ca(OH)_{2(aq)}\right] \qquad (4.10)$$

Considering the domain:

$$Ca(OH)_{2(aq)}(x,t) \qquad 0 \le x \le L \; ; \; 0 \le t < \infty$$

Under the boundary conditions:

$Ca(OH)_{2(aq)}(x,0) = Ca(OH)_{2(aq)i}$ For $x > 0$

$\frac{d}{dx}Ca(OH)_{2(aq)}(0,t) = 0$ Zero-flux boundary

$\frac{d}{dx}Ca(OH)_{2(aq)}(L,t) = 0$ Zero-flux boundary

In this way, by solving Equations (4.9) and (4.10) simultaneously, it is possible to obtain the concentrations of both solutions involved in the carbonation reaction using the "Method of Lines" proposed by Cutlip and Shacham (Cutlip and Shacham, 2007). For concrete elements, thermal diffusion is much higher than mass diffusion, which is why the temperature of the concrete is assumed uniform over time, and it is unnecessary to consider the heat generated by the reaction (Talukdar, 2013).

It is recognised that carbon dioxide, in addition to reacting with $Ca(OH)_2$, also reacts with the Calcium–Silicate–Hydrate (C-S-H) (Peter et al., 2008; Borges et al., 2010). However, some studies have shown that only half of CO_2 reacts with $Ca(OH)_2$ and that the other half reacts with C-S-H, happening this process consecutively (Glasser and Matschei, 2007). So, carbon dioxide reacts with C-S-H once it has reacted firstly with the $Ca(OH)_2$ and the calcium hydroxide is entirely consumed. Furthermore, considering that the approach of the degradation model considers the carbonation in non-pozzolanic cement, the reaction is considered only between the carbon dioxide and the calcium hydroxide of the concrete.

4.3.2 Corrosion Propagation Stage

For this degradation stage, the proposed model employs an analytical thick-walled uniform cylinder which assumes that the generation of a volume of corrosion products (rust) around the corroding steel causes an expansion in the diameter of the steel. This expansion exerts a uniform pressure in the concrete that surrounds the rebar, which when the limit stresses is overtaken, it causes the cover cracking that accelerates the corrosion process. Therefore, the corrosion products formed are expansive in a proportion between 2 to 10 times greater than the original state, precipitating at the reinforcing-concrete interface. Then, if a pressure of swelling is produced in the concrete with values of 3-4 MPa, the concrete breaks down due to the tension forming the first cracks. As is depicted in Figure 4.5, these cracks due to excess corrosion are formed from the surface of the rebar to the nearest external surface of the concrete (Yeomans, 2004).

According to Raupach, there are at least two reasons why it is important to consider the propagation stage of corrosion in a degradation model. On the one hand, for structures located in aggressive environments, the propagation stage could last several years or even decades. On the other hand, there is a need to be able to estimate the propagation stage for existing structures during the design phase of the project. This allows knowing the progress of the corrosion and being able to evaluate the actions of repair or protection in the structure (Raupach, 2006).

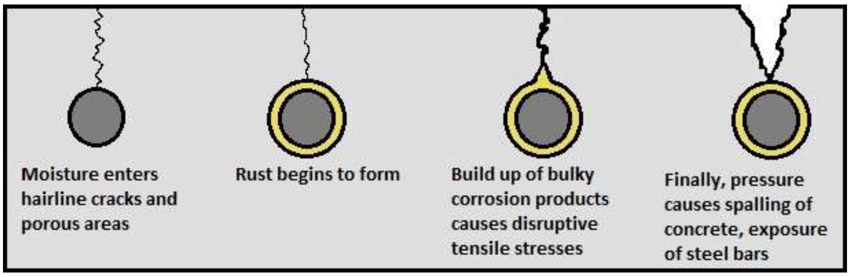


Figure 4.5 Progress of corrosion-induced damage in concrete (Yeomans, 2004).

Within the formulation of this stage of corrosion degradation, the model considers parameters such as the activation energy and the reference current flow density which are not generally taken as constants by other models. These parameters may depend on the interaction between the concrete resistivity, the saturation level and the cover thickness. As in the first stage of degradation, for the propagation stage the author of the model proposed a flowchart which facilitates the understanding and application of the model formulated and is represented in the following figure:

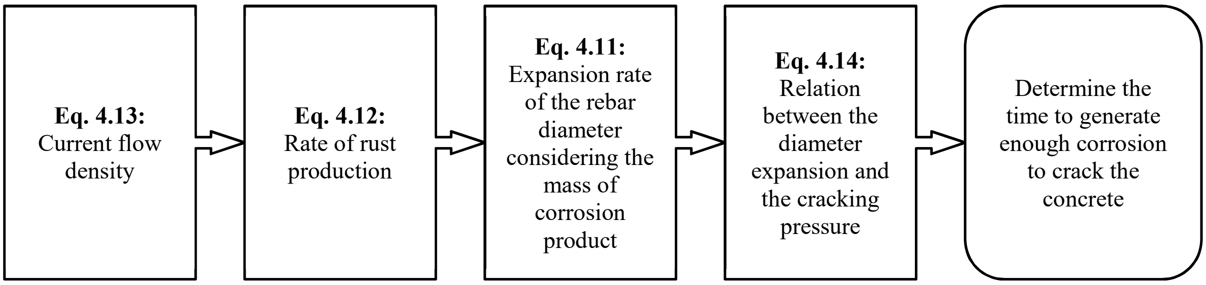


Figure 4.6 Modelling of propagation stage of corrosion (Talukdar, 2013).

As can be seen from Figure 4.6, basically this second stage of the model is composed of four stages. These stages aim to determine the time for enough formation of oxide in the concrete element that causes its cracking. Then, considering the established by Tuutti, with this last phase can determine the end of the period of the service life of the structure. The expressions of the propagation stage model match the El Maaddawy and Soudki's corrosion model where the Arrhenius Theory and Faradays Law are considered. So, the model gives the time taken for a rust formation equal to 10μm to generate a pressure which cracks the cover concrete (El Maaddawy and Soudki, 2007).

4.3.2.1 Expansion rate of the rebar diameter

As already mentioned, due to the effect of corrosion on the reinforcement, an expansion of its diameter is verified during the degradation process. This increase in the cross-section is represented according to Equation (4.11), with which a formulation of the scalar quantity of this value is obtained.

$$\Delta d = \frac{2W_{rust}}{\pi d}\left(\frac{1}{\rho_{rust}} - \frac{\gamma_{sr}}{\rho_{steel}}\right) - 2\delta \qquad (4.11)$$

Where d represents the initial diameter of the rebar (mm), ρ_{rust} is the density of the rust (g/mm^3), ρ_{steel} is the density of the steel reinforcement (g/mm^3), δ is the thickness of the porous zone (μm), γ_{sr} is the relation between the steel mass and the rust, and W_{rust} is the mass of the corrosion product per unit length of the reinforcement (g/mm). For the formulation of Eq. (4.11) is assumed that the expansion generated by a larger volume of corrosion products compared to that of the lost steel is modelled by a uniform increase in the diameter of the cylindrical hole around the reinforcing bar. Moreover, deformations of the corrosion products and the remaining steel are neglected (Chernin and Val, 2011).

4.3.2.2 Rate of rust production

The value of W_{rust} may be determined as a function of time by relating it to the corrosion current density and other parameters according to the following expression using Faraday's law of electrolysis:

$$\frac{dW_{rust}}{dt} = \frac{I(t)M}{zF} \tag{4.12}$$

Where t is the time of the corrosion process (s), $I(t)$ is the corrosion current density regarding time (A/mm^2), M is the molar mass of the steel rebar (g/mol), z is the valence of the reaction, and F is the Faraday's constant (C/mol). Then, if the equation is integrated with respect to time, it is obtained the rate of rust production that may be considered for the Eq. (4.11).

4.3.2.3 Corrosion current density ($I(t)$)

For the above equation, it was necessary to calculate the corrosion product by means of the relationship with the corrosion current density. However, this density needs to be calculated previously from a mathematical expression that relates it to the effect of temperature during the process, as it is presented in the following expression:

$$I(t) = I_o^{-\alpha_{ae}\left(\frac{1}{T(t)}-\frac{1}{T_0}\right)} \tag{4.13}$$

Where I_o is the current flow density at the reference temperature (A/mm^2), α_{ae} is the activation energy constant (K), $T(t)$ is the temperature for a given time (K), T_0 is the reference temperature.

4.3.2.4 Relation between the expansion and the cracking pressure

Once the previous equations are solved, the effect produced by the expansion caused by the rust accumulation in the cracking pressure generated must be related. It is assumed that resulting cracking occurs after a sufficiently long time since corrosion initiation. Therefore, Eq. (4.11) must be related to the following expression:

$$\Delta d = \frac{d}{E_{eff}}\left(1 + \vartheta - \frac{d^2}{4b(b+d)}\right)\frac{2bf_{ct}}{d} \tag{4.14}$$

Where E_{eff} is the effective modulus of elasticity of concrete (MPa) that, for this case, will be established as $E_{eff} = E_c/3$, E_c is the modulus of elasticity of the concrete at 28 days (MPa), ϑ is the Poisson's ratio of concrete, d is the rebar diameter (mm), b is the cover thickness of the concrete (mm), and f_{ct} is the yield stress of the concrete (MPa). To the formulation of the last equation, it was assumed that the concrete cover is completely cracked when the average tensile stress in it becomes equal to the tensile strength of concrete. This is equivalent to assuming a perfect plastic behaviour of the concrete from the instant when the maximum tensile stress in the concrete cover is achieved until the full cracking of the cover (Chernin and Val, 2011).

Now, as already mentioned above, the mathematical model proposed by Talukdar has a set of variables whose values in some cases are previously established, and in other cases, it is necessary to set them as an input value according to the constructive and climatic context of the study. Table 4.3 shows the values of these variables that are applied to obtain the carbonation depth curves in the structures. The variables that are not shown in the table are due to the fact that they are input values of the carbonation model that must be indicated by the user, such as the cement content, the water content, the relative humidity, among others.

Table 4.3 Values of the variables applied in the carbonation model

Variable	Value	Unit	Source
$\boldsymbol{A_D}$	1.64×10^{-6}	*m²/s*	(Papadakis et al., 1991b)
$\boldsymbol{\alpha_D}$	1.8	–	(Papadakis et al., 1991b)
$\boldsymbol{\rho_c}$	3120	*Kg/m³*	(Talukdar et al., 2012a)
$\boldsymbol{\rho_w}$	1000	*Kg/m³*	(Talukdar et al., 2012a)
$\boldsymbol{Q}$	39000	*J/mol K*	(Talukdar et al., 2012a)
$\boldsymbol{R_1}$	8.314	*J/mol K*	(Talukdar et al., 2012a)
$\boldsymbol{T_{ref}}$	298	*K*	(Talukdar et al., 2012a)
$\boldsymbol{m}$	2.2	–	(Papadakis et al., 1991b)
$\boldsymbol{H_{ref}}$	34.2	*mol/m³ atm*	(Fogg and Sangster, 2003)
Δ	2400	*K*	(Fogg and Sangster, 2003)
$\boldsymbol{R_2}$	8.2×10^{-5}	*m³ atm/K mol*	(Saetta and Schrefler, 1993)
$\boldsymbol{U}$	40000	*J/mol K*	(Khunthongkeaw and Tangtermsirikul, 2005)
$\boldsymbol{\beta_r}$	1390	*m³/mol/s*	(Khunthongkeaw and Tangtermsirikul, 2005)
$\boldsymbol{\rho_{rust}}$	3.93	*g/mm³*	(Chernin and Val, 2011)
$\boldsymbol{\rho_{steel}}$	7.85	*g/mm³*	(Chernin and Val, 2011)
$\boldsymbol{\delta}$	10	*µm*	(El Maaddawy and Soudki, 2007)
$\boldsymbol{\gamma}$	0.622	–	(Chernin and Val, 2011)
$\boldsymbol{\vartheta}$	0.18	–	(Chernin and Val, 2011)
$\boldsymbol{M}$	55.85	*g/mol*	(El Maaddawy and Soudki, 2007)
$\boldsymbol{z}$	2	–	(El Maaddawy and Soudki, 2007)
$\boldsymbol{F}$	96458	*C/mol*	(El Maaddawy and Soudki, 2007)
$\boldsymbol{I_o}$	0.1	*A/mm²*	(Talukdar and Banthia, 2013)
$\boldsymbol{T_0}$	293.15	*K*	(Talukdar and Banthia, 2013)
$\boldsymbol{\alpha_{ae}}$	7500	*K*	(Talukdar and Banthia, 2013)

These values must be considered within the numerical model for resolution. Nonetheless, two of these values were recently modified in another investigation carried out by the author of the carbonation model. For equation 4.1, it has been determined through a more rigorous examination that the empirical values of A_D and α_D should be calibrated with respect to the original values proposed by Papadakis *et al*. It was then determined that for values of A_D equal to 1.65×10^{-5} m^2/s and α_D equal to 0.8, the numerical model could predict the experimental results with reasonable accuracy (Talukdar and Banthia, 2015). Therefore, for the application of the model in this research, these latter values have been adopted instead of the values of A_D and α_D shown in Table 4.3.

4.4 Degradation curves for Structures in Paraguay

In this section, the results obtained after applying the numerical carbonation model described above are shown. These results include the advance curves of the carbonation depth along the service life of RC structures considering two different climatic scenarios of the IPCC. Furthermore, a control curve is developed to be able to monitor the expected increase in the degradation rate caused by the climate change effect. On the other hand, it is also possible to determine through the degradation model what are the times of corrosion initiation caused by carbonation, as well as the time to the cover cracking caused by corrosion. These critical times are presented and analysed in this section considering factors such as the cover thickness, the concrete strength and the climatic scenario foreseen about climate change. Finally, the accuracy of this model is verified through a correlation analysis between the results obtained by the simulated curve and the real carbonation data presented in the previous chapter in Section 3.7.1.

4.4.1 Parameters considered for the simulation

The numerical model chosen to perform the degradation curves for structures in Paraguay is governed by both climatic parameters and parameters of the construction system, i.e., cement quantity, cement type, water/cement ratio, etc. Therefore, it is important first to define the variables associated with the three climatic parameters that directly influence the carbonation advance in the structures, namely temperature, relative humidity and the atmospheric concentration of carbon dioxide. It has been found that one of the environmental parameters that mainly affect the carbonation rate is the CO_2 concentration in the air (Saetta and Vitaliani, 2004, 2005). One of the major concerns regarding the carbonation-induced degradation comes from the dramatic increase of CO_2 volume in the environment, which has reached its maximum historical peaks by the middle of 2018.

Considering the incidence of environmental conditions in the degradation by carbonation and also in the numerical model adopted, it is relevant to define the climatic conditions to which the structures in the country are exposed. Paraguay is geographically located in South America and is crossed by the Tropic of Capricorn. Annual average temperatures are 24 °C with great thermal variations determined by its landlocked condition (i.e., a country without sea cost) and its topography practically flat. The country has springs and winters with pleasant temperatures, frequently, without frost, with average temperatures of 19 °C. Summers are scorching with a high percentage of humidity, and in some regions, including Asunción, the temperature may exceed 41 °C. Regarding precipitation, they are more abundant in the South-East region, where average values of 1800 mm are registered, as well as towards the Northwest region where 700 mm are reached, almost all of the precipitations occur in Summer season (Portillo et al., 2001).

Among the different climate classification systems, the most widely used is the empirical scheme called Köppen-Geiger (Krolak, 2001; Peel et al., 2007). According to Köppen-Geiger classification, Paraguay is divided into five different climatic zones, as depicted in Figure 4.7, where two of them are predominant: The Tropical savanna climate (Aw) and the Warm oceanic climate/Humid subtropical climate (Cfa). As per the characteristics for the Cfa climate group, the average temperature of the coldest month of the year is higher than -3 °C but below than 18 °C, the summers are hot and the winters mild. In the regions with this type of climate, the air masses are unstable and cause rainfall throughout the year. Thunderstorms in summer and frontal precipitation in winter are frequent, the mean monthly temperature in summer is around 27 °C, and in winter it varies from 5 °C to 12 °C (NCERT, 2006).

For obtaining the degradation curves considering the climate change effect for structures in Asunción, a data set from the CORDEX Project for each scenario was obtained. CORDEX is a project created with the objective of developing a coordinated framework for evaluating and improving Regional Climate Downscaling (RCD) techniques and producing a new generation of RCD-based fine-scale climate projections for identified regions worldwide (Giorgi et al., 2009). Moreover, the CORDEX Project contributes to the IPCC Fifth Assessment Report (AR5) and the climate community beyond the AR5.

The climate model called Hadley Global Environment Model 2 - Earth System (HadGEM2-ES) was chosen for the generation of raw data for monthly temperatures and CO_2 concentrations databases for the period 2017-2077 corresponding to the city of Asunción. To consider the effect of climate change, two climate scenarios proposed by the last IPCC report were adopted. For the best case, the RCP 4.5 climatic scenario was chosen. Whereas for a worse case, the RCP 8.5 climatic scenario was used to obtain the projections of the climatic parameters (i.e. temperature and carbon dioxide concentration). The database generated by HadGEM2-ES was linked into the carbonation model addressed in this chapter in order to obtain the degradation curves that shows the carbonation depth over a lifespan of 60 years in concrete structures in the city.

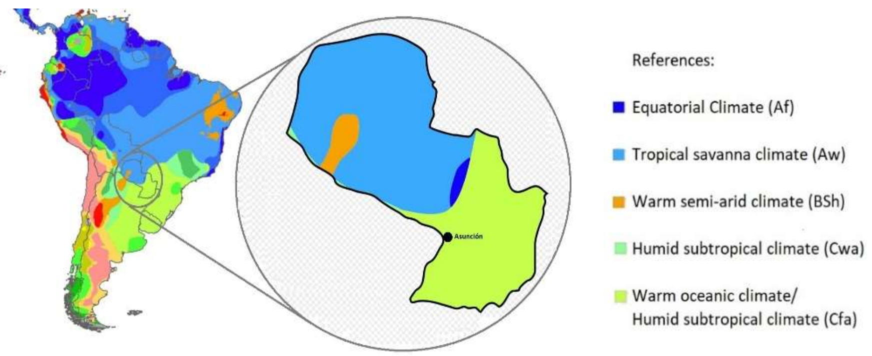


Figure 4.7 Köppen-Geiger Climate classification map of Paraguay (Peel et al., 2007).

On the other hand, there has always been a great debate among those researchers who assume that relative humidity will remain constant due to global warming increase and those who have demonstrated a (slight) increase in this parameter. Nevertheless, a large amount of IPCC's climate change studies are based on climate models that assume constant the relative humidity. Because of this, the carbonation model applied in this research assumes that relative humidity will remain constant over the next decades and, therefore, the value of current relative humidity in Asunción has been considered for the simulations. Therefore, for the control scenario, the relative humidity and mean annual temperature were held to the values 75.1% and 23.7 °C respectively (DGEEC, 2015). Unfortunately, the CO_2 concentrations are not measured in the country, and the value of this parameter is considered in this research according to the mean value of the concentration regarding the last 15 years registered globally by the international recordings (NOAA-ESRL, 2018). In other words, the control scenario considers that the climate change does not take place for the simulation and the climatic parameters do not present alteration over time.

Another aspect that must be considered is the role of cracks in the corrosion of reinforcements that is a controversial issue. On the one hand, cracks reduce the service life of structures because they allow CO_2 to diffuse more quickly into concrete. For this reason, the carbonation depth of concrete tends to increase with the increase in the w/c ratio and the crack width (Wang, 2018). On the other hand, although the cracks accelerate the onset of corrosion, it is merely localised. Therefore, as was exposed in the ACI reports for the control of cracking (cracks before corrosion onset) in concrete structures: "*after a few years of service, there is a little difference between the amount of corrosion in cracked and uncracked concrete. More important parameters for corrosion protection are concrete cover and concrete quality*" (ACI, 2002). Therefore, the carbonation model that has been chosen, which does not consider the cracking in the concrete, satisfies the requirements to develop the analysis of the degradation by carbonation through the simulations of such model.

The results obtained by the carbonation model will be contrasted with the results of carbonation tests carried out in a set of real structures located in the city of Asunción. These in-situ tests have shown wide variability in the compressive strength values of those structures (18 to 46 MPa). For this reason, simulations for RC structures with representative strengths of 20, 25, 30 and 45 MPa have been considered. Through this consideration, it is possible to perform a more significant analysis of the results obtained with the model regarding the real degradation of structures in Asunción.

Furthermore, similarly to the parameter of concrete strength, the entrained air content is considered into the model for the generation of carbonation curves. It was challenging to access a database with this sort of measurement in real structures in Paraguay. However, the climate in Paraguay is not extremely cold, and structures are not exposed under a freeze-thaw cycle. Indeed, temperature measurements do not register values below 0 °C or snowfall during the whole year. Because of this, the use of entrained air in concrete is not frequent and hence, for the simulations, the value of this parameter has been considered equal to 0%.

4.4.2 Results of the numerical simulation

The carbonation model provides a valuable perspective on how the carbonation front advances over time through concrete structures under the influence of climate change. This section shows this behaviour with the expected degradation in RC structures of Paraguay for the climatic scenarios RCP 4.5 (best case) and RCP 8.5 (worst case). The results depicted in Figure 4.8 show the carbonation depth for RC structures with the characteristic compressive strength of 20, 25, 30 and 45 MPa. These curves were generated through simulations that were carried out with the carbonation model using the software MATLAB, version R2015a. These simulations exclusively include the environmental conditions to which structures located in the urban area of the city of Asunción are exposed.

Moreover, the parameters have been chosen to represent as accurately as possible the constructive characteristics of these structures. The differential equations of the carbonation model were numerically solved using nodes spaced 5 mm in order to obtain the concentration of gases for each node over time, i.e. the concentration of gases at each point in the concrete structure. The gap between nodes has been set considering that carbonation-induced corrosion begins once the carbonation front is at least within 5 mm of embedded rebar (Yoon et al., 2007). Figure 4.8 shows the carbonation depths obtained from the numerical model.

From the curves obtained with the model, some conclusions can be drawn regarding the corrosion risk by carbonation in the concrete structures of Paraguay. Concrete structures in Asunción would reach an ultimate carbonation depth (UCD) between 15 and 40 mm corresponding to a service life of 50 years (T_{SL}), depending on the concrete quality and the climatic scenario predicted. Herein, the ultimate carbonation depth is considered as the maximum depth of carbonation front reached in the simulation. The ultimate carbonation depth is subjected to the boundary conditions of the model. From the results of Figure 4.8, it should be noted that despite the expected service life considered herein is 50 years, some cases showed that the UCD has been reached after the 50 years for both the control and worst scenarios. So, to visualise the entire carbonation curve, the period considered for simulation was 60 years.

Analysing the results depicted in Figure 4.8, it can be seen that the carbonation curves present different profiles depending on each scenario. For example, for the RCP 4.5 scenario, the carbonation curve reaches its UCD before the 50 years. This behaviour is because the curve profile is governed by the chemical reaction between the carbon dioxide and the $Ca(OH)_2$. The reaction is produced until both components of the process are entirely consumed, forming calcium carbonate. When the diffusion of gas finds a boundary, its future evolution is affected by the condition of that boundary.

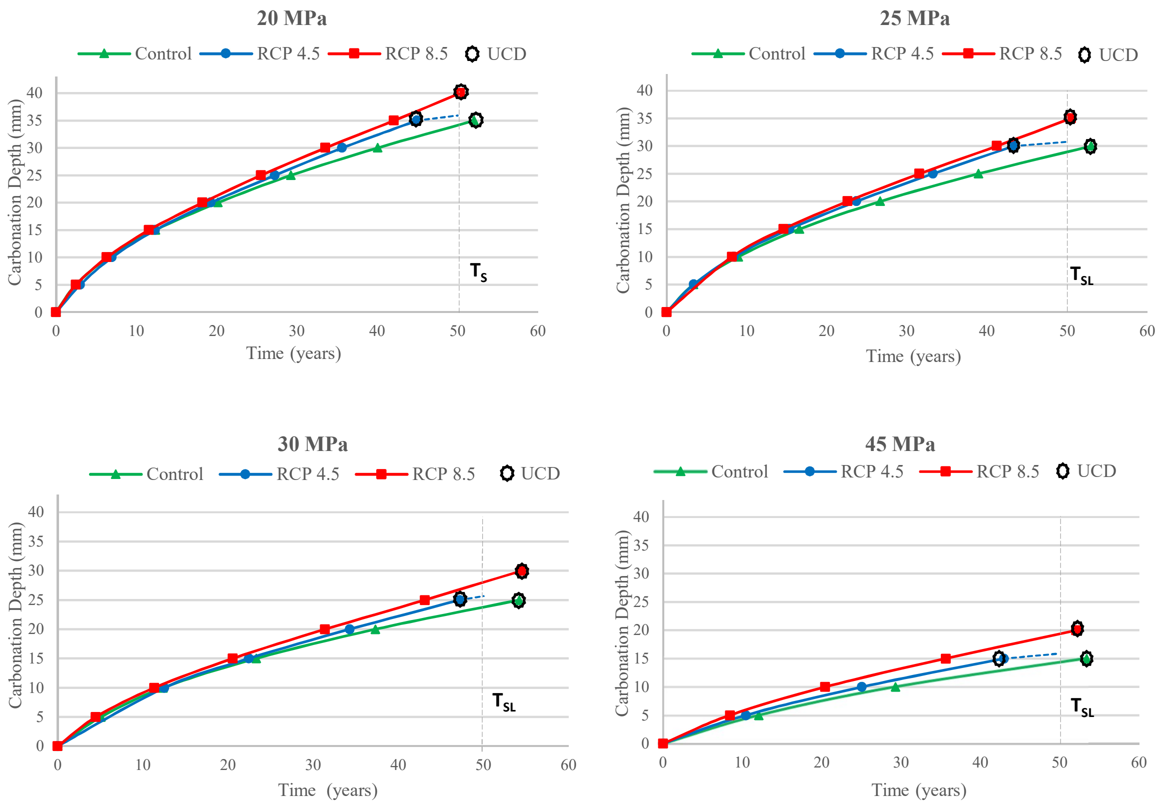


Figure 4.8 Expected carbonation depths for RC structures in Asunción until 2077

The carbonation model applied in this chapter employs a zero-flux boundary condition, which delimits the point for which the advective and diffusive flows are balanced exactly (Crank, 1975). The zero-flux boundary condition is commonly applied in systems of one-dimensional equations such as the carbonation model herein applied. This boundary condition determines that the concentration will no longer change concerning the position in which the analysed node is located. That is, the concentration could change over time, but not with the position.

Another issue that influences the curve profile in Figure 4.8 is the projections regarding the CO_2. It is well-known that carbon dioxide is a very influential parameter in carbonation, being one of the main components of the chemical reaction of this phenomenon. Regarding the climate scenarios considered in this chapter, Figure 4.9 depicts that the CO_2 emissions for RCP 8.5 scenario have a steady upward projection; while for the RCP 4.5 scenario, the projection rises until a peak emission which is visualised between 2040-2050 and then drops down sharply. Furthermore, the carbonation rate decreases as the concrete become more carbonated due to the reduction of porosity (Van Gerven et al., 2007).

In consequence, what is happening with the carbonation curve for the scenario RCP 4.5 is that a further 5 mm of carbonation is not expected in the next few years before the service life ends. Therefore, it was performed the simulation again for this scenario for a longer length of time to determine the time to reach the next node. Afterwards, through a linear extrapolation was found the carbonation depth until reach the service life of 50 years (blue dotted line in Figure 4.8), was not greater than 0.8 mm regarding the position of the last node reached in the first simulation.

These are estimated results based on some uncertainty, however, considering previous papers developed about the accuracy of the numerical model applied in this research (Talukdar et al., 2012a; Talukdar et al., 2012b; Talukdar and Banthia, 2015), it can be said that the RC structures in Paraguay would be under considerable risk of corrosion considering the forecasting climate changes. It has been found that the simulation has not presented a difference concerning the UCD between the control scenario and the best scenario. However, the difference was slightly relative to the temporal aspect, where the UCD is reached in the best climate scenario between 12% and 20% earlier than in the control scenario. On the other hand, the carbonation depth could increase by 25% for the worst-case scenario.

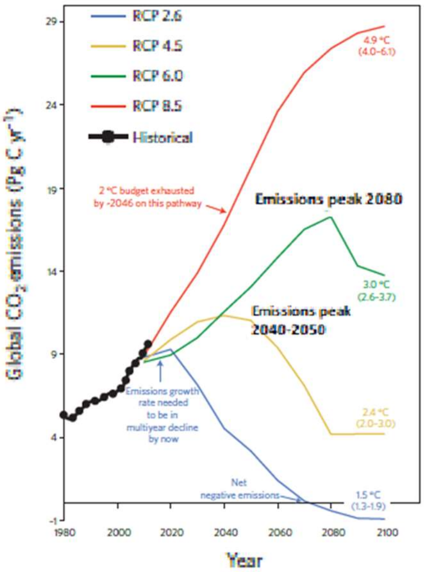


Figure 4.9 Projections trends for CO_2 emissions and temperature (Sanford et al., 2014)

As might be expected, the structures most vulnerable regarding carbonation-induced degradation are those whose strength is between 20 and 30 MPa. The structures with 45 MPa present acceptable and expected values regarding durability. In fact, a survey carried out on concrete structures in several cities in Paraguay in 2008 showed that the typical strength values for structures are between 18 MPa and 30 MPa. The referred study covered a total of 72 structures of the main cities of the country, of which 48 are located in Asunción. Furthermore, it was possible to determine that the use of concrete produced at a construction site was registered in 79% of cases, and only 21% of analysed cases used premixed concrete (Gonzalez et al., 2008). This construction practice is often substandard and leads to an earlier onset of carbonation-induced corrosion shrinking the service life of structures.

In addition, the results of the simulation for a control scenario where the effect of climate change is not considered is shown in the curves. These results are useful as a reference to appreciate the significance of the carbonation depths obtained from the other two scenarios. With the results of the degradation curves, it was possible to perform an analysis to determine the service life period of concrete structures in Paraguay. This analysis was performed considering the characteristic strengths of the structures and the concrete cover thickness. In this context, the time for corrosion initiation and cracking due to carbonation-induced corrosion for different kinds of structures in Asunción are given in Table 4.4. It has been considered that the time to corrosion initiation takes place when the carbonation depth is at 5 mm for a cover of 10 mm, and when the carbonation depth is at 20 mm for a cover of 25 mm. For the time to cracking, the propagation time formulated by the model has been computed.

The results depicted in Table 4.4 show a remarkable difference in deterioration times obtained with the simulation for both cover thickness values considered for the same structure. This behaviour coincides with what is exposed in (ACI, 2002), where is denoted the importance of the concrete quality and the cover thickness for the protection against corrosion, and thereby to prolong the service life of structures. A significant difference between the applications of concrete with 10 mm of cover versus concrete with 25 mm of cover can be noted. The importance of this analysis is later addressed in this chapter, where the reason why the study was made considering those cover thickness values is argued.

Depending on the cover thickness, the analysis performed in Table 4.4 also shows a different sequence for both the time to initiation and time for crack. It can be seen that for a cover of 25 mm, both times follow a marked sequence that is the highest time for the control scenario, an intermediate time for the best scenario and the lowest time for the worst scenario, which is repeated for each characteristic strength considered. Nevertheless, for a cover thickness of 10 mm, these times are not described by fixed sequence. This difference is because of the three curves practically coincide with each other during the first ten years of lifespan. Consequently, is only after 10 or 20 years that a definite difference in the carbonation depth among the curves is noted. So, for a cover of 10 mm, carbonation-induced corrosion might begin when the carbonation front had reached 5 mm of depth (< 10years). Instead, for a cover of 25 mm, it would be when the carbonation depth is at 20 mm (> 10years), being this behaviour the reason for that difference.

Table 4.4 Expected Service life for RC structures in Asunción.

Scenario	Time to initiation (years)	Time to cracking (years)	Propagation time (years)
Strength: **20 MPa** – Cover thickness: **10 mm**			
Control	2.44	4.27	1.83
RCP 4.5	3.00	4.56	1.56
RCP 8.5	2.47	4.37	1.9
Strength: **20 MPa** – Cover thickness: **25 mm**			
Control	20.15	22.47	2.32
RCP 4.5	19.27	22.22	2.95
RCP 8.5	18.24	21.12	2.88
Strength: **25 MPa** – Cover thickness: **10 mm**			
Control	3.35	5.21	1.86
RCP 4.5	3.38	5.34	1.96
RCP 8.5	4.08	5.57	1.49
Strength: **25 MPa** – Cover thickness: **25 mm**			
Control	26.65	29.31	2.66
RCP 4.5	23.69	26.38	2.69
RCP 8.5	22.57	25.33	2.76
Strength: **30 MPa** – Cover thickness: **10 mm**			
Control	5.07	6.36	1.29
RCP 4.5	6.27	8.24	1.97
RCP 8.5	4.48	6.36	1.88
Strength: **30 MPa** – Cover thickness: **25 mm**			
Control	37.31	40.18	2.87
RCP 4.5	34.29	37.13	2.84
RCP 8.5	31.41	34.2	2.79

Furthermore, this research has been noticed how the propagation time is influenced by the limit of generated rust considered in the simulation that results in a pressure which cracks the cover concrete. The numerical model considers that before the expansive pressure of the oxide is generated, it is necessary that the porous zone of the concrete/steel interface be wholly filled with the corrosion products. This porous zone has been initially considered in the numerical model with a value of 10 µm. However, many other studies consider different values for this parameter ranging from 50 µm to 100 µm (Siemes et al., 1985; Broomfield, 2007).

In a sensitivity analysis, it was found that an increment of 50% of the limit parameter considered to obtain the results of Table 4.4 can increase the propagation time in a range of 10-60%. Instead, if a limit of 53.6 µm is considered as in (Köliö et al., 2015), so the propagation time will be on average three times greater than the results shown in Table 4.4. This limit of rust generation is a controversial topic that is influenced by the porous zone between the concrete and rebar. In turn, this porous zone is caused by the transition from cement paste to steel, entrapped/entrained air voids, and capillary voids in the cement paste into which corrosion products diffuse (El Maaddawy and Soudki, 2007). Therefore, concrete quality is always a critical factor regarding durability in RC concrete infrastructures.

4.4.3 Correlation Analysis of the results

The results obtained from the simulations carried out with the model described in the previous section have been compared with reports of *in-situ* tests carried out previously in Paraguay. These reports enable to attempt to verify the reliability of the model used by mean of real degradation data in concrete structures. It is also important to note that the real data showed alarming values for both the carbonation depth and the cover thickness detected in structural elements analysed. A detailed analysis of these real carbonation data in Asunción has already been addressed in Section 3.7.1 of the previous chapter. These same data are used here to contrast the results simulated by the model with the real degradation rate of the structures in Paraguay.

Most of the existing RC structures in Paraguay are made with Ordinary Portland Cement. Indeed, it was not usual to use pozzolanic concrete in constructions until the end of the last century, where there has been a sudden growth in infrastructures in the country, especially in the capital city of Asunción. For this reason, the carbonation model applied in this research was the most suitable to perform correlation analysis. Therefore, to determine if the test results are well represented by the carbonation model, a correlation analysis has been performed to verify analytically the correlation between real carbonation depth measurements and theoretical carbonation data given by the numerical model. Thus, to verify the reliability of the model, a quantitative statistical indicator was used in this study. This quantitative indicator is the coefficient of determination (R^2) that is the regression sum of squares divided by the total sum of squares of a data sample. For an accurate verification of the correlation between real data and the carbonation model, R^2 should approach 1.0 as closely as possible in the Equation 4.15. The usefulness of R^2 is its ability to find the likelihood of future events falling within the predicted outcomes.

$$R^2 = \frac{SSR}{SSTO} = \frac{\sum_{i=1}^{n}(\hat{y}_i - \bar{y})^2}{\sum_{i=1}^{n}(y_i - \bar{y})^2} \tag{4.15}$$

Where SSR is the regression sum of squares, $SSTO$ is the total sum of squares, $\hat{y}_i$ is the theoretical value (calculated by the numerical model), y_i is the measured value (real carbonation data), $\bar{y}$ is the mean of the sample, and n is the number of data in the sample.

The variation among the carbonation test results is extensive and it does not show a particular pattern. Contrasting the results of the carbonation model with the real carbonation data measured in existing structures, matches in some points can be seen, however, also some discrepancies at other points are shown. On the other hand, it can be said as a general perception that the structures tested had a very high carbonation depth considering their lifespan.

For the correlation analysis, structures with a characteristic strength of 20 MPa were considered because it was the one with the most significant number of data. In fact, the most frequent strength of concrete structures in the country is between 18 and 21 MPa (Gonzalez et al., 2008). This set of data corresponds to apartment buildings located in the urban area of the city with a height of 3 to 7 floors. Several of these departments have not completed their construction yet, that means, they are not inhabited. Therefore, Figure 4.10 shows a correlation analysis between the *in-situ* tests results for different structures and the carbonation curves for a 20 MPa concrete structure in Asunción. The effect of being sheltered or unsheltered on the carbonation progress into the *in-situ* tested element has been considered.

The coefficient of determination for each sort of structures exposure was calculated. It was found that for concrete elements sheltered from the weathering, the correlation between data has depicted an R^2 equal to 0.5953 for the control scenario. Conversely, for unsheltered concrete elements, this value was 0.4059 for the control scenario. For the other scenarios, R^2 did not show values below 0.498 and 0.250 respectively. Clearly, the model has a potential prediction that is better adapted for the case of structures sheltered from the weathering. For the analysis shown in Figure 4.10, the real carbonation data were contrasted with the carbonation depth curve generated by the model for the control scenario. This scenario, where the climate change effects are not considered, is suitable for the correlation analysis for structures built in the previous century (case study). Moreover, one of the main features of the analysis herein presented is that the same simulation can illustrate not only the advance of carbonation depths for a past time frame (control scenario) but also for future projections of this parameter (best and worst scenario).

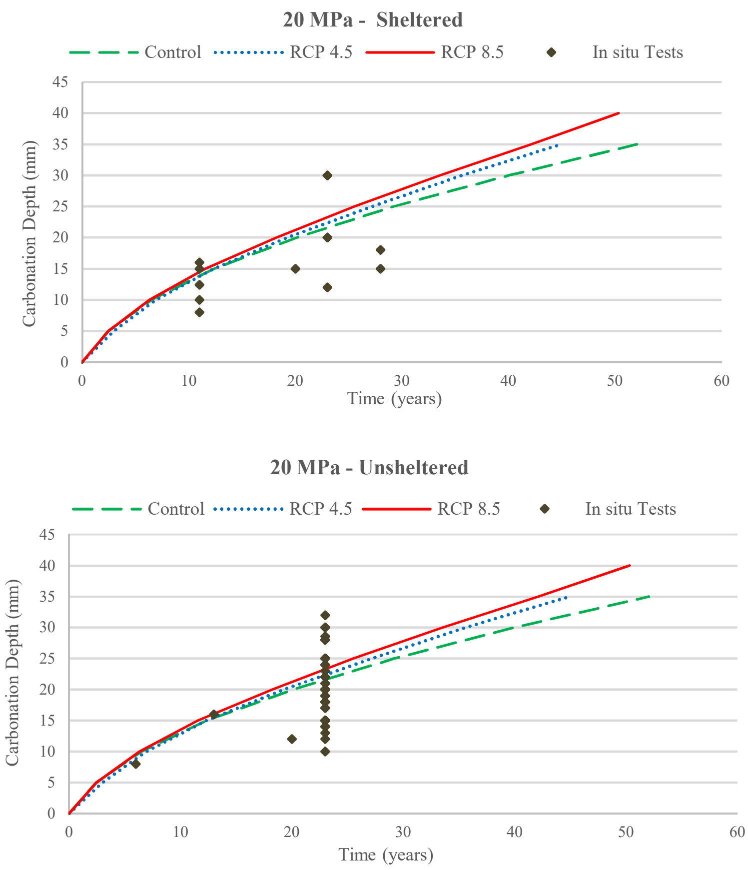


Figure 4.10 Correlation between real carbonation data and carbonation curves of the model.

The significance of R^2 depends mainly on the size of the sample. For instance, if a sample of 10 data is considered and the result of R^2 is equal to 0.99, the correlation may not be very significant due to the lack of data. Conversely, an R^2 equal to 0.6 in a sample of 1000 data may be more significant than the previous one, since the sample could be more representative. Hence, a high value of R^2 does not must necessarily be a strong correlation; it is merely a non-zero correlation. Therefore, it could be said that the analysis performed in Figure 4.10 provides results which have a better correlation for sheltered structures forehead to the unsheltered ones. Nonetheless, this correlation is not strong enough to achieve a determining conclusion.

The results of the numerical carbonation model presented in this investigation correspond to a simulation based on a period of 60 years. The data obtained from the *in-situ* tests carried out in Paraguay were performed in different structures with a lifespan of until 28 years. This difference in the temporal context does not allow an overall analysis of the correlation between the data of real cases and the curve generated with the simulation of the model. With this sort and amount of data, it is complicated to determine a clear correlation between both results. Nevertheless, some appreciations can be drawn after applying the quantitative statistical indicator (R^2) used in this investigation.

Through the statistical analysis developed and considering the uncertainty involved in the degradation mechanism studied, it can be said that the model presents a low correlation between the carbonation curve generated by the simulation and the real carbonation data measured in both sheltered and unsheltered structures. However, the work herein presented shows high carbonation depth values in existing structures of Asunción that are represented by the ultimate carbonation depth values obtained with the numerical model.

To performing the simulations in the model, the input parameters were established considering the constructive characteristics of the tested structures. However, these might not necessarily be the real parameters, and thus, a low correlation could be expected. Furthermore, specific parameters such as air content were estimated since there were no measurements for this variable. Therefore, it must be recognised that the model is not necessarily going to match with the data unless the specific properties to input are known and that be able to have data generated at regular intervals, in the same structure, over some years. For this reason, a suggestion for further research to perform this type of analysis would be done to have a structure that is exposed to service conditions, both environmental and use, which can be examined over the years. However, it is necessary to know all its constructive characteristics from the beginning, and it must regularly be monitored throughout its service life to generate a reliable database. These data must be carefully studied before being contrasted with numerical models, in order to have more conclusive results in a correlation analysis.

Although the numerical model selected to perform the degradation simulations was previously validated through laboratory tests in other research, this study presents a new approach where the degradation curves have been compared with real cases of carbonation measurement. Furthermore, the model was applied for a different geographical and climatic conditions from those the same model has been implemented in previous studies. It is important to emphasise the wide difference between the concrete elements manufactured in the laboratory of those corresponding to an existing structure. In Paraguay, this difference between both sorts of structures is even more significant due to the lack of control during the on-site execution of concrete structures. This condition could be the justification for the high carbonation depths in the structures of Paraguay.

The climatic conditions adopted for the simulation in the degradation model has generated degradation curves with higher ultimate carbonation depth compared to other curves obtained with the same model for other countries (Talukdar and Banthia, 2013). The results of Figure 4.8 show that the climatic parameter also has a high influence in the city of Asunción to obtain the carbonation depths. This parameter, associated with the poor quality of RC structures in Paraguay, reflects the vulnerability of the country's structures under carbonation-induced degradation, which presents a high risk of corrosion caused by this phenomenon.

Also, the values of air content and fissure rate with a fixed value for all structures were considered for the simulations. This enables to standardise the outcomes and to be able to analyse the variation among curves. However, it is necessary to recognise that for structures, which are not fabricated with a ready-mixed concrete, and even more in Paraguay, the concrete properties vary independently in each structure. If the control and the application of construction standards are established, then this variation of properties might be less significant, and the analysis of the degradation would be more reliable. Meanwhile, to treat this variation, it would be interesting to perform a more thorough statistical analysis, which could not be performed in this research due to the difficulty to access a more meaningful database. The study developed in this chapter has the influence of uncertainty corresponding to the parameter of the climatic conditions and the uncertainty that presumes the degradation of structures. However, the results obtained in this study represent an essential contribution to the field of durability and maintenance of concrete structures in the country since research in this field is underdeveloped in Paraguay.

Finally, a reduction in the service life of the structures in Asunción due to an earlier onset of carbonation-induced corrosion could be expected, which could be caused by faulty construction practices or a cover thickness under the minimum value. Also, the approach of this chapter is solely oriented to the carbonation-induced degradation, so when considering the other various degradation mechanisms, the service-life time of these structures could be even lower than those indicated by the simulations. Similarly, the climate change effects will create favourable conditions (i.e. increase in temperature and CO_2 concentrations) for the corrosion induced by carbonation to be worse in the coming decades. Therefore, it will be necessary to take actions to counteract the expected degradation rate due to this global climatic phenomenon.

4.5 Summary

Since the end of the last century, many numerical models for the prediction of the service life of concrete structures have been developed in several countries. Many of these models are part of independent scientific research as well as projects that were created specifically with the aim of expanding knowledge regarding the durability of structures. It is significant to mention that depending on the degradation condition of a structure, the maintenance and repair tasks could vary both in its complexity and in the expected cost. Therefore, it is essential to be able predicting the deterioration times in order to perform an optimised intervention in the structure. The evolution of the complexity and reliability of these models with the passage of time has been addressed in this chapter.

On the other hand, the probabilistic methods are better adapted to the reality of the degradation of the structures since they consider the randomness of the variables, which is a significantly greater advantage over the deterministic methods. However, its disadvantage lies in the fact that the probabilistic method uses complex mathematical models that hinder its applicability in some cases. In turn, the engineering methods seem to show the future of the models of the service life prediction, since these methods seek to apply the probabilistic method more simply. However, there are still not many studies developed under this methodology in the literature.

It is important to have certain considerations regarding the results obtained when applying a prediction model since they are stated according to specific conditions and parameters that could not be suitable in the context of the study developed. For example, in the case of numerical models, considerations are made based on a set of values for the parameters, which are sometimes arbitrarily adopted; and in the case of empirical models, it is important to take into account the exposure conditions considered for their formulation. For the case of the numerical model described in this chapter, it has been validated through accelerated carbonation test where climatic conditions have been set for different cities around the world. Nonetheless, these countries corresponded to climate regions completely different from those that are found in Paraguay. Therefore, the parameters initially used in the model were adjusted to represent the degradation rate of the country's structures accurately.

Many of the numerical models developed are based on the degradation of concrete caused by corrosion and its different mechanisms (carbonation, chloride attack, among others). Considering then the stochastic nature of these degradation mechanisms, the quantification of the limit state through the results obtained in the laboratory or on-site inspection is often a complex task. On the other hand, the lack of understanding of the degradation process or the material properties often forces arbitrary assumptions that are made during the formulation of the model. For this reason, these assumptions must be verified through a validation that ensures the reliability of the results.

One of the most significant handicaps of the degradation prediction models is the difficulty in adopting the parameters in a precise and quantifiable way. In some cases, the input parameters are set arbitrarily due to the inability to obtain representative or easily updated values, such as steel-concrete interface size, the chemical composition of the formed corrosion product, modulus of elasticity, Poisson's ratio, and coefficient of sliding of concrete. In this chapter, the influence of a parameter (thickness of the porous zone in the steel/concrete interface) used in the carbonation model for obtaining the results has been demonstrated. For this case, a single parameter can define the propagation times of the corrosion in the structure as has been seen in Section 4.4.2.

Another point to consider during the formulation of mathematical prediction models is that to justify its application in a specific study, it is necessary to perform a corresponding validation previously. This process consists in sustaining the results obtained in the model through a comparison with the experimental results. Many of the models found in the literature have never been validated, or the corresponding validation has not been successful, as it is in the case of the models proposed by Bažant in 1979 and by Molina *et al*. in 1993, where its application in real structures is not supported by reliable results.

Regarding the validation of results, a technique commonly applied consists of accelerated tests performed in the laboratory (carbonation or accelerated corrosion). Despite being a widely used technique, its effectiveness and reliability have always been questioned with the argument that the results are not representative of the natural behaviour of the degradation process. Many studies have shown that there are certain assumptions or mathematical formulations that cannot be validated by this means, as it could lead to erroneous estimates of degradation. For this reason, it is recommended that the results obtained in accelerated tests be applied carefully for the validation of numerical models.

Many contradictions can arise when evaluating several degradation models. This is commonly due to several factors such as the lack of knowledge of the chemical composition of corrosion products, the lack of availability of data that require arbitrary assumptions for certain values, errors in the formulation of the criteria, incorrect validation, adoption of the limit states, among others. However, if the model is well chosen, the formulation of maintenance and repair strategies can be satisfactorily established based on these prediction models.

It is true that these numerical models seek to describe, in an increasingly precise way, the natural behaviour of the degradation processes of structures. However, there is still necessary to carry out a lot of research in this area to achieve this objective entirely. Recently, many degradation models have sought to link the phenomenon of climate change and its influence on degradation parameters, which represents a significant advance. On the other hand, the estimation of the service life of structures under the influence of cracking or the system of loads to which a real structure is subjected is still insufficiently studied due to its complexity. In the context of this research, the influence of cracks on the carbonation rate and the corrosion rate are determining factors for obtaining more realistic results.

The degradation model applied in this chapter was considered suitable to represent the estimated carbonation-induced degradation of concrete structures over time. Likewise, as a function of a climatic scenario and concrete properties, it can determine the service life of RC structures under this degradation mechanism. This feature allows the model to predict the degradation of concrete structures under consideration of climate change, which is directly related to the context of this research.

An attempt to correlate the results of a numerical carbonation model with real carbonation data from existing structures has been developed. The lack of information in the structures of the case study has been a significant obstacle in the study. However, it was possible to notice that the model has shown, regarding the control scenario, a slightly better correlation for the sheltered concrete than for the unsheltered one in existing structures. This result is highly influenced by the amount of data used to make the correlation analysis. For the case presented in this work, the real carbonation data were much higher for the unsheltered concrete elements than for the sheltered ones, which may affect the significance level of the correlation. Therefore, after the analysis herein presented, it can only be concluded that the carbonation model is more reliable for sheltered structures than for unsheltered.

Nonetheless, further research is necessary to improve the correlation of the model for existing structures considering their weathering exposure. It should be mentioned that the applied model in this chapter does not consider this aspect. Considerations regarding the effect of relative humidity in both indoor and outdoor environments on the diffusion coefficient could be made to consider the difference in carbonation depth between sheltered and unsheltered concrete elements. Nevertheless, that analysis is beyond the scope of this research Thus, the findings of this study suggest that the best way to achieve a more reliable correlation between the carbonation model results and the *in-situ* test data would be a statistical analysis based on the measurements that must be performed periodically in the same set of structures. However, this would involve performing tests at specific intervals that lead to decades of study and analysis to obtain a broad database.

One of the most outstanding results given by the carbonation model for the structures of Paraguay was the expected early degradation in the next years due to climate change effects, see Figure 4.8. Thus, for the worst climate scenario, in the second half of this century is expected an average increase by 25 %, in the maximum carbonation depth regarding a control scenario. Meanwhile, the time to reach the same maximum carbonation depth of the control scenario can be reduced between 7 and 10 years for the best climate scenario, depending on the quality of the concrete. Furthermore, the curves obtained for Asunción also suggest that concrete with higher quality decreases the value of the ultimate carbonation depth. However, considering the results of Table 4.4, it is possible to notice that corrosion initiation times increase more significantly for structures with a higher cover thickness (from 10 to 25 mm) than for structures with a better concrete quality (from 20 to 25 MPa). Therefore, according to the study developed in this chapter, the cover thickness is referred as the most influential parameter to reduce the time to corrosion initiation.

The primary objective of this chapter has been to develop state of the art regarding the modelling of concrete structures degradation by carbonation. Among the existing models in the literature, one has been chosen that fits within the context of this thesis. The carbonation model allows knowing the expected degradation curves for the concrete structures of Paraguay under the influence of climate change. In this way, the next chapter of this research seeks to propose maintenance strategies that allow facing this problem and thus preserve the durability of these structures.

CHAPTER 5

Maintenance Strategies for RC Structures

CHAPTER 5

5 MAINTENANCE STRATEGIES FOR RC STRUCTURES

5.1 Introduction

So far, this research has developed a comprehensive analysis regarding the degradation of RC structures caused by corrosion. It has shown the incidence of this anomaly in the durability of these structures and the influence that climate change could have on the degradation of structures. It has also been seen that modelling the degradation process of a structure comprises a complex task where a large number of variables must be taken into account. These variables, in turn, have a stochastic behaviour for which it is necessary to include analyses from a probability perspective.

Regarding the study of the maintenance management of structures, it is necessary to establish the same approach as for the study of degradation. The state of a structure throughout its service life cannot be accurately predicted, so its modelling must always be approached from a probability perspective. Then, the maintenance planning must be formulated as the optimal execution of the activities subject to one or more constraints. One of the most common constraints is the budget limitation, which determines that a structure cannot be intervened at any time and that maintenance cannot be performed continuously. Furthermore, inspections represent an important task in maintenance since inspections help to make decisions and their results define the future condition of the structure. Hence, maintenance and inspections should be scheduled in such a way that costs are minimised, reliability and safety maximised, or the combination of these objectives are considered in an optimised manner (Kallen, 2007).

In this chapter, it is proposed strategies related to the maintenance of reinforced concrete structures with the risk of degradation due to corrosion of the reinforcement. These strategies mainly comprise two approaches. On the one hand, the optimised inspections planning which includes the probability of damage detection of the inspection techniques and cost analysis. On the other hand, a dynamic decision-making model that considers not only inspections but also repair activities and its influence over the service life of the structures. In this way, this chapter will be conducted into two main studies that are described in sections 5.2 and 5.3.

Reinforced concrete is one of the most frequent used materials worldwide in structures and infrastructures due to its versatility and relatively low cost (Saetta and Vitaliani, 2004). Therefore, the study of its performance and durability comprises an important issue of scientific research in the field of civil engineering, which has been developed for several decades (Roque and Moreno, 2005; Folić, 2009; Correa et al., 2010; Fan et al., 2010). Nevertheless, as well as any other constructive material, concrete tends to deteriorate, compromising its service life. Because of that, establishing maintenance strategies for RC structures is a fundamental task to guarantee its durability.

As it was mentioned in previous chapters, corrosion of the reinforcement is one of the most typical and expensive causes of the degradation of RC structures. Corrosion in reinforced concrete may be caused by two typical phenomena: carbonation and chloride attack, being the former most favourable in tropical regions and the second one in the coastal environment (temperate climate regions) (Ekolu, 2016). Regarding carbonation, the cement paste in the structure remains exposed to atmospheric carbon dioxide once the concrete structure has been constructed and placed into service. Thereby, this environment- structure interaction enables that carbonation takes place commonly in urban areas (Wang et al., 2018).

Considering the foregoing, a proper maintenance strategy should be indispensable to preserving the performance of structures above its required level of safety and reliability. Maintenance affects the reliability of components and the system: if too little is done, this can cause an excessive number of costly failures and poor system performance and, therefore, reliability is degraded. Instead, if it is done too often, reliability may improve, but the maintenance cost will increase drastically. Then, in a cost-effective scheme, both expenditures must be considered (Endrenyi et al., 2001). Therefore, the maintenance management should be a tool that satisfactorily ensures the reliability of the construction system, whose purpose is the extension of the service life of a constructive element or, at least, the mean time for the occurrence of the upcoming failure whose repair may be expensive (Endrenyi et al., 1998). For the optimal planning of the inspection, it is necessary to develop a systemic decision-making framework, which must consider several methods and address a large number of objectives involved in the optimisation problem from the point of view of efficiency (Kim and Frangopol, 2018). Nonetheless, the choice of an inspection method, as well as the time of the intervention, will affect the decision-making results both technically and economically (Malioka, 2009).

The maintenance strategy may be structured as a two-stage process as is depicted in Figure 5.1. On the first stage, the inspections activities are planned, being necessary an assessment of the time-dependent degradation process during its service life, thereby establishing the proper inspections times. On the second stage, repair probabilities are analysed regarding the inspections results in order to face the damage before the occurrence of a failure. When considering the inspections planning before the occurrence of the failure, maintenance management is carried out under the perspective of preventive maintenance. As will be seen later, this approach is the most appropriate to ensure the durability and safety of the structures at a controlled cost.

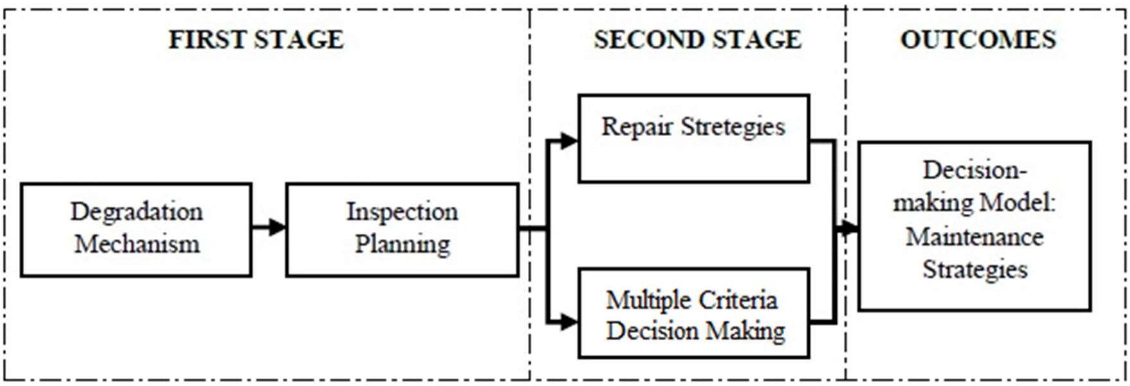


Figure 5.1 Proposed framework for the formulation of maintenance strategies.

Several studies have been developed in the literature on the inspections planning through optimisation techniques of objectives subject to specific constraints. These studies allow knowing a set of optimal strategies to perform maintenance depending on the criteria taken into account by stakeholders. However, these studies do not allow knowing properly which of the maintenance strategies is the most appropriate. To face this limitation found in the studies that only apply multi objective optimisation techniques, this research proposes to carry out a combined analysis with an efficiency analysis. Therefore, the outcome of this study provides an inspection strategy that establishes the optimal solution for both the inspection times and the most suitable inspection technique to be applied considering the efficiency of the solution.

In the second stage of the formulation of the maintenance strategies, the repair activities are collectively analysed with the inspections. The purpose of this study is to provide a useful and easy application tool for the maintenance of structures. For this, the decision-making method known as the Analytic Hierarchy Process (AHP) is applied in this second stage. The AHP method is a multi-attribute decision-making method that has already been applied previously in other studies for maintenance planning. Nevertheless, the method has the disadvantage that it performs a somewhat subjective analysis, which is why it has received criticism from some researchers. Therefore, this study adopts the traditional AHP method to formulate a dynamic model for decision-making that includes the stochastic approach within the analysis. In this way, the subjectivity of the final results is reduced, and it is possible to formulate a model adapted to the requirements of the maintenance management in structures.

5.2 Optimal Inspection Planning

In this section, the first stage of the maintenance strategy management is considered as an optimisation problem. The inspection planning consists in determining when to inspect the structure and which inspection technique is more suitable to be applied. So, the optimisation problem aims to minimise the cost associated with such inspection planning. Moreover, the study herein presented is focused on maintaining a minimum level of reliability regarding the probability of failure in the structure. The parameters involved in the optimisation correspond to the corrosion degradation in RC structures and the cost related to each inspection technique and, therefore, the total cost of the inspection planning. Thus, due to the natural uncertainty of the degradation process, the work herein presented is developed based on a probabilistic approach.

Likewise, an efficiency analysis is performed in order to propose a more suitable solution regarding the inspection management of RC structures. So far, much of the research in the maintenance of structures result in a set of optimal solutions/strategies without discretising which of these solutions is the best overall. Therefore, through an analysis of efficiency, this research seeks to identify which is the best strategy among a set of solutions that meets several criteria simultaneously. Two methods were assessed to verify its feasibility for the inspection planning, namely the Stochastic Frontier Analysis (SFA) and the Multidirectional Efficiency Analysis (MEA). This efficiency analysis aims to study the results of the optimisation and evaluate if the solutions provided are efficient.

The efficiency analysis is then performed considering mainly two aspects: the number of different inspection techniques applied in the inspection planning; and the total period employed by the inspection planning, i.e. the time gap between the first and last intervention in the structure. These efficiency parameters are quite meaningful since it not only considers the durability of the structures but also the particular interests of a construction company on a competitive level. Regarding the latter, the efficiency analysis seeks the solution that requires the least amount of resources and spends the least possible time of work to achieve the same or better results than those obtained only with an optimisation analysis over the inspection costs.

Studies on efficiency analysis have been widely developed in other fields of scientific research such as agriculture, technological innovations, economics, industrial productivity, transportation, health sciences, energy among others (Lim et al., 2012; Wang et al., 2013; Giorgio et al., 2016; Otsuka, 2017; Vasco et al., 2017; Balliauw et al., 2018). Nonetheless, it is difficult to find studies in the literature that apply these methods of efficiency analysis in the area of civil engineering structures and infrastructure maintenance. Therefore, the main purpose of this section is proposing a decision-making model for the inspection planning in RC structures under risk of carbonation-induced corrosion. This planning comprises a strategy that establishes the optimal solution for both the inspection times and the most suitable inspection technique to be applied considering the efficiency of the solution. The degradation mechanism by carbonation is considered in this study through the formulation of corrosion initiation time.

5.2.1 Efficiency Analysis

The optimisation of the inspection and repair activities in infrastructures seeks to improve the processes of these tasks in order to increase performance and productivity. Likewise, the term optimisation is commonly associated with the efficiency of the obtained result. Therefore, the term "efficient" and "optimal" are often related (Balk, 2001). Unfortunately, the construction industry has received less attention in economic literature than other sectors of the economy and, for this reason, there is little research that analyses the efficiency of the construction sector (Fernández-López and Coto-Millán, 2015). The analysis performed in this section attempt to ensure that the best possible performance is achieved while the minimum amount of resources is used.

First of all, it is important to be clear about the term *efficiency*. Basically, the efficiency analysis is based on four elements: decision-making units, inputs, outputs and a production function. The latter is a mathematical function that describes the transformation of inputs into outputs (Mutz et al., 2017). It is common to misunderstand the term productivity with the term efficiency, and despite being related to each other, they are not the same thing. The production frontier defines the relationship between the input and output where the maximum output achieved for each input level is represented. When a solution is on that frontier, then it is technically efficient and, otherwise, it will not be technically efficient (Coelli et al., 2005). In Figure 5.2, the inputs are represented in the abscissas axis while the outputs are located on the ordinate axis. The line $\overline{OF'}$ represents the production frontier function relatively which the point A is an inefficient solution while B and C represent efficient points.

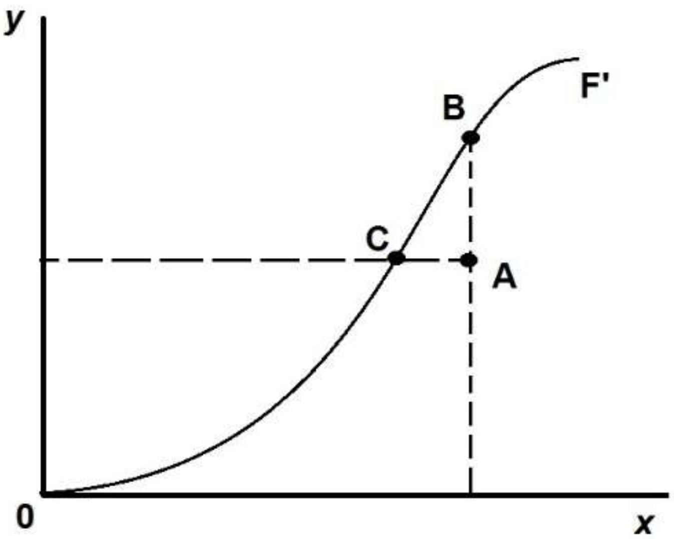


Figure 5.2 Production Frontiers and Technical Efficiency (Coelli et al., 2005).

In past decades, it has been evident that an exerted pressure on organisations of all kinds due to the competitiveness of a global world where the interlaced finance markets and the linked crises are common. This trend requires more performance understanding and more resilience, especially regarding measurement, monitoring and, hence, the finding of its inefficiencies. This inefficiency is determined through an efficient frontier where the distance to that limit is measured indicating the potential of efficiency enhancement. Therefore, this frontier represents the maximum of different outputs with different combinations of inputs and, furthermore, the minimal combination of required inputs for different outputs is also considered (Lampe and Hilgers, 2015).

For the efficiency analysis, two inputs were considered referring to the strategy adopted in the inspection plan. These two inputs were, on the one hand, the number of inspection techniques applied during the period of service life of the structure. Secondly, the time gap between the first and the last intervention performed was also analysed. Both inputs are considered important from the viewpoint of the resources with which a company can count for the maintenance management of structures. Therefore, the efficiency within the inspection planning can be achieved through a decision model that allows the use of the smallest number of resources (inspection techniques) in a shorter intervention period and ensuring the minimum reliability.

Defining how outputs should be measured is an integral part of efficiency analysis. In economics theories of production can be found two commonly used methods for measuring the efficiency. These are the data envelopment analysis (DEA) and the stochastic frontier analysis (SFA) (Shankar, 2014). DEA uses mathematical programming methods from a non-parametric approach to identify the highest output levels by combining different inputs. SFA applies econometric methods from a parametric approach to identify the highest production levels (Holmgren, 2018). The parametric approach (SFA) is probably the most common method for the estimation of production frontiers (Novaes et al., 2010).

Meanwhile, the DEA method has the advantage of being able to deal with multiple inputs/outputs that could be more appropriate for modelling the decision-making strategy in maintenance planning, where various criteria are often considered. As a derivation of the traditional DEA approach, in this research is applied the Multi-directional Efficiency Analysis (MEA) method to assess the efficiency of the inspection planning. The MEA method was first addressed by Bogetoft and Hougaard in 1999 and since then several studies have been developed supporting its applicability for the efficiency analysis (Bogetoft and Hougaard, 1998).

In a multiple input/output model, efficiency may be formulated as the ratio of the weighted sum of outputs to the weighted sum of inputs. In DEA, the weights for the inputs and outputs are determined using mathematical programming, so that each weight maximises the efficiency of the assessed decision-making units while restricting the efficiencies of the other units within 0 and 1. Thus, DEA differs from other efficiency methods since, unlike statistical models such as the SFA, it does not require imposing an explicit functional form (Sakthidharan and Sivaraman, 2018). On the other hand, the SFA method makes assumptions about the parameters of the population distribution from which data are drawn. The deterministic and stochastic frontiers are included in this parametric approach and depend on the assumptions regarding the disturbance terms. Then, SFA may be defined as an alteration of a regression model characterised by a composite error that comprises the inefficiency terms and a stochastic component, which measure the possible non-controllable conditions that a utility would deal (Guerrini et al., 2018).

Several choices must be made when defining the SFA model such as cost or production function, functional form, and the treatment of unobserved heterogeneity. Firstly, cost and production may be established through a formulation that measures the highest amount of output that can be produced from a given amount of input. Thus, a cost function estimates the lowest cost incurred to produce a bundle of output, given input prices. Subsequently, the functional form may be derived from the Cobb-Douglas function. However, some limitations may be found with this approach so is advisable to apply a translog cost function. Lastly, the treatment of unobserved heterogeneity may include models with time-invariant and time-variant inefficiency terms, and models that are disentangling individual and time-invariant heterogeneity from time-variant inefficiency (Guerrini et al., 2018).

Regarding the efficiency method applied in this research, the multi-directional efficiency analysis (MEA) defines a vector of efficiencies corresponding to a benchmark constructed from the improvement potentials in each of the variables and thus is not limited to the radial contractions of inputs or radial expansions of outputs of a traditional DEA analysis. Therefore, the MEA method is ideally suited for analysing decision problem where the objective is to reduce the consumption of some inputs as well as enhance the production of some outputs, but without demanding direct assumptions about the relative importance of some improvements over others (Asmild and Matthews, 2012). An overview of the MEA methods is depicted in Figure 5.3, where $X(X_1, X_2)$ is the input combination; L is the production possibility set determined by convex data envelopment; S^R, S^F, S^{PI} are productions plans; and X^R is the ideal production (Holvad et al., 2004),

The implementation process of the MEA method is based mainly on two sequential steps. Firstly, the input coordinates of an ideal point are derived by solving linear programmes for each input separately. The ideal reference point denotes the most significant possible reduction in each input. In the second step, a single linear programme is implemented in which the outcome is used to compute a vector of input-specific inefficiencies. Furthermore, it should be mentioned that the MEA approach can also be generalised to the full input-output space in the efficiency analysis (Kapelko and Oude, 2017).

Considering that MEA scores are absolute, a large part of inefficient production tends to reach large MEA scores. This aspect has a determining impact when the MEA scores are compared with the relative scores of the DEA method. The interpretation of the MEA scores can be somewhat complex, which makes the method less interesting from the analyst's point of view. However, MEA is based on a better benchmark selection where the potential for "local" improvement is considered (Asmild et al., 2003).

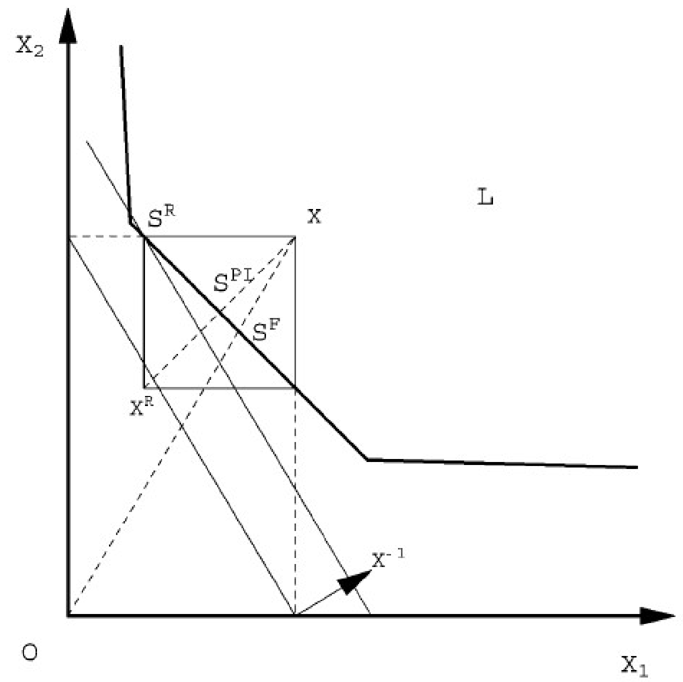


Figure 5.3 Implementation of MEA model for efficiency analysis (Holvad et al., 2004).

Therefore, when comparing both models, the MEA scores are generally the largest, i.e. a greater excess in the use of inputs is identified. Likewise, the difference between scores is independent of all the characteristics of the data set, this being the main difference between the MEA and DEA method (Asmild et al., 2003). Further details and features regarding the mathematical formulation of the efficiency analysis methods are developed in the next section of this chapter.

5.2.2 Mathematical Formulation for the Inspection Planning

The degradation of infrastructures requires an analysis highly governed by stochastic processes, for which reason their study must be addressed mathematically from a probabilistic approach. Herein, the mathematical formulation for the optimal inspection planning of concrete structures is presented sequentially. This approach is based on structural reliability and aims to determine both the intervention times and the most appropriate inspection technique for the maintenance of concrete structures. Furthermore, the analysis comprises the costs associated with this inspection planning strategy. Subsequently, the results obtained from the multi-objective optimisation are subjected to an efficiency analysis that allows to minimise the use of resources (different inspection techniques) and to minimise the total maintenance time required in the structure. Figure 5.4 shows a scheme for the mathematical formulation of the decision-making model.

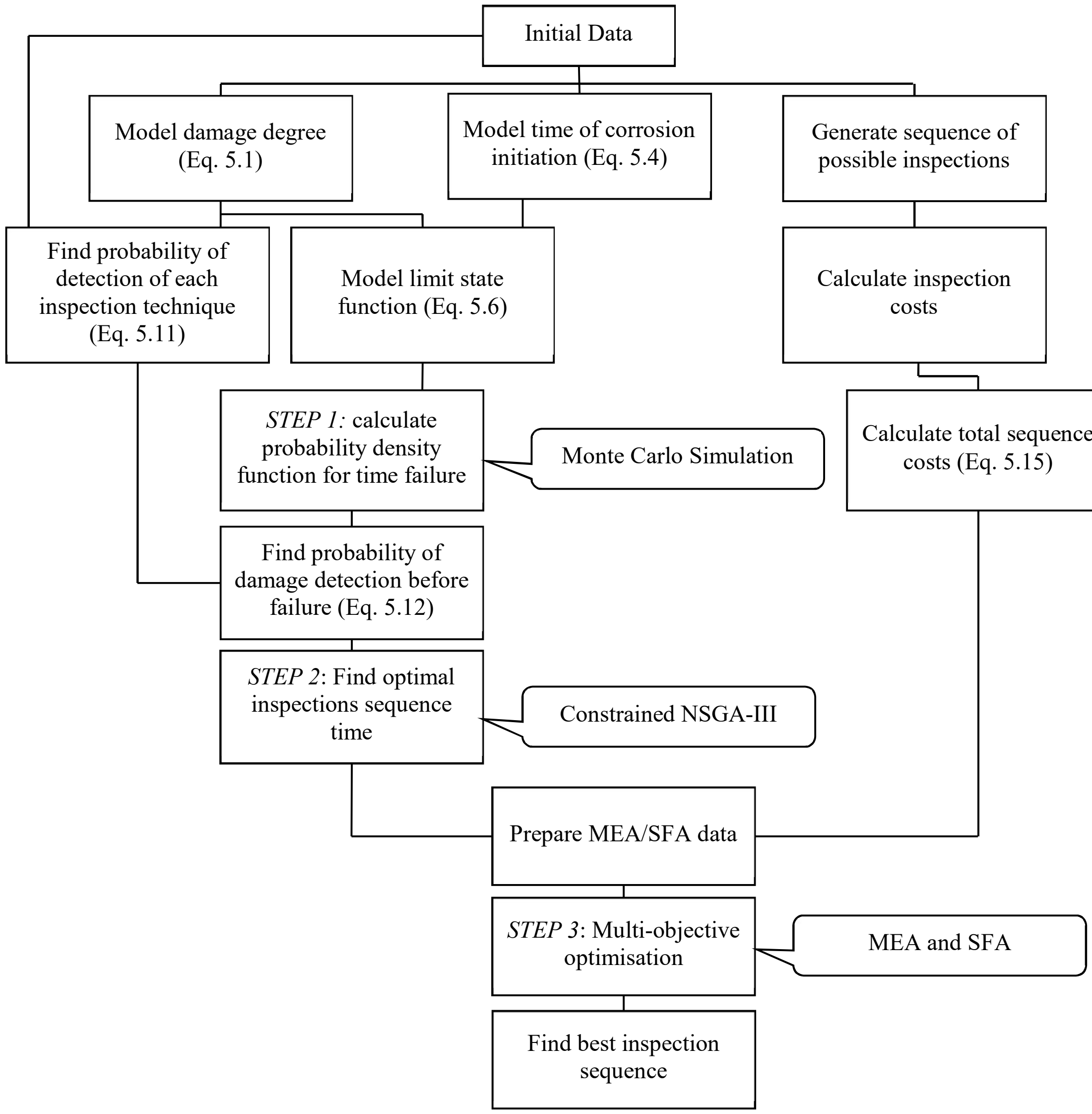


Figure 5.4 Flowchart for the inspection planning formulation.

5.2.2.1 Step 1: Probability distribution of failure time

§*1.1 Damage Degree and Time of Corrosion Onset*. First of all, it is quite meaningful to define the process of damage in the structure. For this, the damage degree caused by corrosion on the reinforcement of RC structures may be quantified by the function denoted as $\eta(t)$ that range from zero to one. Thus, the damage degree $\eta(t)$ is formulated as the cross-sectional loss that the reinforcement experiences due to the corrosion phenomenon as follow (Frangopol et al., 1997):

$$\eta(t) = \begin{cases} 0, & t < T_{icorr} \\ \dfrac{d_0 - d(t)}{d_0}, & t \geq T_{icorr} \end{cases} \qquad with \qquad d(t) = d_0 - 2V_{corr}(t - T_{icorr}) \tag{5.1}$$

which, for $t \geq T_{icorr}$, Equation (5.1) is invertible as:

$$t = \frac{d_0 \eta}{2V_{corr}} + T_{icorr}, \tag{5.2}$$

where d_0 is the initial rebar diameter (cm); $d(t)$ is the rebar diameter over time (cm); V_{corr} is the corrosion rate (cm/year); T_{icorr} is the time of corrosion initiation (years) and t is the time of intervention over the service life of the structure (years). It should be noted that for $t < T_{icorr}$ the corrosion does not exist yet and, hence, the corrosion damage is zero. The factor 2 in Eq. (5.1) considers the uniform corrosion propagation process on the surface of the reinforcement. Moreover, it should be mentioned that this research focuses its study on buildings structures with a design for a service life of 50 years (EN-1990, 2005).

Studies define the corrosion deterioration as a three-stage process: onset of corrosion, crack initiation and crack propagation. In the third stage, crack propagation and spalling of the concrete cover lead to the service failure of structures (Cui and Alipour, 2018). The initiation time for carbonation-induced corrosion T_{icorr} is a random variable that may be determined by the Häkkinen method which states that (Hakkinen, 1993):

$$t(C_d) = \left(\frac{C_d}{\alpha_t C_{env} C_{air} \left(f_{ck} + 8\right)^{\beta_t}} \right)^2 \tag{5.3}$$

where $t(C_d)$ is the time (in years) for attaining the carbonation depth C_d (mm), C_{env} and C_{air} are the environmental and air content coefficient respectively, α_t and β_t are parameters depending on the binding agent, an f_{ck} is the characteristic compressive strength of concrete (N/mm^2). Then, T_{icorr} is obtained from Eq. (5.3) by setting the value of C_d equal to the critical carbonation depth (d_{cr}) for corrosion initiation. This parameter allows introducing the effect of carbonation-induced corrosion in the context of the study. As was found in (Yoon et al., 2007), the carbonation-induced corrosion begins once the carbonation front is located at least within 5 mm of the rebar surface. Therefore, the critical depth may be formulated as $d_{cr} = b - 5$, where *b* is the cover thickness of concrete (mm).

$$T_{icorr} = \left(\frac{b - 5}{\alpha_t C_{env} C_{air} \left(f_{ck} + 8\right)^{\beta_t}} \right)^2 \tag{5.4}$$

Determining the time above is essential for the inspection planning since the damage degree is analysed from this time until the structure reaches the failure. Although the time to corrosion initiation has been determined through the carbonation model adopted and applied in Chapter 4, the modelling of the inspection planning requires that this variable is considered from a stochastic perspective. Therefore, the value of T_{icorr} will be solved again in this section. Nonetheless, this value will be similar to the values obtained in Table 4.4 of the previous chapter.

§*1.2 Time to attain the failure condition.* Whether the residual strength of a structural component is defined by a stochastic process and the characteristics of loads or actions in the structure are known, then the probability of failure of the structure can be evaluated as a function of time (Ellingwood and Mori, 1997). In a structural reliability assessment, the construction system needs to fulfil with certain requirements regarding safety and functionality, which can be evaluated from the point of view of the ultimate limit state or the service limit state (Afzal et al., 2016). If these requirements are not met, then the system could be considered under failure condition. On these terms, the probability of failure P_f may be formulated as:

$$P_f = P\big[g(\eta(t)) \geq \eta_{cr}\big], \tag{5.5}$$

where $g(\eta(t))$ is the limit state function that defines the time for which the structure attains a specific damage degree. Hence, from Eq. (5.2), the limit state function may be expressed as:

$$\eta(T_f) = \eta_{cr} \quad \Leftrightarrow \quad T_f = \frac{d_0 \eta_{cr}}{2V_{corr}} + T_{icorr} \tag{5.6}$$

where T_f is the failure time (years) and η_{cr} is the critical damage degree for which a structure reaches the failure. In reliability analysis, it is possible to approach different criteria to determine the failure of a structure according to different limit states, for instance, the serviceability limit state and strength/stiffness limit state (Wang et al., 2017). The verification of the limit state must be developed through a function known as the limit state function $G(X)$. This function associates the loads (S_G) to which the structure will be subjected and the corresponding design resistance (R_G) as is shown in Equation (5.7). All these variables of the reliability function are time-dependent and defined by stochastic variables, i.e. their expected values may change over time. The failure in the structure is presented when $G(X) \leq 0$. Hence, the probability of failure should not be obtained as an absolute value but relative to the time of service life (Neves et al., 2012). Thus, the failure condition in a structure may be considered as the loss of capabilities to fulfil the functions that are required (ISO 15686-1, 2000).

$$G(X) = R_G - S_G \tag{5.7}$$

Considering the above, the failure analysis can be appraised through a carbonation reliability approach, where the corrosion cumulative failure probability is estimated regarding the cross-sectional loss in reinforcement. Commonly, the concrete carbonation depth is considered as a load effect "S_G", and the cover thickness as carbonation resistance "R_G" in order to obtain the limit state function "$G(X)$"(Jiao et al., 2016). However, in this research, the critical damage degree is evaluated over the life time to obtain the probabilities of corrosion failure in a RC structure. As has been seen in Chapter 3, corrosion damage in RC structures causes a loss in the cross-sectional of the rebar that decreases its bond strength (Broomfield, 2007). When the reinforcement registers a cross-sectional loss greater than 25% of its original state, it could present changes in the structural behaviour and reduce the margin of safety significantly (Cheung et al., 2012). According to Eq. (5.1), this cross-sectional loss is directly related to the damage degree in the structure. Consequently, a critical damage degree η_{cr} that defines the structural capacity is assumed equal to 0.25.

§1.3 Probability Density Functions $\psi_{T_{icorr}}(t)$ and $\psi_{T_f}(t)$. Considering the uncertainty of the above values, Monte Carlo Simulation (MCS) can be a favourable tool to performing the probability of failure for a structure given a limit state function of failure that describes the degradation mechanism. Then, both $T_{icorr}$and T_f are random variables which are determined as non-linear functions of other random variables and fixed parameters. Thereby, this simulation technique provides the probability density function regarding the time to failure $\psi_{T_f}(t)$ of the structure over its lifespan.

Thus, considering the variables of Eq. (5.6) as random variables and developing a number of iteration in the MCS, the probability distribution for the failure time is obtained. Similarly, by applying MCS in Eq. (5.4), the probability distribution for the corrosion initiation time $\psi_{T_{icorr}}(t)$ is found. The assumed values for such random variables and their respective distributions will be addressed in Section 5.2.3 of this chapter.

5.2.2.2 Step 2: Optimal inspection sequence times

§*2.1 Inspection Techniques and their Detectability Function*. In this chapter, it is considered a set of different inspection techniques represented by $\theta \in \Theta = \{A, B, C, \cdots\}$. These inspection techniques can be any technique available to detect, in this case, corrosion in the reinforcement of concrete structures. The quality of an inspection technique θ is usually characterised by a probability of detection function (i.e. detectability of θ) which depends on $\eta_{0.5}^{\theta}$ and σ^{θ}, where $\eta_{0.5}^{\theta}$ is the damage intensity at which the inspection technique has a 50% probability of detection, and σ^{θ} is its standard deviation of this variable (Frangopol et al., 1997).

The detectability function of θ may be modelled in several ways depending on the deterioration mechanism and building structure (Chung et al., 2006; Sheils et al., 2010; Kim and Frangopol, 2011; Yang and Frangopol, 2018b). For corrosion damage, the expression in commonly defined as

$$P_D^{\theta}(\eta) = \begin{cases} 0 & \text{, if } \eta \leq \eta_{min}^{\theta} \\ \Phi\left(\dfrac{\eta - \eta_{0.5}^{\theta}}{\sigma^{\theta}}\right) & \text{, if } \eta_{min}^{\theta} < \eta \leq \eta_{max}^{\theta} \\ 1 & \text{, if } \eta > \eta_{max}^{\theta} \end{cases} \tag{5.8}$$

Where Φ is the standard normal cumulative distribution function (CDF), η is the damage degree and η_{min}^{θ}, η_{max}^{θ} are given values that represent the threshold damage degree for the detectability. It should be noted that the function P_D^{θ} is not continuous due to the boundary established by the maximum and minimum damage degree. Defining an accurate value for the parameters considered into the formulation of detectability function involve a complex task. Thus, several studies assume values for these parameters as $\sigma^{\theta} = 0.1\eta_{0.5}^{\theta}$, $\eta_{min}^{\theta} = 0.7\eta_{0.5}^{\theta}$, and $\eta_{max}^{\theta} = 1.3\eta_{0.5}^{\theta}$ (Frangopol et al., 1997; Kim and Frangopol, 2011). Under such assumptions, the discontinuity gap is quite mild, but additionally, for each fixed value of η, the detectability function may be expressed as

$$P_D^{\theta}(\eta) = \Phi\left(\frac{\eta - \eta_{0.5}^{\theta}}{0.1\eta_{0.5}^{\theta}}\right) = \Phi\left(\frac{\varrho^{\theta} - 1}{0.1}\right) \qquad with \quad \eta = \varrho^{\theta}\eta_{0.5}^{\theta} \tag{5.9}$$

Where ϱ^{θ} is an expression that considers the damage parameters for the inspection technique. From the Eq. (5.9) it can be inferred that if $\eta_{0.5}^{\theta_1} \geq \eta_{0.5}^{\theta_2}$ then $P_D^{\theta_1}(\eta) \geq P_D^{\theta_2}(\eta)$ because Φ is non-decreasing. Therefore, the best inspection technique turn-out to be always the one with the smaller $\eta_{0.5}^{\theta}$ value. In order to address this discontinuity, a natural way to improve the detectability function of an inspection technique θ is to assume that

$$P_D^{\theta}(\eta) = \begin{cases} p_1(\eta) & \text{, if } \eta \leq \eta_{min}^{\theta} \\ \Phi\left(\dfrac{\eta - \eta_{0.5}^{\theta}}{\sigma^{\theta}}\right) & \text{, if } \eta_{min}^{\theta} < \eta \leq \eta_{max}^{\theta} \\ p_2(\eta) & \text{, if } \eta > \eta_{max}^{\theta}, \end{cases} \tag{5.10}$$

where p_1 and p_2 are polynomials of degree *n* verifying

$$p_1(0) = 0, \qquad p_1(\eta_{min}^{\theta}) = \Phi\left(\frac{\eta_{min}^{\theta} - \eta_{0.5}^{\theta}}{\sigma^{\theta}}\right), \qquad p_2(\eta_{max}^{\theta}) = \Phi\left(\frac{\eta_{max}^{\theta} - \eta_{0.5}^{\theta}}{\sigma^{\theta}}\right), \qquad p_2(1) = 1$$

Another choice for p_1 and p_2 is to assume it as cubic splines, so the resulting polynomials will agree in monotonicity and concavity at the boundary points of the middle part of P_D^{θ}, which is generated by the standard normal CDF. Finally, the detectability function of θ may be formulated as a function of time considering the expression of η from Eq. (5.1) as

$$\mathcal{P}_D^{\theta}(t) = P_D^{\theta}\left(\frac{2V_{corr}}{d_0}(t - T_{icorr})\right) \qquad for \quad t \geq T_{icorr} \tag{5.11}$$

§*2.2 Inspection Sequences for an Early Detection of Damage.* The structural health monitoring is quite meaningful for in-service RC structures where corrosion of steel reinforcement is a serious problem. Therefore, the early warning of corrosion is in practical demand in order to maintain concrete structures under safe and durable conditions. However, considering the difficulty of quantifying corrosion damage in its early stage of development, accomplishing a proper intervention is often a great challenge (Patil et al., 2017).

One of the main objectives in the maintenance planning of structures is the need to detect the damage before the failure is reached. Achieve this goal depends mainly on the damage degree in the structure at the inspection time, as well as the detectability of the applied inspection technique. It should be noted that the damage detection will be influenced significantly by the uncertainty level. This influence will be more meaningful for those elements whose damage degree are not so prominent since it is complex to determine if the variation of modal parameters come from damages or uncertainties (Xu et al., 2015).

Hence, for a set of inspection techniques $\theta \in \Theta = \{A, B, C, \cdots\}$, it is defined a set of inspection sequences $S_{N_s} = \{A, B, C, AA, AB, \ldots\}$ for all possible combinations of elements Θ up to N_s elements. Then, for any particular sequence ρ, the number of techniques in the sequence is defined as $|\rho|$, and the method which lies at the position $i \in \{1, \ldots, |\rho|\}$ is expressed as ρ_i. For instance, for the sequence $\rho = ABC$, it can be seen that $|\rho| = 3$, and $\rho_2 = B$. As was defined by Soliman *et al.*, the probability of damage detection before failure P_{DBF}^{ρ} of an inspection sequence $\rho \in S$ can be formulated considering both the probability of detection of such inspection sequence $\mathcal{P}_D^{\theta}(t)$ and the probability of performing the intervention before the time of failure $P(t_i \leq T_f)$. Therefore, the probability of early detection of damage may be mathematically described as (Soliman et al., 2013):

$$\begin{aligned} P_{DBF}^{\rho}(t_1, \cdots, t_{|\rho|}) &= \textstyle\sum_{j=1}^{|\rho|}\left(\prod_{i=1}^{j}\mathcal{P}_D^{\rho_j}(t_j)\left(1 - \mathcal{P}_D^{\rho_{i-1}}(t_{i-1})\right)P\left(t_i \leq T_f\right)\right) \\ &= \textstyle\sum_{j=1}^{|\rho|}\left(\prod_{i=1}^{j}\mathcal{P}_D^{\rho_j}(t_j)\left(1 - \mathcal{P}_D^{\rho_{i-1}}(t_{i-1})\right)\left(1 - \int_{-\infty}^{t_i}\psi_{T_f}(\tau)d\tau\right)\right) \end{aligned} \tag{5.12}$$

Where for notation simplicity, it was assumed that $\mathcal{P}_D^{\rho_0}(t) \equiv 0$ and $t_0 = 0$. The expression of Eq. (5.12) is based on the Branch 2 of the event tree model depicted in Figure 5.5. The significance of this formulation is that the same approach may be easily extended to consider different time-dependent degradation mechanisms. Furthermore, results obtained with this same formulation have shown that high-quality inspection techniques do not have to be applied routinely during the whole service life of the structures. Schedule a limited number of inspection techniques in optimal times can be enough to achieve a high probability for the previous equation.

When the damage in a structure is not treated in time, maintenance costs often tend to increase drastically. That is, the early detection of corrosion damage is also quite meaningful from the viewpoint life-cycle cost. For this reason, it is necessary that such changes in the structure performance may be detected as soon as possible in order that the maintenance action is carried out promptly minimising the life-cycle costs (Moughty and Casas, 2017).

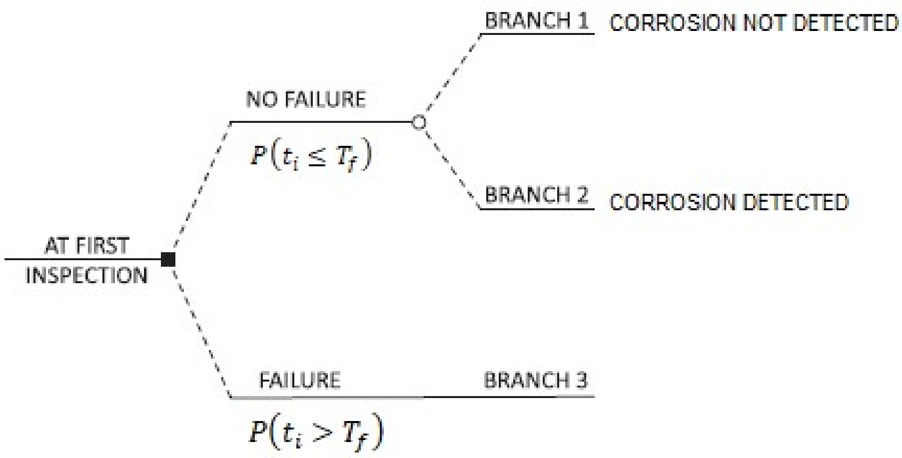


Figure 5.5 Event tree for the probability of early detection of damage (Soliman et al., 2013).

§*2.3 Optimal times for Inspection.* For each inspection sequence $\rho \in S_{N_s}$ the aim of the optimisation is to find the time $\overline{t_i}$ for which the inspection technique ρ_i will be applied in order to maximise the total probability of the inspection sequence to detect the corrosion damage before failure. Therefore, the optimisation process may be formulated as follows:

$$PD^{\rho} \equiv P^{\rho}_{DBF}\left(\overline{t_1}, \cdots, \overline{t_{|\rho|}}\right) = \max\ P^{\rho}_{DBF}\left(t_1, \cdots, t_{|\rho|}\right) \qquad for\ \left(t_1, \cdots, t_{|\rho|}\right) \in [0, T_{SL}]^{|\rho|} \quad (5.13)$$

Under the constraints:

$$t_i - t_{i-1} \geq Y \quad and \quad \sum_{i=1}^{|\rho|} t_i - t_{i-1} \leq T_{SL} \quad (5.14)$$

Where $t_0 = 0$ and Y is fixed as the minimum time admissible between consecutive inspections. The continuity of P^{θ}_{D} by using the extended definition in Eq. (5.10) is now relevant to the study. So, this discontinuity together with $\psi_{T_f}(t)$ enables that the maximum in Eq. (5.13) is attained by the Weierstrass' Theorem. This theorem shows that the continuous real-valued functions on a compact interval may be uniformly approximated as close as possible by polynomial functions (Young, 2006). Likewise, it should be noted that the optimal times $\overline{t_i}$ may not be unique. Therefore, this step associates a sequence of times $T^{\rho}\left(\overline{t_1}, \cdots, \overline{t_{|\rho|}}\right)$ to each inspection sequence ρ, where the sequence has the maximum probability of damage detection before failure is equal to PD^{ρ}.

5.2.2.3 Step 3: Multi-objective optimisation via Frontier Analysis

§*3.1 Cost of Inspection Sequences.* It is well known that both the design costs and the construction costs of a construction system are subject to rigorous care and attention. However, less than 15% of the total cost of a construction system is produced during the design and construction stage, while the broader phase of the life cycle, meaning the operations stage, constitutes approximately 60% of the total cost (Akcamete et al., 2010). Whether a building with a service life of 50 years is considered, approximately between 75 and 80% of the total costs are presented during the use and maintenance stage (Madureira et al., 2017). The considerations established on the previous steps allow for the modelling of the capabilities of the inspection techniques for detecting the damage properly during an intervention. Nonetheless, a higher quality of inspection implies a higher operating cost of maintenance activities.

For an optimal maintenance management planning, it is necessary to select the most suitable inspection technique for each stage of the inspection schedule. Furthermore, the optimal time between inspections that results in a lower expected cost should be considered under the combination of inspection techniques of different quality (Sheils et al., 2010). The total inspection cost of the maintenance planning is obtained from the individual cost of each inspection technique considered for the sequence. Moreover, the net discount rate that allows obtaining the Net Present Value of money for future investments during the service life of the structure is considered. In this sense, the total inspection cost is directly related to the detectability of the inspection technique applied and the number of interventions performed during the service life of the structure (Mori and Ellingwood, 1994). Then, for a given inspection sequence $\rho \in S_{N_s}$, the cost of the sequence at the intervention time is given by:

$$C^{\rho}\left(t_1, \cdots, t_{|\rho|}\right) = C_0 \sum_{i=1}^{|\rho|} \frac{\alpha_{insp}^{\rho_i}}{(1+r)^{t_i}} (1 - \eta_{min}^{\rho_i})^{20} \tag{5.15}$$

Where $\alpha_{insp}^{\rho_i}$ is the cost of the inspection technique apply in the sequence ρ associated to the technique $\theta \in \Theta$, which is assumed as a fraction of the initial construction cost C_0, and r is the annual discount rate of money.

It should be noted that the approach of this research seeks to find the optimal inspection times to carry out maintenance planning from a preventive perspective. That is, performing the inspection actions before attaining the failure in the structure. Thus, preventive maintenance aims to decrease or interrupt the degradation of the structure to reduce the number of major repairs, structural rehabilitation or the replacement of damaged elements. Through the perspective of preventive maintenance, it is also feasible to reduce the expected repair costs that could be carried out after the structure has reached the critical damage.

§3.2 Optimal Times considering the Early Damage Detection versus Inspection Costs. The optimal times for an inspection sequence ρ may be formulated as the time T^{ρ} by which a trade-off between the maximum probability of detection P_{DBF}^{ρ} and a minimum of cost C^{ρ} is reached. If the time interval is prolonged, a decrease in inspection cost can be expected. However, if once the damage is detected, it has a magnitude that exceeds the repair limit, the cost of the intervention could be increased widely and jeopardise the structural safety, besides an increment of the life-cycle cost (J. a. Mullard and Stewart, 2009). Thus, the main objective of a maintenance policy is to avoid failure occurrence at the lowest cost (Dieulle et al., 2003). The optimisation problem may be modelled as a vector value for the constrained maximisation by:

$$O_{\rho}\left(\overline{t_1}, \cdots, \overline{t_{|\rho|}}\right) = \max \left\{P_{DBF}^{\rho}\left(t_1, \cdots, t_{|\rho|}\right), -C^{\rho}\left(t_1, \cdots, t_{|\rho|}\right)\right\} \quad for \quad \left(t_1, \cdots, t_{|\rho|}\right) \in [0, T_{SL}]^{|\rho|} \tag{5.16}$$

Under the same constraints imposed in Eq. (5.14). The above optimisation problem is usually solved by gradient-free optimisation methods as genetic algorithms (GA) or non-dominated sorting in genetic algorithms (NSGA-III). The algorithms provide a Pareto optimal set of solutions which are optimum trade-offs between the two objectives. Nevertheless, this approach does not consider the resources needed for the implementation of the inspections or indirect administrative costs, neither give a relative ordered ranking which allows determining which are the best sequences. Therefore, additional analysis is proposed below based on an efficiency analysis applied to the previous optimisation problem.

§3.3 Other indirect cost considered for the analysis. There are several other costs associated with the realisation of an inspection that many times can be associated with other variables which correlate well with such costs, which a priori are difficult to estimate. Herein in this analysis, two variables which are sought to be minimised are considered: (a) the number of different inspection techniques

in the sequence ρ denoted as N^{ρ}, and (b) the "window" of inspection of the sequence ρ denoted as $W^{\rho} = T^{\rho}_{|\rho|} - T^{\rho}_{1}$, i.e. the time frame between the optimal time for the first inspection up to the optimal time of the last inspection.

These two variables are thoroughly relevant for the maintenance planning since their tackle directly the interests of companies, such as the competitiveness in the construction business. In other words, maintenance managers will always seek to use the lowest amount of resources that allow them to have the same optimum result for the inspection planning. Likewise, if the inspection planning requires a lower number of inspections and a lower window of inspection times (W^{ρ}), it will contribute to reducing the environmental impact associated with the infrastructure systems. Maintenance of RC structures usually involves the use of concrete for the repair tasks. The use of concrete as a construction material represents a significant global environmental impact since the production of cement is liable for 7% of the total carbon dioxide emissions worldwide (Dong, 2018). Hence, there are significant opportunities regarding global sustainability challenges for the reduction of environmental impacts associated with concrete infrastructure repair, rehabilitation, and use (Lepech et al., 2014).

§3.4 Frontier Analysis via SFA and MEA. To overcome some limitations of *§3.2,* benchmarking techniques are implemented to compare the (technical) efficiency of the sequences, considering inputs/resources and outputs/results. As has been mentioned at the beginning, the techniques that are considered for the efficiency analysis are variations of the Stochastic Frontier Analysis (SFA) and the Data Envelopment Analysis (DEA), which are the most dominant method in the literature. For a single output, the SFA is a technique that uses regression analysis to estimate a conventional cost function. For that purpose, the efficiency of a process is measured using the residuals from the estimated equation.

Afterwards, the error term is divided into a stochastic error term and a systematic inefficiency term. The most common technique in Frontier Analysis for measuring the efficiency is the so-called Data Envelopment Analysis (DEA), introduced with linear programming versions of the model by (Charnes et al., 1978). The DEA approach has been widely investigated and applied to many fields and industrial problems (Mahadevan, 2002; Chapelle and Plane, 2005; Ramli et al., 2013). In DEA, it is possible to apply radial contractions of the inputs and undesirable outputs and/or apply radial expansions of the desired outputs. However, to further assess whether the financial crisis led to changes in efficiency patterns it is useful to understand also which variables were used inefficiently.

Therefore, in this research is considered other non-parametric deterministic method for measuring efficiency, namely a model based on the Multidirectional Efficiency Analysis (MEA), proposed by Bogetoft and Hougaard (Bogetoft and Hougaard, 1998). In contrast to DEA, the input reduction and output expansion benchmarks in the MEA approach are selected proportional to the potential improvements in efficiency identified, while is considered the improvement potential separately in each input and output variable. Thus, in addition to efficiency levels, MEA allows investigating changes in efficiency patterns. A brief description regarding the mathematical derivation of the MEA model used for the efficiency analysis of the inspection planning is included below.

Firstly, let consider that $[m]$ denotes the set $\{1, \dots, m\}$. From the previous steps, to any given sequence $\rho \in S_{N_s}$ it is possible to associate $J \in \mathbb{N}$ outputs $y_j(\rho)$, $j \in |J|$ and $I \in \mathbb{N}$ inputs $x_i(\rho)$, $i \in |I|$. Some of the input variable may be discretionary (i.e. their values can be changed) but others may be non-discretionary (i.e. they are fixed values). From now on, the discretionary variables are represented by the first indices from 1 to $d \in [1, I]$. So, $x(\rho)$ is the vector of all the inputs and $y(\rho)$ is the vector of all the outputs. DEA/MEA model may change with respect to a chosen set of complementary variables. Then, the variable returns to scale (VRS) model is considered for the efficiency measurement (Bogetoft and Otto, 2011), by defining the set:

$$\Lambda^N = \left\{ \lambda \in \mathbb{R}^{\mathbb{N}} : \sum_{n=1}^{N} \lambda_n = 1 \wedge \lambda_n \geq 0 \right\} \tag{5.17}$$

Where N is the number of sequences under the study and λ denotes a vector of intensity variables that form linear combinations of observed inputs and outputs. Alternatives definitions of Λ^N allow other models, not relevant in this study, which are known as Decreasing Returns to Scale (DRS), Free Disposability Hull (FDH), Free Replicability Hull (FRH), Increasing Returns to Scale (IRS), Constant Returns to Scale (CRS) or Returns To Scale (RTS). Then, the MEA score is found by solving the following linear programming optimisation problems:

Problem $P_m^\alpha(\bar{\rho})$:	Problem $P_j^\beta(\bar{\rho})$:	
$\min \alpha_m(\bar{\rho}) s.t.$	$\max \beta_j(\bar{\rho}) s.t.$	
$\sum_\rho \lambda_\rho x_m(\rho) \leq \alpha_m(\bar{\rho})$	$\sum_\rho \lambda_\rho x_i(\rho) \leq x_i(\bar{\rho}), i \in [I]$	Eq. (5.18a) & Eq. (5.18b)
$\sum_\rho \lambda_\rho x_i(\rho) \leq x_i(\bar{\rho}), i \in [I], i \neq m$	$\sum_\rho \lambda_\rho y_s(\rho) \leq \beta_j(\bar{\rho}), S \in [J]$	
$\sum_\rho \lambda_\rho y_l(\rho) \leq y_l(\bar{\rho}), l \in [J]$	$\sum_\rho \lambda_\rho y_l(\rho) \leq y_l(\bar{\rho}), l \in [J], l \neq j$	

Here, $\lambda \in \Lambda^n, \alpha_m^*(\rho)$ and $\beta_j^*(\rho)$ are the optimal solutions to the problems $P_m^\alpha(\bar{\rho})$ and $P_j^\beta(\bar{\rho})$ respectively. The next step of the MEA model determines the global solution for the efficiency analysis by solving the following linear programming:

$$\begin{aligned}
&\text{Problem } P^\gamma(\alpha, \beta, \bar{\rho}):\\
&\max \gamma(\bar{\rho}) s.t.\\
&\sum_\rho \lambda_\rho x_i(\rho) \leq x_i(\bar{\rho}) - \gamma(\bar{\rho})\big(x_i(\bar{\rho}) - \alpha_i^*(\bar{\rho})\big), i \in [I]\\
&\sum_\rho \lambda_n x_i(\rho) \leq x_i(\bar{\rho}), i \in [I] \setminus \{m\}\\
&\sum_\rho \lambda_\rho y_l(\rho) \geq y_l(\bar{\rho}) + \gamma(\bar{\rho})(\beta_l^*(\bar{\rho}) - y_l(\bar{\rho})), l \in [J]
\end{aligned} \tag{5.19}$$

Where $P^\gamma(\alpha, \beta, \bar{\rho})$ represents the global solutions for the problem given by Eq. (5.18a) and Eq. (5.18b) in the efficiency analysis. The MEA score is obtained by the directional contribution of each input and each output variable. Therefore, the MEA score of a sequence ρ is obtained from the expression:

$$MEA(\rho) = \frac{\dfrac{1}{\gamma^*(\rho)} - \dfrac{1}{D}\sum_{i=1}^{D} \dfrac{x_i(n) - \alpha_i^*(\rho)}{x_i(\rho)}}{\dfrac{1}{\gamma^*(\rho)} + \dfrac{1}{J}\sum_{j=1}^{J} \dfrac{\beta_j^*(\rho) - y_j(\rho)}{y_j(\rho)}} \in [0,1] \tag{5.20}$$

A simple but quite raw rule to decide which are the inputs versus outputs is to consider as inputs, the variables to minimise, and as outputs, the variables to maximise. In this case, it is assumed as inputs the variables N^ρ (as non-discriminatory) and W^ρ (as discriminatory), and as outputs the variables and $CC^{\rho} = -C^{\rho} + max_{\rho}C^{\rho}$, although many others may be added by the decision makers. Therefore, the best inspection sequences are the ones with higher MEA score, since it is a relative ranking between zero and one.

5.2.3 Application of the Methodology

A numerical example regarding the methodology proposed is presented in this section. The main objective is to illustrate the importance of the efficiency analysis in the inspection planning. A hypothetical case study is supposed for the maintenance management of a RC structure with a service life of 50 years ($T_{SL} = 50$). To simplify the study, the cost analysis is performed under the consideration of referential cost. These costs are related to an initial total cost of construction equal to 1000-unit cost ($C_0 = 1000$). Likewise, the net discount rate of money r for Paraguay is assumed as 5.5% (CEPAL, 2017).

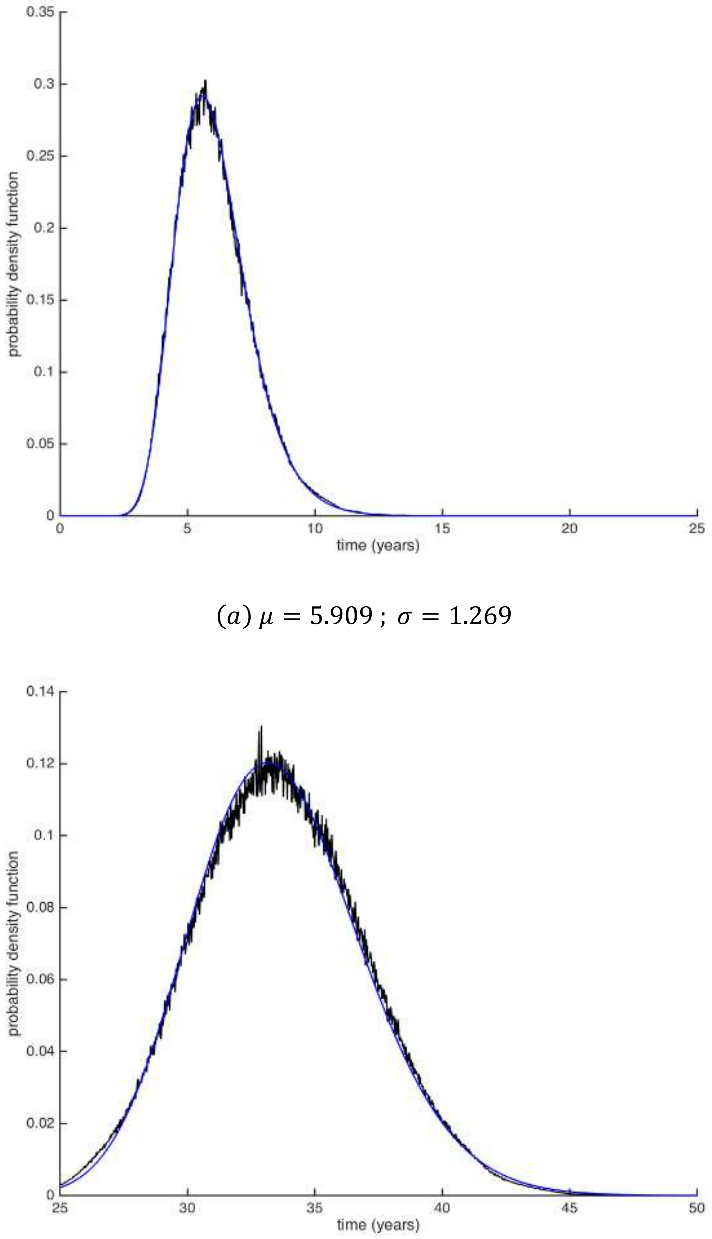
In the first step of the methodology, mathematical formulation to determine the time of corrosion initiation and the time of failure is addressed. Considering the uncertainties of parameters presented in the Eq. (5.4) and Eq. (5.6), the time of corrosion onset and the probability of failure must be addressed stochastically. Thus, the probability density function for both time (T_{icorr}, T_f) can be obtained from the Monte Carlo simulation (MCS) method, where the randomness of the parameters is assumed as it is shown in Table 5.1 (CPH, 2008; Kim and Frangopol, 2011; Cheung et al., 2012). For that, a simulation with a sample size of 50000 is performed. The outcome was obtained by subdividing the sampling region into 1000 evenly spaced regions where the number of occurrences (failure) is computed for each region.

Table 5.1 Parameters for the generation of T_{icorr} and T_f.

Variable(s)	Units	Distribution/Value(s)		
α_t; β_t	*Coefficient*	1800.0, -1.7		
C_{air}, C_{env}	*Coefficient*	1.0, 1.0		
b	*mm*	$LogN,$	$\mu = 20.0,$	$\sigma = 0.200$
d_0	*cm*	$LogN,$	$\mu = 1.6,$	$\sigma = 0.020$
η_{cr}	*Percentage*	0.25		
f_{ck}	*N/mm²*	$LogN,$	$\mu = 20.0,$	$\sigma = 3.380$
V_{corr}	*cm/year*	$LogN,$	$\mu = 0.0075,$	$\sigma = 0.0015$

Establishing the most suitable distribution for each variable involved in the analysis requires a comprehensive study based on real data. Nevertheless, the normal distribution is frequently used in civil engineering studies for random variables of real value with a continuous probability distribution whose distribution is unknown (Xu et al., 2015). For the random variables of Table 5.1, log-normal distribution ($LogN$) was assumed. After applying the MCS, the probability distribution associated with the corrosion initiation time and failure time is shown in Figure 5.6. For the assumptions of Table 5.1, the best fitting after the MCS was a lognormal distribution which is represented in the figure with a blue line. The probabilistic values ($\mu; \sigma$) shown in Figure 5.6 correspond to the geometric parameters of such lognormal distribution.

The second step involves obtaining the optimal times for inspection. For this analysis are considered the times calculated in the previous step together with some features of the available inspection techniques. The capability to detect the corrosion damage that an inspection technique has is a meaningful variable for this point. As has been established in Section 5.2.2.1, this capability is expressed as a detectability function that depends on the damage degree in the structure and of the quality of the applied inspection technique, which in turn affects the total cost of the inspection planning.



$(a)\ \mu = 5.909\ ;\ \sigma = 1.269$

$(b)\ \mu = 33.471\ ;\ \sigma = 1.105$

Figure 5.6 Probability Density Function for (a) T_{icorr} and (b) T_f.

Three types of inspection techniques were considered in this application example of the proposed numerical model for the maintenance planning. These methods have been described in Chapter 3 and include the following techniques for detecting corrosion: Linear Polarisation Resistance (LPR), Half-Cell Potential, and Resistivity measurement. As seen in (Soliman et al., 2013), the parameters that define the detectability of these techniques depend on several factors such as the type of structure, the location to be inspected, the technician's experience and environmental conditions. In practice, define these parameters is a complex task since it requires extensive experimental research that goes beyond the purpose of this thesis. Therefore, the values for the parameters that define the capabilities of each inspection technique have been assumed in this study considering the findings of other research (Millard and DTI DME Consortium, 2000; Song and Saraswathy, 2007; Hornbostel et al., 2013; Verma et al., 2014; Belda Revert et al., 2018). Table 5.2 shows the parameters assumed for each inspection technique.

Table 5.2 Parameters estimated for the inspection techniques.

Inspection Technique θ	$\eta_{0.5}$	σ	η_{max}	α_{insp}
Half-Cell Potential (A)	0.15	0.015	0.78	0.003
LPR (B)	0.18	0.030	1.00	0.010
Resistivity (C)	0.22	0.015	1.00	0.004

For simplicity notation in the calculus and presentation of results, these inspection techniques have been considered with the nomination of techniques *A*, *B* and *C*. After the mathematical formulation described in Section 5.2.2.1 has been computed and solved using the MATLAB software version R2015a, Figure 5.7 shows the curve of cumulative distribution of the detectability function for each inspection technique. This curve represents the probability of detection concerning the corrosion damage degree of the structure. In this case, for the values assumed in Table 5.2, the technique of lower quality (higher $\eta_{0.5}$) is the resistivity measurement technique (C), while the half-cell potential (A) is the one with a greater detectability (lower $\eta_{0.5}$). In Figure 5.8 it is possible to interpret the behaviour of the detectability function throughout the service life of the structure. In this way, the capability of each technique to detect the corrosion damage before failure can be visualised.

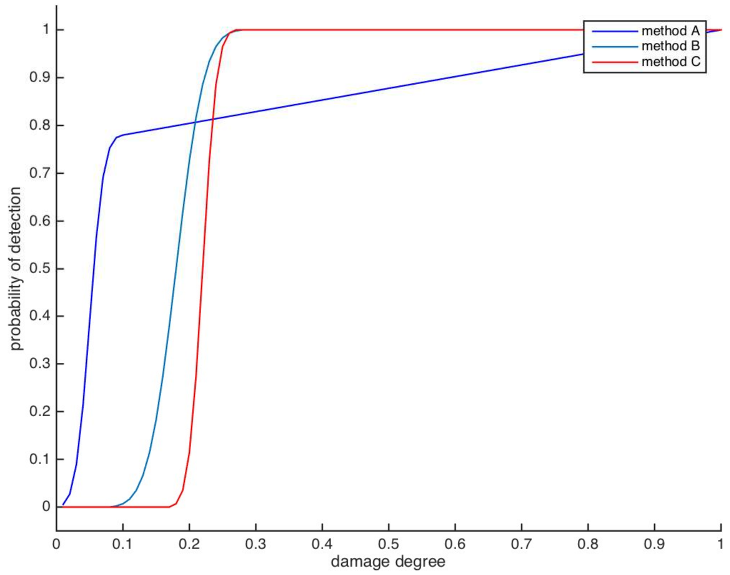


Figure 5.7 Cumulative Density Function for the Detectability regarding the damage degree

Another analysis that has been carried out within the model is the influence of the boundary points adopted in the determination of the detectability of the inspection technique. As it has seen in the previous section, an extension of the original detectability model has been formulated to reduce the discontinuity of its probability function. Figure 5.9 shows the maximum and minimum damage degree ($\eta_{min}^{\theta}, \eta_{max}^{\theta}$) of each inspection technique concerning the damage degree η throughout the service life of the structure.

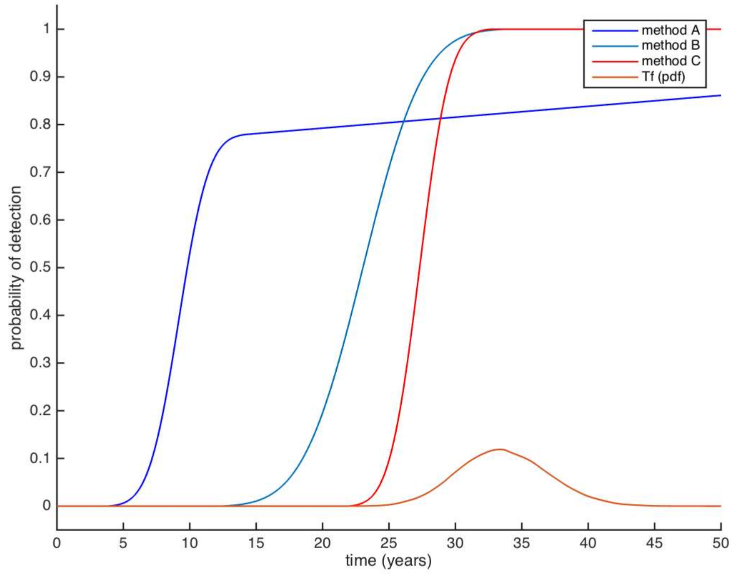


Figure 5.8 Cumulative Density Function for the Detectability over the service life time.

For the case shown in this section, the resistivity measurement technique (C) covers a shorter period in the service life of the structure for which its detection capacity can be assessed. In contrast, for the LPR technique (B), this time gap is more significant concerning the resistivity measurement, which is an advantage from the point of view of the damage detection. On the other hand, the half-cell potential technique (A) can only be evaluated (in the illustrative example) for its application during the first 10-15 years of the lifespan of the structure. However, these results are highly influenced by the corrosion rate assumed in Table 5.1 which defines the degradation speed of the structure.

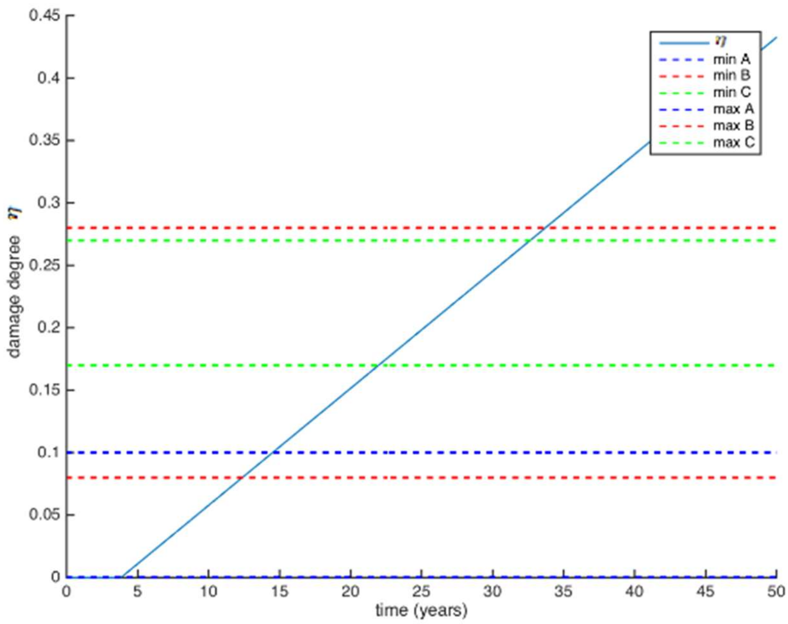


Figure 5.9 Boundary points of damage degree for each inspection techniques.

The last step of the method for the inspection planning involves the multi-objective optimisation of the problem through an efficiency analysis. This process results in the optimal times for the interventions be applied and the most appropriate technique for each time. The outcomes of this process are presented in Table 5.3 and will be addressed in Section 5.2.4 of this chapter. The table shows the optimal sequence for the inspection technique (*InspSeq*); the optimal times of inspections (t_1, t_2, t_3); the required number of techniques N^{ρ}; the total time of the planning W^{ρ}; the maximum probability of damage detection before failure PD^{ρ}; the total cost of the inspection planning C^{ρ}; and the results of the $SFA(\rho)$ and $MEA(\rho)$ analysis, where the output treated in $SFA(\rho)$ is PD^{ρ}. For these last two parameters, the closer the value is to the unit, the more efficient is the inspection plan.

The result of Table 5.3 represents the main purpose of this study. As can be seen, unlike other studies where only optimisation techniques are applied, the result of the efficiency analysis allows knowing clearly which is the best strategy to be implemented for the inspection planning. A more detailed discussion is addressed in the next section regarding this obtained result.

Table 5.3 Outcomes of the Efficiency Analysis for the Optimal Inspection Planning.

InspSeq	t_1	t_2	t_3	N^{ρ}	W^{ρ}	PD^{ρ}	C^{ρ}	$SFA(\rho)$	$MEA(\rho)$
C	29.944	0.000	0.000	1	0.000	0.807	0.024	0.844	1.000
B	28.409	0.000	0.000	1	0.000	0.886	0.289	0.913	1.000
CC	29.498	30.499	0.000	1	1.000	0.959	0.048	0.971	1.000
BB	27.903	28.903	0.000	1	1.000	0.986	0.578	0.998	1.000
CCC	29.077	30.077	31.077	1	2.000	0.990	0.072	0.991	1.000
BBB	27.406	28.406	29.406	1	2.000	0.998	0.867	1.000	1.000
A	24.811	0.000	0.000	1	0.000	0.802	0.795	0.839	0.251
AA	24.204	25.204	0.000	1	1.000	0.961	1.599	0.973	0.244
BCB	28.253	29.253	30.253	2	2.000	0.996	0.578	0.999	0.051
BBC	27.904	28.904	29.945	2	2.042	0.997	0.602	1.000	0.050
BBA	27.579	28.579	29.576	2	2.000	0.996	1.203	0.999	0.049
BAB	27.161	28.161	29.161	2	2.000	0.996	1.250	0.999	0.049
CAB	27.829	28.831	29.841	3	2.012	0.985	0.935	0.989	0.041
ABB	24.808	27.904	28.904	2	4.096	0.997	1.372	0.979	0.024
ABC	24.806	28.410	29.945	3	5.139	0.996	1.108	0.968	0.021

5.2.4 Discussion of results

The maintenance management of structures is a paramount issue within engineering whose main objective is to preserve the durability and safety of infrastructure. The main challenge in this field includes the correct interpretation of the deterioration mechanism of the structure, which is highly influenced by uncertainty. There are several degradation mechanisms that jeopardise the durability of structures during its service life. This research attempt to find the best strategy to intervene reinforced concrete structures under corrosion risk. The corrosion phenomenon may occur through several mechanisms and processes. Nonetheless, the study herein presented has focused specifically on the carbonation-induced corrosion.

Optimal inspections planning is achieved through a methodology that includes three main steps: the determination of corrosion onset time and failure time; the probability of damage detection before the failure occurs; and the formulation of the optimal inspections sequence in the structure through an efficiency analysis. In the first step, the degradation mechanism of the structure (corrosion by carbonation) is included in the context of the study by means of the mathematical formulation of the corrosion initiation time. The existing literature is comprehensive concerning the study of the corrosion initiation time in carbonate structures. However, this research takes into account a widely referenced study (Yoon et al., 2007), which establishes an early onset of corrosion concerning other initial studies, where the corrosion initiation was considered when the carbonated front reached the surface of the rebar.

Figure 5.6 depicts the probability distribution for the time of corrosion initiation as well as the time of corrosion failure. The time of corrosion initiation depends mainly on the concrete cover thickness and the quality of the concrete, in this case, represented by its characteristic compressive strength. The higher these two parameters, the longer the time for the corrosion initiation by carbonation in the reinforcement. In the context of carbonation-induced degradation, environmental exposure is another determining factor for the corrosion onset. Environmental factors such as relative humidity, temperature and the carbon dioxide concentration affect the degradation rate of these structures. The variables considered for the calculation of the time of corrosion initiation by the Häkkinen method correspond to a concrete structure with Portland cement (α_t = 1800; β_t= -1.7), a structural element exposed to weathering ($C_{env} = 1$), and a concrete element with an air content less than 4.5% ($C_{air} = 1$). So, for these values and other variables referring to the concrete, the corrosion onset time in the structure is near to 6 years of lifespan for the case considered.

Time of failure is considered as the limit state function where the critical damage degree is such that the safety margin is reduced significantly, i.e., a cross-sectional loss higher than 25% of its original section. Corrosion degradation induces the loss of the cross-sectional of the reinforcement in the concrete structure over time. This loss significantly affects the structural capacity due to the decrease of adherence on the steel/concrete interface. Likewise, if corrosion damage is excessive, the degradation may lead to cracking and spalling of the cover that leaves the reinforcement completely vulnerable. Therefore, failure in the structure does not imply a situation of collapse essentially but only that the structural safety could be endangered. All the parameters considered in the limit state function affect the failure time. However, the corrosion rate is the most influential since the higher the corrosion rate; the lower is the time elapsed to attain the failure in the structure. For this study, the corrosion rate assumed is relatively low. Thus, an alternative result for a higher value of corrosion rate is presented later concerning those results in Section 5.2.3.

A critical factor in the maintenance of structures is the on-time detection of corrosion of the reinforcement. Once corrosion begins, the durability of the structure is threatened by the critical damage degree to reach the failure. From that point, an intervention of the structure could lead to a high investment that would affect the total life-cycle cost of the infrastructure. Under this approach, one of the key points of the methodology herein applied is the early detection of damage. This goal is achieved through the analysis of the probability of damage detection before the failure occurrence that is directly related to the damage degree and the quality of the applied inspection technique.

Once the degradation times of the structure are known, the maintenance model addressed here analyses the capabilities of the inspection techniques to detect the corrosion damage. It should be noted that the detectability function is influenced by the mean and standard deviation of damage intensity that an inspection technique can detect. Therefore, this function must be updated according to a dataset generated after each inspection. Unlike other studies, the standard deviation for the inspection techniques has been considered variable for each technique. That is, the detection capacity varies not only according to the mean damage degree $\eta_{0.5}$ but also according to the amplitude of such parameter. Once the actual parameters that establish the detectability of the inspection technique are known, then any technique available in the construction market can be introduced into the maintenance model to detect corrosion.

In Figure 5.7 the probability of damage detection for three inspection techniques is shown. From the cumulative curve for the half-cell potential technique, it can be seen that it is the one that has, *a priori*, higher quality since it needs a lower corrosion degree in the structure to detect the damage. By contrast, the resistivity measurement technique is shown as the worst quality ones. Nevertheless, the quality of an inspection technique is not established only by a single parameter but are multiple the parameters that finally determine its detectability. Figure 5.8 depicts the detectability of the inspection techniques over the service life of the structure, where the capability that each technique has to achieve the maximum probability of damage detection before failure can be visualised. Also, it can be seen that the LPR and resistivity measurement techniques, despite not having high initial detectability, over time they acquire a greater detectability than the half-cell potential technique. This controversy of criteria is frequent in real cases whereby is challenging to take an accurate decision beforehand regarding the best inspection technique to be employed.

In a decision-making model, the controversy among criteria does not allow to know in advance which option/solution is the most advisable. For instance, from a logical point of view, it could be expected that a high-quality inspection technique registers a greater detectability. On the other hand, considering that the detectability is not associated solely with the quality of the technique, the intervention in a structure may exceed the budget costs if only high-quality techniques are applied. For this reason, it is necessary to perform a detailed analysis that allows establishing a cost-effectiveness compensation of each proposed inspection technique to be applied throughout the service life of a structure.

The last step of the proposed maintenance model establishes an efficiency analysis that quantitatively guarantees the cost/efficiency compensation of the inspection planning. This analysis allows, besides to optimising the resources necessary to fulfil the objective functions, to determine indeed which is the best option among a set of optimal solutions. Two methods of analysis were selected, namely SFA and MEA. The main difference between the two methods is that the MEA method considers a broader analysis allowing the assessment of several inputs and outputs within the process. Table 5.3 has shown the results for all possible combinations of inspections techniques that give the optimal sequence for the inspection planning. These results are ordered according to the most efficient sequence. On the other hand, Table 5.4 shows a summary of the best sequences of inspections that satisfy the four last variables of the preceding table.

Table 5.4 Best inspection sequences for each variable considered.

Variable	First Position	Second Position	Third Position
$\boldsymbol{PD^{\rho}}$	BBB (0.998)	BBC, ABB (0.997)	ABC, BCB (0.996)
$\boldsymbol{C^{\rho}}$	C (0.024)	CC (0.048)	CCC (0.072)
$\boldsymbol{SFA(\rho)}$	BB, BBC (1.000)	BCB, BBA, BAB (0.999)	BB (0.998)
$\boldsymbol{MEA(\rho)}$	C, B, CC, BB, CCC, BBB (1.000)	A (0.251)	AA (0.244)

This table allows a broader understanding of the importance of the efficiency analysis for the context of the study. For instance, for the ABC sequence is obtained one of the best results from the viewpoint of the probability of early damage detection. Nevertheless, after an efficiency analysis, the same sequence shows one of the worst results. Moreover, the inspection planning can be optimal and effective without to be required performing the intervention several times and without being necessary rely on a wide variety of inspection techniques to planning the maintenance of the structure optimally. This approach is highly significant not only for the competitiveness of the company in charge of maintenance but also from the environmental and sustainable perspective of a maintenance strategy.

Furthermore, repeatedly intervening in a structure implies an extra consumption of human and material resources. The environmental impact associated with concrete infrastructures is mainly given by the use of cement during repairs. Therefore, carrying out the minimum possible interventions at the right time allows more sustainable maintenance with a lower environmental impact. Finally, the same decision-making model proposed in this research was applied again for the same conditions of the numerical example but increasing the corrosion rate V_{corr} assumed in Table

5.1. Then, through a new numerical simulation and a corrosion rate estimated at 0.015 cm/year, the sequence and optimal times are shown in Table 5.5.

Table 5.5 Optimal inspection planning for $V_{corr} = 0.015\ cm/year$.

InspSeq	t_1	t_2	t_3	N^{ρ}	W^{ρ}	PD^{ρ}	C^{ρ}	$SFA(\rho)$	$MEA(\rho)$
CC	16.785	17.785	0.000	1	1.000	0.978	0.068	0.991	1.000
BBB	15.495	16.495	17.495	1	2.000	1.000	0.941	0.993	1.000
AAA	14.032	15.032	16.032	1	2.000	0.993	3.644	0.986	0.162
BCA	16.178	17.178	18.178	3	2.000	0.999	1.378	0.993	0.020
CAC	15.283	16.283	17.283	2	2.000	0.990	1.206	0.984	0.018
ACA	15.651	16.964	17.964	2	2.313	0.995	2.246	0.987	0.017

From the results of Table 5.5, it can be seen that the inspection times were reduced concerning the results of Table 5.3. This is an expected result since the increase in the corrosion rate leads to an acceleration regarding the time to attain the corrosion failure. However, it should be noted that the decrease in inspection times is not proportional to the increase in the parameter under consideration, i.e. the corrosion rate. This means that there is no linear relationship between the parameters considered by the decision-making model and the results obtained. Lastly, it should be mentioned that the results shown in Section 5.2.3 are merely referential and are highly influenced by the values assumed in Tables 5.1 and 5.2. Therefore, by resetting such values, it will be possible to obtain an adequate inspection planning for each deterioration condition and each exposure condition of a RC structure.

5.3 Dynamic Decision-Making Model

As mentioned throughout this chapter, performing structures' maintenance is meaningful to ensure their safety and durability. Nonetheless, the choice of the best maintenance strategy is usually established based on a set of criteria, i.e. safety, cost, available resources, accessibility of the structure, and so on. To properly address the study of maintenance management, it is important to establish decision-making methods based on multiple criteria analysis. A maintenance strategy should be oriented to reduce the amount and frequency of maintenance, improving maintenance operations, decreasing the complexity effect, reducing the skills required for maintenance, among others (Dhillon, 2002).

The choice of an option among a set of alternatives based on some criteria is what is considered as decision-making. This decision can be based on multiple criteria instead of a single one, where the main objective is to obtain a relative ranking of alternatives regarding a decision problem. The decision-making based on a simple criterion or requirement used in the past has allowed current investigation of highly complex decision problems that include a multitude of variables that are usually stochastic (Bhushan and Rai, 2004).

The maintenance of existing buildings has become meaningful since the cost of new construction has been increased in the last decades. The efficiency of the building's maintenance is highly relevant concerning its durability and functionality, which requires a precise method for planning the different intervention tasks involved (Mendes Silva and Falorca, 2009). Hence, the maintenance strategies should be suitable and cost-effective to allow the best allocation of budgets and minimise the deterioration of the building over its whole life cycle (Flores-Colen and de Brito, 2010). Appropriate knowledge of the degradation process and measurement techniques are essential to improve the durability of structures as well as the monitoring, evaluation and repair procedures. All these considerations should be an integral part of any durability strategy, incorporating the planning into the design phase (Schiessl, 1996).

The maintenance can be established in three large groups: the preventive, the corrective and the predictive. The main difference among them lies in the time at which the repair or maintenance task is implemented (Mobley et al., 2008). Preventive maintenance includes scheduled actions to reduce the probability of failure or an unacceptable level of degradation. It usually comprises the most significant proportion of the total maintenance effort. Corrective maintenance, instead, includes unscheduled actions that are carried out once the deficiencies are detected and whose purpose is to return the damaged element to a defined state. Predictive maintenance is associated with the continuous monitoring and processing of damages that allow diagnosing the condition of the structure during the service (Dhillon, 2002).

Maintenance must be carefully designed to be adapted to existing technical, geographical and personnel situations. An essential preventive maintenance program includes the periodic evaluation of critical elements to detect potential problems and immediately schedule maintenance activities that will prevent any severe degradation. Likewise, preventive maintenance must be designed to anticipate the need for corrective maintenance and prolong the service life of the structure. Furthermore, a poor maintenance or repair task often results in more damage to the structure (Mobley et al., 2008).

This research focuses its study on a preventive maintenance strategy. The advantages and shortcomings of a preventive maintenance strategy depend on the performance knowledge, decision criteria, economic and technical characteristics of each intervention technique, and the structured data (Flores-Colen and de Brito, 2010). Among the advantages of preventive maintenance, it is possible to identify the improvement of safety, the efficiency in the use of time and economic resources, and the cost/benefit optimisation during maintenance (Dhillon, 2002). Therefore, in building lifecycle management, the most advantageous strategy for the extension of its service life is the application of preventive maintenance throughout all the life cycle of the structure. In other words, preventive maintenance is the cheapest alternative and it is the only one that enhances the durability of materials and construction elements (Rodrigues et al., 2018).

Unlike the numerical model described in Section 5.2, herein is presented a new decision-making model that considers not only inspections but also repair activities through the Multi-Criteria Decision-Making (MCDM) approach. The MCDM process may comprise the solution of the problem referred as how to derive weights or rankings of importance for a set of alternatives/criteria according to their effect on the objective of the decision taken, see Fig. 5.10 (Bhushan and Rai, 2004). Thus, the MCDM problems may be classified into two main groups: multiple-attribute decision-making (MADM) and multiple objective decision-making (MODM). The MADM method is applied to decision problems with a limited number of predetermined alternatives and discrete preference ratings. The MODM aims to achieve the optimal goals by considering several interactions within the given constraints (Tzeng and Huang, 2011). Hence, in the management of maintenance strategies, the possible answers to a decision problem are finite and MADM is the category which must be chosen for the study (Sabaei et al., 2015).

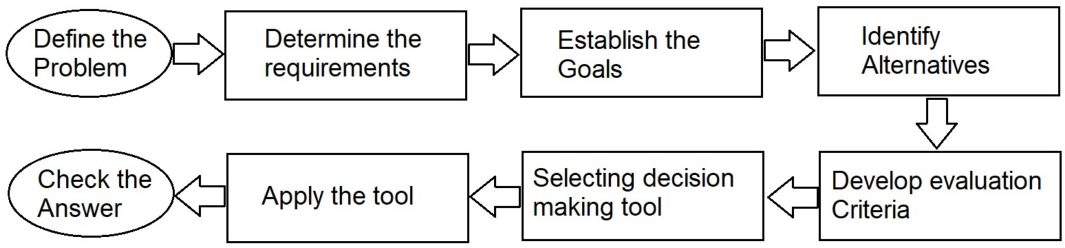


Figure 5.10 General process of a MCDM model (Sabaei et al., 2015).

Considering the complexity of classifying alternatives in multiple individual criteria, a common practice is to take a weighted average of the satisfaction of an alternative for the individual criteria, i.e. the importance of the individual criteria (Yager, 2018). This is the main approach of the well-know MADM method called the Analytic Hierarchy Process (AHP) model. However, as such analysis involves some subjectivity in the final results, this chapter proposes a new perspective for the traditional AHP model that includes a stochastic analysis for the weight assignments of criteria and alternatives.

Hence, the purpose of this study is to provide a useful tool to the decision maker so that they can schedule the best maintenance strategy in RC structures subject to corrosion degradation, including both inspection and repair. The support tool comprises a dynamic decision model based on the AHP method. Some basic notions of the traditional AHP method and the mathematical formulation of the dynamic decision-making model for maintenance planning are presented below.

5.3.1 Analytic Hierarchy Process

The Analytic Hierarchy Process (AHP) was developed by Thomas L. Saaty and since then it has been widely studied and applied for decision-making in several fields of science (Saaty, 1980). AHP is a general theory of measurement that is used to derive ratio scales from the discrete paired comparison. It consists in a nonlinear framework for developing both deductive and inductive thinking by taking several factors in considerations without the use of syllogism (Saaty, 1987). The AHP method is a convenient tool to deal with complex decisions about the most general structures encountered in real life that involve dependency and feedback analysed in the context of costs, risks, benefits and opportunities. Thus, a quality of the AHP method is that it produces results that consider the external risks concerning the decision and not only the values of the decision maker (Saaty, 2008b).

In essence, the AHP method gives as an outcome a plan of preferences and alternatives based on the level of importance obtained for the different criteria where the comparative judgements of experts are taken into account (Mu and Pereyra-Rojas, 2015). Some studies in construction and engineering maintenance are found in the literature concerning the application of the AHP method to solve decision problems (Lin et al., 2008; Wang et al., 2008; Reza et al., 2011; Chua et al., 2015). Its success lies in its easy implementation and understanding as well as in its almost universal adoption as a new paradigm for decision-making. Furthermore, it has been found to be a methodology capable of determining results that are in agreement with general perceptions and expectations (Bhushan and Rai, 2004). Therefore, it has been proven that AHP is a useful tool to solve complex decision problems on which controversy can be expected among the different judgements of experts regarding which one is the best alternative to achieve a specific objective.

The simplest way to structure a decision-making problem is through a hierarchy of three levels as is depicted in Figure 5.11. The three levels comprise the goals, criteria, and alternatives. The organisation of the problem in hierarchies allow a better understanding regarding the decision that must be achieved, the criteria that will be used and the alternatives that will be assessed. The participation of experts is crucial at this stage of the method since they ensure that all criteria and alternatives are considered properly (Mu and Pereyra-Rojas, 2015). Thus, once the problem is structured, the AHP method is simple to apply to solve decision problems (Saaty and Vargas, 2012). The existence of a relation of hierarchical dependence between elements of the structure is marked by the line that connects them (Brunelli, 2015).

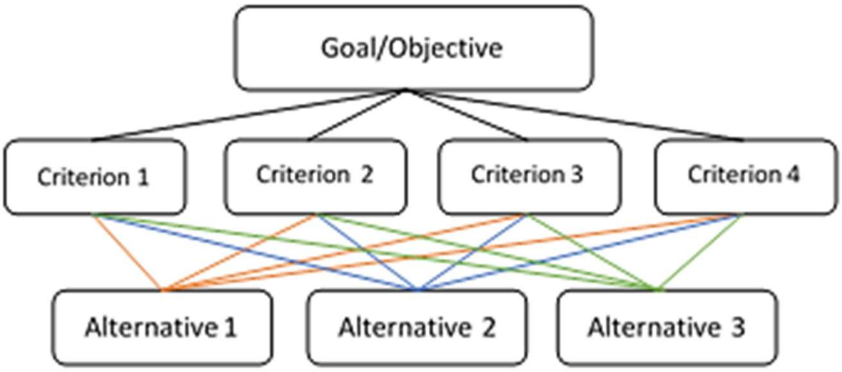


Figure 5.11 Hierarchical structure for decision-making (Saaty and Vargas, 2012).

Once the problem is structured, the method continues with the construction of the pairwise comparison matrices to solve the decision problem. The best option to concentrate the weights judgement of each criterion is to take a couple of the elements and compare them concerning a simple property or attribute. In this way, the comparison by pairs in combination with the hierarchical structure is worthwhile for deriving measurements. A pairwise comparison matrix of criteria is built, and the weights assignment of each criterion is given by experts or decision makers. These weights will determine which criterion has the highest priority or importance in the decision problem to achieve the goal. The unification of the multidimensionality of the problem in a unified dimension from the perspective of the final result is given through the use of a ratio scale of comparisons (Bhushan and Rai, 2004).

Some linguistic expressions have been proposed in the AHP to help the decision maker assign values to the judgments of criteria/alternatives. That is, the decision maker can express opinions in pairs through linguistic terms that are then associated with real numbers (Brunelli, 2015). The importance scale is represented by the numbers that establish the verbal judgements as is shown in Table 5.6 (Ferdous et al., 2016). Furthermore, a logical assumption is established regarding the reciprocals of all scaled ratios that are ≥ 1 in the transpose position of the pairwise comparison matrix (Saaty, 2008b). Subjective bias may be created using semantic labels. To compensate for such a handicap, it is convenient to analyse, compare and, eventually, modify the resultant weights (Caño et al., 2012).

Table 5.6 Fundamental scale for verbal judgements (Ferdous et al., 2016).

Importance Scale	Description of judgements
1	Equal importance
3	Moderate importance
5	Strong importance
7	Very strong importance
9	Extreme importance
2,4,6,8	Intermediate values

Considering that the numerical values (importance scale) are derived from the subjective preferences of the experts, it is impossible to avoid some inconsistencies in the final matrix of comparison. Therefore, the method calculates a consistency ratio (CR) from the relation between the consistency index (CI) of the comparison matrix and the consistency index of a random-like matrix (RI). A consistency ratio of 0.10 or less is permissible to validate the AHP analysis (Mu and Pereyra-Rojas, 2015). Therefore, to calculate the CR is necessary to calculate first the CI and RI. The CI can be obtained from Eq. (5.21) and the RI is a fixed index obtained from Table 5.7 which, depends on the number of elements (n) of the comparison matrix (Ferdous et al., 2016).

$$CI = \frac{\lambda_{max} - n}{n - 1} \tag{5.21}$$

$$CR = \frac{CI}{RI} \tag{5.22}$$

where λ_{max} is the eigenvalue and n is the number of comparisons. The eigenvector is a particular vector associated with a linear system of equation that is widely used for subjective assessments by researchers (Sabaei et al., 2015).

Table 5.7 Values for random index (Saaty, 2008b).

n	1	2	3	4	5	6	7	8	9	10
RI	0.00	0.00	0.52	0.89	1.11	1.25	1.35	1.40	1.45	1.49

Subsequently, the analysis continues with the calculation of the priority vector of the comparison matrix. The best-known method for estimating a priority vector is considering that this vector should be the principal eigenvector of the comparison matrix. Thus, the priority vector may be obtained by summing each row in the matrix and dividing each by the total sum of all the rows, or approximately by adding each row of the matrix and dividing by their total (Saaty, 2008a). The priority vector is important for the process since it influences the final ranking of importance of alternatives formulated for the decision problem.

The next stage comprises the derivation of local priorities for the alternatives in the lower level of the hierarchy. For this, once again is performed the pairwise comparison matrix but, in this case, between alternatives. However, each criterion is considered in the matrix. So, the set of alternatives are analysed for each criterion in the decision problem following a similar process as in the previous stage. Likewise, in this stage shall be verified the consistency of weights before to derive the priority vector on each matrix (Mu and Pereyra-Rojas, 2015).

Lastly, after having elaborated the comparison matrices for each level and after having calculated the priority vectors, it is then possible to establish the global priority of the elements of these matrices. The global priority vector is calculated through the elaboration of an overall matrix that includes the local priorities of each alternative concerning each criterion. Then, each column of vectors is multiplied by the priority corresponding to each criterion and add across each row which results in the desired vector of best alternatives ordered in a ranking of importance or preference (Saaty, 1990).

In summary, the AHP is a methodology for relative measurements. Relative measurements theory adapts suitably some decision problems where the best alternative has to be chosen. Thus, the ultimate scope of the AHP is to apply pairwise comparisons between alternatives as inputs, to produce a rating of alternatives, compatible with the theory of relative measurements (Brunelli, 2015). Further details regarding the traditional AHP method can be found in (Saaty, 1987, 1990, 2008b). Nevertheless, the methodology described in the next section of this chapter follows the same process as the AHP method, but some changes are introduced to adapt it to the context of the study.

5.3.2 Dynamic Maintenance Model for RC Structures

As it has been seen, decision-making is essential in the maintenance management process of buildings, structures and infrastructures. The intervention planning is directed by a set of multi-criteria, which generally diverge from each other concerning the objective sought by the decision-making process. In this section, a decision-making model based on the AHP method is developed. The proposed model has two main advantages. One is the weights assignment for the alternatives concerning each criterion carried out through a probability formulation instead of the experts' judgement. This leaves aside the subjectivity of the traditional AHP method by considering the uncertainty of the deterioration process, which is stochastically addressed.

The other advantage consist on development of a dynamic model since all the mathematical formulations are in function of time, which allows evaluating the decision-making for any moment of the service life of the structure. Furthermore, it is possible to consider any number of inspections and repair techniques available permitting maintenance planning to be suitable to each case. Hereafter, the dynamic model for decision-making is systematically explained and this methodology will then be applied to a hypothetical case study to illustrate the applicability and usefulness of the proposed model.

5.3.2.1 Step 1: Structure of the MCDM model

§*1.1 Hierarchical structure of the problem.* Implementing the traditional AHP, the decision problem is hierarchically structured. For the problem under study, the goal is to find the best intervention alternative for the maintenance planning of RC structures with corrosion risk. At the next level, the criteria that are decisive to attain the objective are established. Four criteria are adopted in this study. For the model, two criteria are formulated regarding the inspection of the structure, namely the probability of damage detection and the inspection planning cost. Furthermore, other two criteria that consider the repair action of the structure are also considered, namely, the effectiveness of the repair action and the repair cost.

In the last level of the hierarchical structure, the available alternatives are set. In this research, the range of alternatives is established combining different inspection techniques and repair methods available to perform the intervention in the structure, allowing to know the best way to perform the intervention in the structure for a specific time.

§*1.2 Inspection techniques and Repair methods.* Knowing the inspection and repair methods available to perform maintenance is essential to address the decision problem since the final result of the study will depend on the capabilities of these methods. As has been addressed in Chapter 3, there are several inspection techniques to detect the corrosion risk in the structure, namely half-cell potential, linear polarisation resistance, resistivity, acoustic emission, among others. Likewise, there are several repair methods for structures damaged by corrosion, such as cleaning the corroded rebar, cathodic protection, realkalisation, and so on (Dyer, 2014). Nonetheless, the main objective of this research is not to prove the effectiveness of each method but to propose a decision-making model that allows, once it is known the characteristics of each technique/method, to select which one is the most suitable way to perform the intervention. Furthermore, to establish the quality of a technique or method is extremely complex since it does not only depend on the equipment used but also on the damage degree of the structure and of the technician's experience in charge of performing the intervention.

Considering the above, this study evaluates the repair and inspection techniques, assuming the values of the parameters that define its quality and capabilities as has been done in the previous section. These parameters are the mean damage degree of the structure required to detecting or repair such a damage $\eta_{0.5}$, the standard deviation associated to the mean σ, the unit cost of each technique/method α that is assumed as a fraction of the total cost of the infrastructure, and a maximum damage degree η_{max} established as an upper boundary point for the function that describes the capacity of detection/repair of each technique/method.

Although the model can be adapted for any number of techniques and methods available for the decision maker, as simplicity, in the application example developed in this study three inspection techniques and two repair methods are considered. Thus, the inspection techniques assumed correspond to the half-cell potential, the linear polarisation resistance and the resistivity measurements (techniques *A, B* and *C* considered in Table 5.2). On the other hand, for the repair methods were considered a minor and a major repair in the structure. A minor repair may involve removing the damaged concrete cover, cleaning the corroded rebar and protecting it with some corrosion inhibitor paint. A major repair may involve, in addition to the aforementioned, the replacement of the cover with a sound concrete and resistant to corrosion agents (e.g. carbonation and chlorides), and finally a coating, membrane or sealer may be applied to the concrete surface (Broomfield, 2007; Dyer, 2014; Von Fay, 2015). Figure 5.12 shows an illustrative scheme of these two repair methods that will be considered in the decision model.

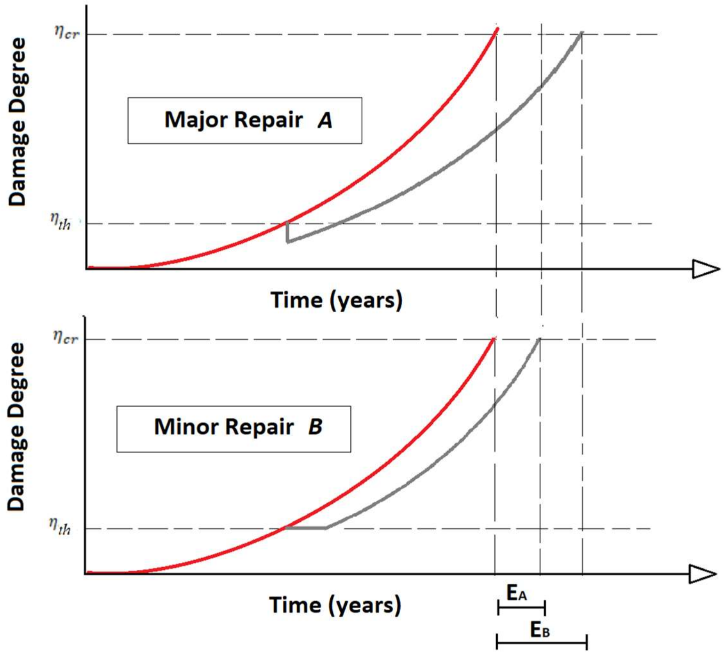


Figure 5.12 Scheme formulated for repair methods.

Where E_A, E_B represents the efficiency of both repair methods, η_{th} is the threshold of damage that defines the need of repair in the structure, η_{cr} is the critical damage degree for failure in the structure. All these parameters will be defined and formulated further on in this chapter. From the previous figure, it can be noticed that the efficiency of a repair method can be obtained by the relationship between the expected time to reach the critical damage in the structure without repair actions (red line) and the performance of the structure including the repair (grey line). In turn, this effect is related to the damage degree in the structure before and after the repair.

In practice, it is possible to note that there is more than one "correct" solution for any given repair project and that the economy will often dictate the choice (Morgan, 1996). Nonetheless, the use of repair materials with good quality and with the addition of corrosion inhibitors remains, frequently, the primary and least expensive solution in corrosion protection in RC structures along with good construction practices and proper quality design (Vaysburd and Emmons, 2000). In summary, the choice of certain inspection/repair techniques within a maintenance strategy depends on aspects such as the size of the project, the type of inspected phenomenon, accessibility to the area of intervention, socio-economic aspects (e.g. availability of resources) and the use of the structure (Bastidas-Arteaga and Schoefs, 2012).

5.3.2.2 Step 2: Set of criteria and alternatives

The second step of the decision-making model includes the definition of criteria and alternatives by means of probability functions. These functions allow adapting the AHP method to the uncertainties of the degradation of structures and, therefore, to carry out the most properly maintenance planning. In the next points, these mathematical expressions are described as a function of time that enables to address the decision problem from a dynamic approach throughout the service life of the structure.

§*2.1 First criterion: Probability of damage detection of the inspection technique.* The detectability of an inspection technique can be defined according to the probability of damage detection in the structure. For this reason, the damage degree in the reinforcement must first be quantified. This damage degree depends mainly on the rebar diameter and the corrosion rate. Therefore, the higher the corrosion rate, the faster the advance of the damage. So, a common way of expressing the corrosion damage degree in the structure is relating the cross-section loss in the reinforcement as a function of time. Therefore, the same formulation described in Eq. (5.1) is applied again in this section to determine the damage degree in the structure. Indeed, the methodology to determine the probability of damage detection follows the same structure of the mathematical formulation addressed in Section 5.2.2.2 of this chapter. Hence, the probability of damage detection will be obtained through the Eq. (5.11) showed previously.

§*2.2 Second criterion: Efficiency of the repair method.* The efficiency of a repair method can be estimated from the damage degree of the structure after the intervention η_{rep} concerning its damage condition before the repair $\eta(t)$. As well as with the inspection techniques, the repair method is more effective the greater its repair capacity in the structure. This can be translated directly in terms of durability, where the adaptability of the repair material is a meaningful parameter for the durability of the repair. Consequently, making a proper choice of the repair material is required to have a durable and efficient repair (Gadri and Guettala, 2017).

To measure the effectiveness of a repair method, it is necessary first to define certain parameters. First of all, it is assumed that the repair is made from a specific damage degree in the structure. So, a threshold of damage degree is established from which the repair must be made so that the maintenance is preventive. According to Cheung *et al*., it is established that for a cross-sectional loss between 10 % and 25 % of the reinforcement, the structure could present a condition of reduction of its initial capabilities, and the repair works must be carried out (Cheung et al., 2012). In this way, a threshold damage degree to perform the repair must be set as $0.1 \leq \eta_{th} < 0.25$. Thus, the probability of doing a repair can be formulated as:

$$P_{DR}(\eta(t)) = P[\eta \geq \eta_{th}] = \begin{cases} 0, & \eta < \eta_{th} \\ 1, & \eta \geq \eta_{th} \end{cases} \tag{5.23}$$

Where $P_{DR}(\eta(t))$ is the probability of "Doing Repair" in the structure; η is the damage degree at the intervention time t, and η_{th} is the threshold established to perform a repair. It should be noted that if $P_{DR}(\eta(t)) = 0$ at the intervention time, the efficiency is null since no repair work is required.

Another parameter that influences the repair is the capability that has the repair method according to the damage degree. In other words, the damage degree can determine the repair method since, for a critical degradation condition, more rigorous repair work may be required. So, this parameter is similar to the effect that the detectability of an inspection technique has on the structure. Hence, through a formulation comparable to Eq. (5.8) and considering the assumption for Eq. (5.10), this repair capability can be defined as the probability distribution expressed as:

$$P_{RR}^{\gamma}(\eta) = \begin{cases} p_1(\eta) & \text{, if } \eta \leq \eta_{min}^{\gamma} \\ \Phi\left(\dfrac{\eta - \eta_{0.5}^{\gamma}}{\sigma^{\gamma}}\right) & \text{, if } \eta_{min}^{\gamma} < \eta \leq \eta_{max}^{\gamma} \\ p_2(\eta) & \text{, if } \eta > \eta_{max}^{\gamma} \end{cases} \tag{5.24}$$

Where $P_{RR}^{\gamma}(\eta)$ is the probability of improve the damage condition in the structure, $\eta_{0.5}^{\gamma}$ and σ^{γ} are the mean and standard deviation of damage degree associated with each repair method γ to improve the damage degree in the structure; η_{min}^{γ} and η_{max}^{γ} are boundary points for the capabilities of repair methods.

With these parameters previously defined, it is possible to estimate the damage degree in the structure after the repair work η_{rep}. This damage degree after the repair could be equal to zero if the repair method attains to eliminate the damage and return the structure to its original state. However, this whole reparation is not always achieved as many aspects must be considered to make a correct repair. A negative value for η_{rep} can also be allowed in some cases, which implies a reinforcement in the structure or a replacement of the damaged element and not only a repair. Nevertheless, the scope of this study is given by the preventive maintenance of the structure through which it is trying to plan repairs before the occurrence of a failure. Thus, the damage degree after repair can be expressed as:

$$\eta_{rep} = K_{\eta}\eta(t) = [1 - P_{DR}(\eta(t))P_{RR}^{\gamma}(\eta)\varpi^{\gamma}]\eta(t) \tag{5.25}$$

Where $\eta(t)$ is the damage degree at the intervention time; K_{η} is a coefficient of degradation improvement given by the repair method; and ϖ^{γ} is a coefficient of maximum repair achieved by the repair method γ. Lastly, the efficiency of a repair method may be inferred through a general form that relates the damage degree at the intervention time with the damage degree after repair through the expression:

$$eff_{R} = \frac{\eta(t) - \eta_{rep}}{\eta(t)} = \frac{\eta(t) - K_{\eta}\eta(t)}{\eta(t)} = 1 - K_{\eta} \tag{5.26}$$

where if it is considered the Eq. (5.24) in the last equation, the efficiency of a repair method may be formulated as:

$$eff_{R} = P_{DR}(\eta(t))P_{RR}^{\gamma}(\eta)\varpi^{\gamma} \qquad for \qquad P_{DR}(\eta(t)) = 1 \tag{5.27}$$

The efficiency of the repair is essential in maintenance planning. An efficient repair work made in time allows to reduce the intervention costs and ensure the structural reliability. Reliability engineering is dedicated to the maintenance function and is focused on the elimination of repetitive failure. It comprises a strategic activity focused on the future that ensures the best life-cycle cost. So, an efficient repair must allow extending the durability of the structure, preserving its capabilities and its service conditions estimated during the design phase. Regarding the preventive maintenance tasks performed in engineering, studies have found that between 33% and 42% of such tasks have little effectiveness to preserve the reliability of the structures. Therefore, preventive maintenance must be based on reliability in order to decrease these no-value tasks through specific maintenance activities that both prevents failures and extends the service life of the structures (Mobley et al., 2008).

§2.3 Third criterion: Cost of the inspection technique. The first two criteria established the capabilities of the inspection and repair methods that can be applied during the maintenance process of a structure. Maintenance planning always aims to apply the intervention with the highest quality and effectiveness to obtain the most optimal results. Nonetheless, the quality of intervention is always related to the operational cost associated with a specific technique or method of intervention. In this way, it is always necessary to make a compensation between the cost and quality of the maintenance planning.

As has been mentioned in the formulation of the decision model for the inspection planning, the cost of the inspection technique depends on several parameters. Therefore, some considerations must be considered such as the capabilities of the equipment to detect the damage (i.e., the minimum damage degree in the structure so that the technique may detect it), the complexity to achieve the structural element to be inspected, the expertise of the operator that carries out the inspection, among others. Thus, the individual cost of each inspection technique θ can be expressed as (Mori and Ellingwood, 1994):

$$C_{insp}^{\theta} = \alpha_{insp}^{\theta}\left(1 - \eta_{min}^{\theta}\right)^{20} \tag{5.28}$$

where α_{insp}^{θ} is the cost associated with an inspection technique assumed as a fraction of the total initial cost of the structure and η_{min}^{θ} is the minimum damage degree that may be detected for an inspection technique θ.

Then, is necessary to consider the value of money over time. For this, a real cost of the inspection technique is calculated through a net discount rate *r* that gives the Net Present Value (NPV) of the inspection cost throughout the service lifetime of the structure (Yang and Frangopol, 2018a):

$$C_{NPV}^{\theta} = C_{insp}^{\theta}\frac{1}{(1+r)^{t}} \tag{5.29}$$

As it can be noticed, the formulation of the costs in this section is quite similar to the Eq. (5.15). Nonetheless, the difference is presented in that in Eq. (5.29) is not considered the sum of the function because, for the dynamic model proposed in this section, the analysis is performed individually for each time of the service life. This approach will be addressed later in the section where the discussion of the application of the model is presented.

The interest rate applied in cost analysis is an important parameter of influence and, due to the uncertainty between the estimated costs and the real costs during the building life cycle, it becomes impossible to make accurate projections in long-term (Rodrigues et al., 2018). Indeed, considering the discount rate in the formulation gives the chance of performing the same maintenance option at a different time in the future where each one has a different calculated present cost. Thus, the optimisation process provides multiple solutions through the application of the same method, but with varying times of intervention (Soliman et al., 2013). The inspection cost is quite important as it works as a counterbalance to the inspection quality within the decision-making process. It is to be expected that the best option in maintenance planning is to periodically inspect the structure to detect the damage in time. However, this implies an increment in the inspection cost that would directly compromise the total life-cycle cost of the structure.

§2.4 Fourth criterion: Cost of the repair method. Similar to the previous criterion, the repair cost is influenced by the method applied in the intervention and by its capacity to decrease the damage. Moreover, the more effective the repair method is, the greater the cost of the intervention will be. Therefore, this controversy of criteria is quite influential in the decision making regarding the maintenance of structures. The cost of this method depends on the damage condition of the structure and the expected reliability degree of the structure after the intervention. There are repair methods that stop the damage for a while and other methods that decrease the damage degree. Both methods finally achieve the aim of any maintenance intervention, which is to extend the service life of the structure and preserve its durability.

The unit cost of the repair is then related to the efficiency of the method implemented and the cost of a specific repair method can be estimated as

$$C_{rep}^{\gamma} = \alpha^{\gamma}\left(eff_{R}\right) \tag{5.30}$$

where α^{γ} is the cost associated with a repair method that is assumed as a fraction of the total initial cost of the structure, and eff_R is the efficiency of the repair method γ. Then, as for the inspection cost, the net present value for the repair cost may be formulated as follows

$$C_{NPV}^{\gamma} = C_{rep}^{\gamma} \frac{1}{(1+r)^t} \tag{5.31}$$

Whether the damage degree is considerable, the repair cost will be high and, even, may be necessary to apply a replacement of the damaged structural element. For this, establishing a damage threshold for a repair work that does not exceed the critical damage of failure is quite meaningful. Studies suggest a damage degree by corrosion higher than 0.25 to reach structural failure (Cheung et al., 2012). Hence, it is advisable do not exceed such value to preserve the preventive maintenance in the structure. Moreover, a failure in a structural element will lead to expensive repair costs which is what is sought to avoid with the maintenance strategy proposed in this investigation.

§2.5 Set of alternatives to achieve the objective. In this study, the alternatives attempt to describe the best way to perform an intervention on a structure damaged by corrosion throughout its service life. Therefore, the set of alternatives consists of the combination of the different inspection techniques and repair methods that were considered in Section 5.3.2.1. Then, considering a set of inspection technique $\theta = \{I_1, I_2, \dots, I_n\}$ and a set of repair methods $\gamma = \{R_1, R_2, \dots, R_m\}$, the set of alternatives can be generated as the vector *A*:

$$A = \begin{pmatrix} A_1 \\ A_2 \\ \cdots \\ A_n \end{pmatrix} = \begin{pmatrix} I_1 R_1 \\ I_1 R_2 \\ \cdots \\ I_n R_m \end{pmatrix} \tag{5.32}$$

These alternatives seek to determine, for a given intervention time, which one is the best inspection technique to be applied. Subsequently, this inspection technique determines the damage degree, which if it is higher than the threshold damage degree η_{th}, then the repair should be carried out by the method combined with the inspection technique for such alternative.

5.3.2.3 Step 3: Pairwise comparison matrix of criteria

For this step, the decision-making model follows the approach of the traditional AHP method elaborating a comparison matrix of criteria to know the priority level among them. In this matrix, the level of importance between criteria is analysed using the weight scale established in Section 5.3.1. A common problem on this step is to find some inconsistency due to the randomness of the experts' opinion. Thus, if there is some inconsistency in the matrix ($CR \leq 0.1$), the decision maker must review the values of the comparison matrix to improve the consistency ratio and so proceed with the analysis (Russo and Camanho, 2015).

To deal with this problem, this research proposes a correction factor λ_{CR} that must be multiplied by each element of the comparison matrix to achieve a permissible *CR* for the analysis. This correction factor will avoid modifying as less as possible the experts' judgement since such factor will reduce the inconsistency of the matrix proportionately.

5.3.2.4 Step 4: Probabilistic index for each criterion - Stochastic approach

The uncertainty in the decision-making for the maintenance planning claims for an analysis from a probabilistic approach. Although the AHP method has been widely validated to solve complex problems in decision- making, the final result is directly influenced by the subjectivity in the weights assignment for each criterion and alternative. To bring down the subjectivity, this research proposes the elaboration of the comparison matrices from a stochastic perspective. Herein, an index is proposed for each criterion considered, which is directly related to each alternative. This index is obtained regarding the equations formulated for each criterion in Section 5.3.2.2. This proposed approach will allow knowing the priority level of each alternative for each time over the service life of a structure. Table 5.8 shows a schematic of how the indexes are established for, in this case, the criterion referred to the inspection cost. However, the same indexes must be formulated for each criterion.

Table 5.8 Stochastic indexes for each criterion.

Criterion	A_1	A_2	A_3	A_n
Inspection Cost	I_i	⋯	⋯	I_j

Although they are called "indexes", the values of the previous table will be represented by equations as a function of time that will allow performing the analysis for each time *t* required. That is, to know the value of such an index accurately, it will be necessary to give the intervention time as an input value in the decision model. In this step, the priority vector of each alternative regarding the criteria considered is calculated as per the same method described in Section 5.3.1.

5.3.2.5 Step 5: Pairwise comparison matrix of alternatives

Once the indexes for each criterion have been established, the matrix of alternatives can be elaborated based on such values. In this step, the new approach for the AHP method is proposed, where instead of the weights assignment for each alternative through expert judgement, these values are calculated. Table 5.9 shows the structure of the comparison matrix, where it can be noted a similar structure to the traditional AHP method.

Table 5.9 Comparison matrix of alternatives.

Criterion	A_1	A_2	A_3	A_n
A_1	1	⋯	⋯	a_{ij}
A_2	⋯	1	⋯	⋯
A_3	⋯	⋯	1	⋯
A_n	$1/a_{ij}$	⋯	⋯	1

This matrix must be formulated for each criterion of the decision model. Hence, for the case considered in this research, four pairwise comparison matrix of alternatives will be elaborated. The a_{ij} values in the matrix are calculated according to the following expressions:

$$\xi_{ij} = \left[(8|I_i - I_j|) + 1\right]^{S_{ij}} \qquad with \quad S_{ij} = \begin{cases} 1, & I_i \geq I_j \\ -1, & I_i < I_j \end{cases} \tag{5.33}$$

where I_i and I_j are two different indexes for a specific criterion established in Step 4, ξ_{ij} is a parameter that considers the level of importance of each alternative regarding a criterion, and S_{ij} is a factor that considers the reciprocity between the elements of the matrix. Then, it is possible to calculate the weight of each alternative through the expression:

$$a_{ij} = \xi_{ij}\hat{C} \tag{5.34}$$

Where $\hat{C}$ is a consistency factor that preserves a consistency ration lower that 0.1 in the comparison matrix. Eq. (5.34) considers the same range established in (Saaty, 1980) for the importance scale and the principle of reciprocity between the elements. As in the previous step, the priority vector of each generated matrix must be calculated for its subsequent application in the next level of the hierarchical structure. This vector is commonly known as "local priorities" and is calculated in the same way as in the other matrices. This step is paramount for the proposed decision model, since the elements of the matrices are no longer designated by the experts but are calculated based on the probability of occurrence of each situation.

5.3.2.6 Step 6: Global Priority Vector of Alternatives

Lastly, the pairwise comparison matrix between alternatives and criteria is elaborated based on the matrices generated in Steps 3 and 5. From this stage, the proposed decision-making model does not differ from the traditional AHP method. That is, the final comparison matrix is elaborated based on the local priority vector of alternatives and criteria. Then, the global priority vector (GPV) is defined for each alternative depending on the time. Table 5.10 shows an example of the final comparison matrix with the global priority vector for each alternative.

Table 5.10 Example for the final pairwise comparison matrix between criteria and alternatives.

	Criterion A	Criterion B	Criterion C	Criterion D	Global Priority Vector
Alternative 1	a_{1A}	a_{1B}	a_{1C}	a_{1D}	GPV_1
Alternative 2	a_{2A}	a_{2B}	a_{2C}	a_{2D}	GPV_2
Alternative 3	a_{3A}	a_{3B}	a_{3C}	a_{3D}	GPV_3
Alternative 4	a_{4A}	a_{4B}	a_{4C}	a_{4D}	GPV_4

The intervention time is the main input value of the model since, for each value, the model gives as an output a different global priority vector for the set of alternatives. Therefore, after this stage of the dynamic decision model, it is defined which one is the best alternative to be applied for the maintenance in a given time. In the traditional AHP method, at the end of the process, a comparison matrix is obtained between alternatives and criteria. The dynamic model for decision-making proposed in this research gives as a final result a graph that represents the curves for each alternative as a function of time. In this way, it is possible to know from the curve for each moment of time in the service life of a structure, which one is the alternative that has the highest priority and how high is this priority concerning the other alternatives.

5.3.3 Results of the Methodology

Similar to what has been elaborated in Section 5.2.3, this section presents a numerical example for the illustration of the proposed decision-making model. The criteria and alternatives considered to attain the goal of the decision problem have already described in the previous section. Considering that the decision model is formulated under a stochastic approach, the values of the parameters involved were assumed with a lognormal distribution and are summarized in Table 5.11. As can be seen, the parameters are following the analysis developed for the inspection planning performed at the beginning of this chapter. Furthermore, the same value determined in section 5.2.3 has been adopted for the time of corrosion initiation.

The threshold value for the damage degree to perform the repair was adopted as explained in (Cheung et al., 2012). These parameters are used to determine the damage degree in the structure during its service life. Subsequently, the inspection techniques and repair methods must be defined. The inspection techniques to be applied for maintenance can differ according to the mechanism of damage. This research considers three different techniques according to certain parameters that define their capacities. Once again, to follow the analysis performed previously, these techniques are the half-cell potential, the resistivity measurements and the LPR technique. Therefore, the parameters considered for these techniques are the same exposed in Table 5.2.

Table 5.11 Random variables considered for the study.

Variable	Units	Distribution and Value
d_o	cm	$LogN, \quad \mu = 1.6, \quad \sigma = 0.020$
η_{th}	Percentage	0.12
V_{corr}	cm/year	$LogN, \quad \mu = 0.015, \quad \sigma = 0.0015$
T_{icorr}	Year	$LogN, \quad \mu = 5.91, \quad \sigma = 1.27$

In the same way as the previous one, the parameters associated with the repair methods regarding the capacity to restore the damage are defined. The description of each of these parameters has already been developed in the previous section and the values are presented in Table 5.12. For the repair were considered only two methods so as not to overextend the number of alternatives analysed. Nevertheless, the decision maker may apply in the decision-making model any number of available methods to determine the best intervention alternative. Thus, these methods correspond to a major repair and a minor repair in the structure.

Table 5.12 Repairs method considered for the analysis.

Repair Method	$\eta_{0.5}$	σ	η_{max}	ϖ^{γ}	α^{γ}
Major Repair (A)	0.15	0.015	0.78	0.90	0.023
Minor Repair (B)	0.26	0.030	0.62	0.90	0.015

For simplicity notation, also the repair methods will be nominated as method A and B. When establishing the different inspections and repairs, the set of alternatives is obtained from the combination of each technique between each other. In this way, a total of six intervention alternatives are established for the numerical example that will be evaluated in the final comparison matrix.

Once the initial parameters have been defined and the decision problem has been appropriately structured, the comparison matrix for the second level of the hierarchical structure is elaborated. This is a matrix between criteria whose main objective is to determine the level of importance of each criterion concerning another. After consulting with some experts regarding the weights assignment for the matrix, an inconsistency in the results was found through a Consistency Ratio (CR) equal to 0.359. To satisfy the admissible CR value (≤ 0.1) and proceed with the analysis, each element of the matrix was multiplied by a correction factor λ_{CR} to reduce the inconsistency. Thus, for a correction factor $\lambda_{CR} = 0.333$, the new value of the consistency rate was $CR = 0.074$. Table 5.13 shows the pairwise comparison matrix for criteria with the final weights after applying the correction factor. In this table, C_1, C_2, C_3, C_4 correspond to the first, second, third and fourth criterion respectively, which have been described in Section 5.3.2.2.

After calculating the weights so that they fulfil the consistency, the priority vector is calculated. For this case, from Table 5.13 and for the assigned weights, the criterion of highest priority is the detectability of each inspection technique while the cost of repair is the one with the least priority. The meaning and the influence of these results in the decision-making will be discussed in the successive section. Subsequently, the process continues with an analysis of the next level. Thus, a comparison matrix between alternatives for each criterion is performed. In this way, four six-by-six matrices are necessary for this numerical example.

Table 5.13 Pairwise comparison matrix of criteria $CR = 0.074$.

	C_1	C_2	C_3	C_4	***Priority Vector***
C_1	1	1.400	0.500	0.250	0.475
C_2	0.714	1	1.500	0.385	0.202
C_3	2.000	0.667	1	0.500	0.177
C_4	4.000	2.600	2.000	1	0.146

Bearing in mind that the proposed decision model needs a given time *t* as an input value to provide the results, this study does not show the values of these four matrices of comparison of alternatives. Actually, the main result of this proposed model is not to determine a final comparison matrix, but a set of curves generated according to the priority of each alternative as a function of time. Hence, once the probabilistic indexes have been established, the weights of this comparison matrix are calculated for each time *t* of the service life of the structure.

Before carrying out the analysis of the final comparison matrix between alternatives and criteria, the formulations of the decision model that define the capabilities of each inspection technique and repair method have been solved. Figure 5.13 shows the probability density function that defines the performance for each technique/method over time. This distribution depends on the parameters defined in Table 5.2 and 5.12 as well as the damage degree in the structure. Moreover, the distribution of inspection and repair capabilities influence the final priority of each alternative of the final comparison matrix.

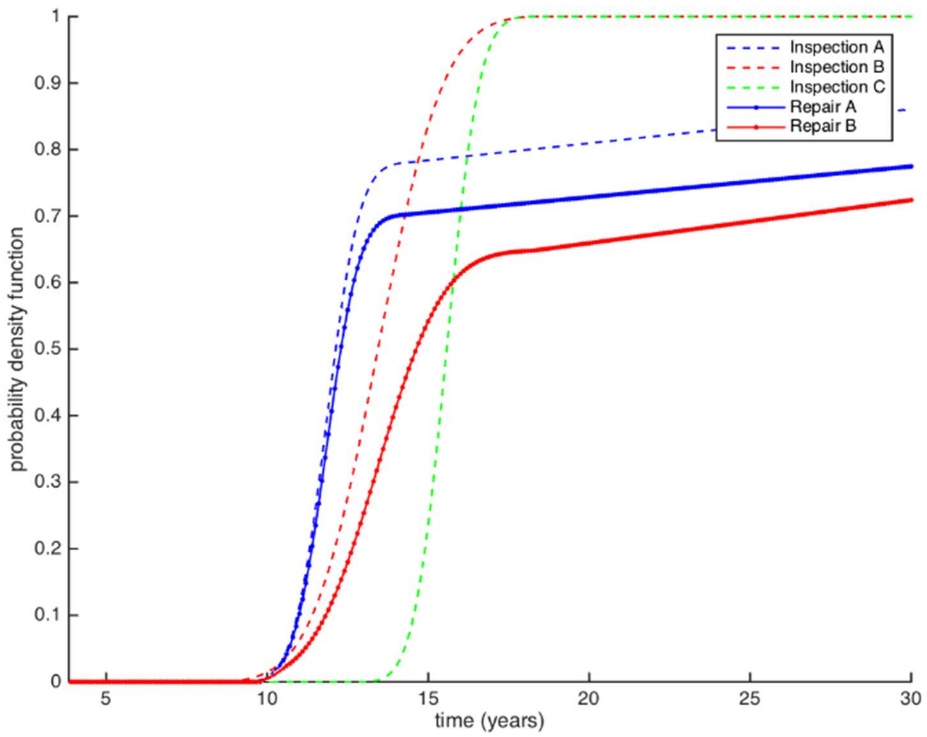


Figure 5.13 Capabilities of the inspection and repairs over the service life.

The traditional AHP model proposes in its last step a final comparison matrix between criteria and alternatives to calculate the global priority vector of each alternative. The proposed dynamic model has been constructed and solved using the MATLAB software, version R2015a. Then, by introducing the time as an input value, the dynamic model allows evaluating the AHP method proposed for each any time systematically. The final result of this simulation is depicted in Figure 5.14 where it can be seen the curves of the distribution function of alternatives concerning time.

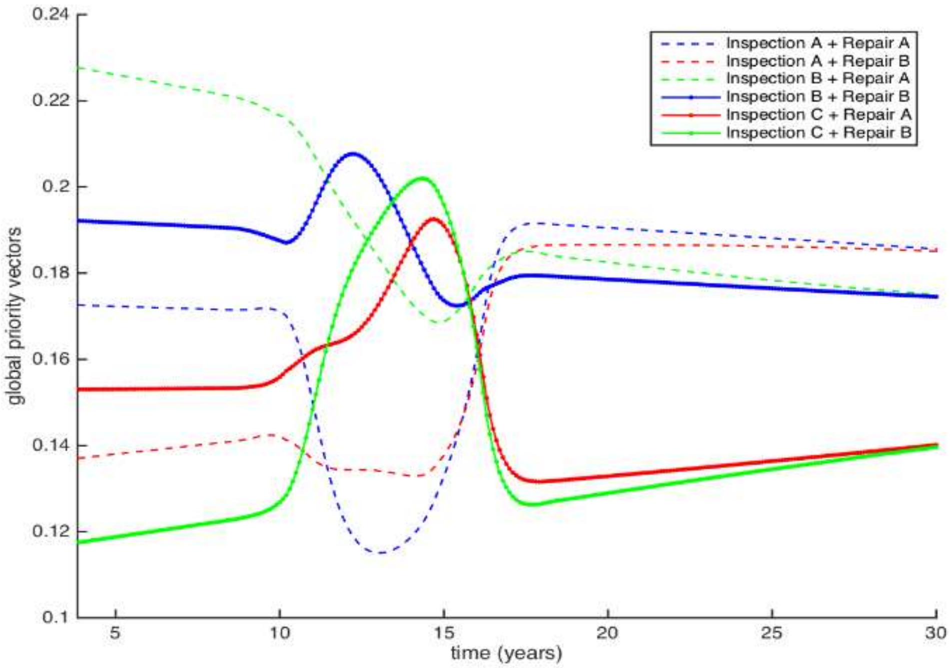


Figure 5.14 Global priorities of maintenance alternatives over time.

As can be seen in Figure 5.14, each alternative has a higher priority than another depending on the lifespan of the structure. This is an expected result since the damage degree and the state of conservation of the structure change over time. Likewise, the results obtained in this figure are directly influenced by the results shown in Figure 5.13. Therefore, to better visualise the result of Figure 5.14, Table 5.14 shows a summary that is the final outcome of the dynamic model proposed in this chapter for decision-making in maintenance management of RC structures.

Table 5.14 Best alternatives of intervention for the maintenance planning of structures

$\Delta Time$ $(year)$	Best Alternative	GPV_{min}	GPV_{max}	GPV_{avg}
0.0 – 11.4	$Inspection\ (B) + Repair\ (A)$	0.202	0.233	0.224
11.5 – 13.4	$Inspection\ (B) + Repair\ (B)$	0.197	0.208	0.204
13.5 – 15.5	$Inspection\ (C) + Repair\ (B)$	0.183	0.202	0.197
15.6 – 15.7	$Inspection\ (C) + Repair\ (A)$	0.177	0.180	0.179
15.8 – 16.5	$Inspection\ (B) + Repair\ (A)$	0.175	0.182	0.179
16.6 – 30.0	$Inspection\ (A) + Repair\ (A)$	0.182	0.193	0.188

This table allows visualising the best alternative (inspection + repair) to be applied in the maintenance planning during a specified period of time ($\Delta Time$). Also, it allows seeing clearly the level of priority that each alternative has for that period. It should be noted that the example described in this section is merely established as a hypothetical case study that allows verification of the application of the dynamic decision model proposed.

5.3.4 Analysis of study

The multi-criteria decision-making models are a useful tool for the maintenance planning of constructed structures and infrastructures. In general, the criteria considered within the maintenance of structures conflict with each other regarding the obtaining of an optimal result. So, the cost/quality relationship of the intervention works must be analysed in detail since the parameters that define these criteria tend to change over time. These changes occur in a random manner generating uncertainties that make the analysis complex. Therefore, it is necessary to generate tools that are simple to apply and, at the same time, effective for the maintenance process.

This section develops a dynamic model for decision-making that can be used for the maintenance planning of concrete structures. This model is based on the AHP, which has already been widely developed in several fields of research. This method has great advantages and limitations. Among its advantages, the AHP has a clear structure that facilitates its application in any decision problem. Also, it allows controlling the weights assignment of criteria through the consistency ratio, which gives some robustness to the analysis and an acceptable approximation to other results obtained from different analytical methods. However, its main shortcoming lies in the high subjectivity with which the analysis is established. For instance, when the comparison matrices are performed, an essential requirement is different experts' judgement about the level of importance between criteria. This leads to an inconsistent comparison matrix for which the decision maker must necessarily readjust its values, according to his own opinion once again, until finding the consistency between weights. In this way, the subjectivity in the allocation of weights for each attribute could be decisive when evaluating the best intervention alternative in a structure maintenance planning.

To counteract this subjectivity and take advantage of the traditional AHP method, this study proposes the formulation of indexes that describe each criterion from a stochastic perspective. These indexes are then used to determine the weights of each alternative instead of being established nominally by experts or by the decision maker. Thus, by developing the method from a probabilistic approach, some variables must be randomly considered according to a particular probability distribution. The parameters adopted in Table 5.1 and 5.11 were assumed based on other studies found in the literature. It should be recognized that the best way to establish the value of these parameters is through a particular case study where these variables can be periodically measured to obtain a database that defines such distribution. Nevertheless, this would entail the development of a comprehensive work to obtain an extensive database for each parameter, which goes beyond the aims of this investigation. Hence, in many other studies in the engineering area, it is common to assume a lognormal or normal distribution for those parameters that are unknown in advance.

It should be noted that these random variables have a high sensitivity in the final result of the study. Moreover, according to Eq. (5.1), it can be inferred that the corrosion rate is very influential in the damage degree obtained over time. In turn, the advance of the damage over time affects the choice of one or another inspection technique, as well as the decision to perform or not the repair action in the structure. Therefore, it is necessary that these parameters are carefully assumed by the decision maker so that the maintenance planning is adapted to each particular case, i.e. the degradation mechanism, environmental exposition, type of structure, and so on.

Another aspect to be mentioned regarding the numerical example are the parameters assumed for the inspection and repair techniques. According to other studies found in the literature, defining these parameters with accuracy is a complex task due to the uncertainty comprise the degradation of structures and their maintenance. Therefore, these parameters should be adjusted after each intervention to update the capabilities of each technique an perform the maintenance more accurately. Likewise, the costs were established according to referential costs concerning the initial investment required to build the structure. Several investigations (Ellingwood and Mori, 1997; Dieulle et al., 2003; Chung et al., 2006; Kim and Frangopol, 2011; Liu et al., 2018) adopt the same criteria (referential cost), and the functionality of the analysis allows then to adapt the study to any market or any case around the world.

For the comparison matrix of criteria, however, it is necessary to perform the weights assignment of each criterion according to the traditional method proposed by the AHP. In this first stage of the analysis, it is defined which criterion is more important for the maintenance of the structure (the goal of the decision problem), for which it is necessary to develop a comprehensive study. Regardless of the amount or the knowledge of the experts involved in the judging of the weights between criteria, some inconsistency can always be expected. Therefore, to interfere as little as possible in this judgment, this research suggests the application of a correction factor λ_{CR} that allows reducing the inconsistency. The value for this factor can be found automatically once the model is formulated within the MATLAB platform. Otherwise, the decision maker should manually search for the value that allows a proper consistency in the matrix. In the example, it could be seen that for a correction factor of λ_{CR} = 1/3, the consistency ratio of CR = 0.359 could be reduced to CR = 0.074.

From the comparison matrix of criteria, it can be seen that the detectability of the inspection techniques is the most important to consider for the decision problem. This can be an expected result because if the inspection technique fails to detect the damage correctly, the repair work could not be carried out in time, leading to the failure of the structure. However, a certain degree of subjectivity always exists in the matrix that is set based on judgements. That is, depending on the type of structure or the maintenance budget established, the criterion with the highest priority may be another different than the one shown in the example. Nonetheless, for the hypothetical example of the previous section, the order of importance of criteria has been shown in Table 5.13 through the priority vector.

For the previous matrix, the intervention time does not directly influence in the weights assignments whereby the scale of importance formulated for the AHP method is used. However, for the comparison matrix of alternatives, the weights are not estimated but are calculated based on proposed indexes. These indexes consider the criteria from the stochastic perspective allowing the method to be adapted to the uncertainties of structures degradation. The method for the elaboration of these matrices has been explicitly developed in Section 5.3.2.5 of this chapter. Considering that there is then a set of four matrices for each given time, in the numerical example, these matrices have not been exposed in this chapter. In other words, if it is considered a time span of 50 years with the analysis performed each year, the model gives 200 matrices of six-by-six that is not interesting for the aims of the decision model since a more useful outcome is obtained at the end of the analysis with the curves of GPV. The proposed model allows calculating (through MATLAB) a set of matrices for each time. Then, the dynamic model gives at the end of the process, not one, but a set of matrices with which the global priority vector is calculated for each moment of time. Thus, with these matrices, the curves are elaborated for each alternative that is the final result of the decision-making model that has been formulated in this investigation.

In Figure 5.13, it can be seen the capabilities of detection/repair of each inspection technique and repair method throughout the life of the structure. With the passage of time, the level of corrosion damage in the reinforcement is increasing. This corrosion damage, if it is not treated in time, causes the formation of cracks due to the expansion of the oxidation products (rust), concluding in the spalling of the cover. The higher the damage degree, the higher the probability that it will be detected by a specific inspection technique. The parameter $\eta_{0.5}$ allows knowing the mean damage degree that determine the detectability of each technique concerning the damage degree. The lower this value, the higher the ability to detect the damage in the early stages of degradation. However, as can be seen in the figure, the half-cell potential technique (denoted as Inspection A) has a priori the highest initial detectability. On the other hand, as early as the maximum damage degree value η_{max} has been reached, this technique becomes less efficient than the other two since greater damage does not significantly alter its detectability. This controversy in the inspection process is quite typical in real cases, so it is not obvious to know in advance which technique is the best for the maintenance planning.

With the repair methods, the same situation presented with the inspections may occur. There is a certain damage degree from which a minor repair is not convenient from the viewpoint of the repair capacity of such method. For this, it is essential that the inspection technique attain to adequately detect the damage degree of the structure to be able to decide to correctly perform the repair. The threshold damage degree established in this work to perform the repair allows maintenance to be preventive. By performing the intervention before the occurrence of the failure, it avoids falling into high costs that affect the maintenance of the structure.

The most worthwhile and meaningful outcome of this chapter can be seen in Figure 5.14. The curves in this figure allow defining decision-making practically and dynamically. Once defined the initial parameters that will be used in the decision-making model, it is possible to obtain a similar curve for each case of study. Regardless of the time established to perform the intervention, this curve will allow the decision maker to know in advance what are the material resources he needs to implement the maintenance intervention, namely the technique of inspection and repair method. However, the repair method will only be applied if the damage degree detected by the inspection is higher than the threshold damage degree η_{th}, which is also defined by the decision maker. Depending on whether it wants to give a more conservative or less conservative approach to maintenance strategy, this value for the threshold damage degree should be established between 0.1 and 0.25 according to other studies. This will allow intervening the structure always in a period before the structural failure, ensuring the durability by means of the preventive maintenance.

It can also be seen in Figure 5.14 that the curves of the alternatives have a diffuse behaviour between 10 and 16 years of lifespan of the structure. This is because the degree of global priority between the different alternatives undergoes abrupt changes for this period. Also, this behaviour can be influenced by the probability distribution of the techniques and methods shown in Figure 5.13. Respectively, for this same period of time, there is a high controversy regarding which technique/method has an improved capacity to deal with damage in the structure. This controversy is then addressed by the AHP method that finally considers simultaneously all the criteria established to determine which one is the best alternative through the global priority vector.

Another secondary outcome of this dynamic decision model is given by the Table 5.14, which is nothing more than a detailed description of Figure 5.14. However, the benefit of this table is to know precisely the period in which to apply a specific alternative of intervention. It should be noted that the value of the global priority vector is a very important indicator of decision-making. Furthermore, it should also be noted that of the six alternatives initially proposed for the numerical example, just one of them is not considered into the maintenance planning. The alternative composed by the half-cell potential and minor repair (denoted as Inspection A + Repair B) is the only one that does not deserve to be considered during the whole service life of the structure since the other alternatives are always more dominant than this one.

Nevertheless, these results should be interpreted as a preliminary analysis of the best alternatives for the maintenance of structures based on a vector of priorities. In turn, this vector of priorities indicates the level of importance of each alternative against the criteria considered. That is, this result allows knowing the consistency of the alternatives for the established criteria through a certain ranking of importance. Therefore, the decision maker could choose a second-best alternative in the case that the analysis results in two different alternatives that do not differ considerably concerning their global priority.

Finally, the model has proven its dynamic feature according to the example of Section 5.3.3, whose main advantage is to be a support for decision-making regarding the maintenance of structures. For this, the proposed model comprises the elaboration of a set of global comparison matrices between alternatives and criteria that determine a curve for each alternative over time. For this particular case, the dynamic decision model analysed more than three hundred global comparison matrices for each moment of time to obtain the final result shown in Figure 5.14.

5.4 Summary

The maintenance of RC structures is an essential task to guarantee their conservation and durability. For this, it is paramount to have an accurate and clear understanding regarding the degradation mechanism that affects these structures. In turn, regardless of the degradation process considered, the uncertainty associated with the deterioration of a structure leads to maintenance planning being complex and often imprecise. Therefore, the best way to deal with the maintenance management of structure is through the formulation of numerical models for the decision-making that allow optimal results.

This chapter gives a study that may help decision makers to choose the most suitable intervention for the maintenance planning of RC structures. For the context of the study, the maintenances strategies have been formulated for the maintenance planning of RC structures with corrosion risk. Therefore, in the preceding sections, it has been presented sequentially how to apply the proposed model from a probabilistic perspective. This approach deals more adequately with the uncertainty inherent in the degradation of structures giving results that are more suitable for the application in real cases.

In the first stage of the maintenance strategies, a decision model for the inspection planning has been formulated. Several investigations developed in this engineering area do not consider the efficiency of maintenance planning directly. In the literature, it can be seen that some studies propose a set of optimal solutions obtained through a maximisation or minimisation of objective functions. To address such limitation, this research has applied two methods, namely SFA and MEA, for the efficiency analysis of a set of optimal solutions. The outcomes allow the decision maker to finally know which of all the optimal solutions is the most efficient.

The proper interpretation of the failure mechanism of the structure enables to know adequately the estimated time for a structure to reach a damage degree that compromises its structural safety. Likewise, this allows accurate planning from the approach of the capabilities of the inspection techniques for the damage detection before this failure is evidenced in the structure. Moreover, an early damage detection leads to a decrease in the total maintenance cost since, once the failure is reached in the structure, the repair costs could increase considerably.

Several methods are described in the literature to determine the probability of damage detection of an inspection technique. In this chapter, an improvement has been proposed for the formulation of such probability for corrosion detection to tackle the discontinuity between the boundary points (η_{min} and η_{max}). It has been seen that this discontinuity can be improved by considering the probability at the limit points as polynomials of cubic splines in order to fulfil the monotonicity and concavity at the boundary lines.

The MEA and SFA methods prove to be a useful tool for planning the maintenance of infrastructures from the efficiency approach. These methods contribute to knowing which one is the best option among a set of optimal solutions for maintenance planning. In addition, the efficiency analysis provides a solution that not only minimises costs and maximises results but also seeks to obtain the best use of resources and the time spent for maintenance. Even so, it should be emphasized that the SFA method seems to be a not entirely appropriate frontier analysis method in the scope of inspection planning and, by contrast, the MEA method provides a more comprehensive analysis for such study. This can be inferred since the SFA method does not compare the efficiency of the best performance of a data set but rather obtains the unit efficiency concerning the average performance. By contrast, the MEA method can deal with processes that consider several inputs and/or several outputs to measure the efficiency more comprehensively.

After obtaining the results of the efficiency analysis, it has been demonstrated that it is possible to achieve results highly efficient for the inspection planning. Such efficiency is evidenced by applying the minor variety of available inspection techniques in a shorter period of time. This avoids the waste of resources that can collaborate indirectly with the reduction of the environmental impact associated with the construction sector and the sustainability of infrastructures.

Through a reset of the input data, the decision-making method addressed in this chapter may be applied for the inspection planning of other types of structures subjected to other degradation mechanisms. In this way, future studies on maintenance management may implement the same efficiency analysis methods but taking into account other inputs/resources that give even better outputs/results from this approach. Nonetheless, it must be recognised that more studies are needed that allow, through real case studies, to adjust the parameters of the different inspection techniques that have been assumed in this research generically.

For the second stage of the maintenance strategies, a decision model based on the multi-attribute decision-making known as Analytic Hierarchy Process (AHP) has been addressed. The AHP-based dynamic decision-making model can be applied as a preliminary analysis to decide how to inspection and to repair structures over its whole service life. The AHP method has been widely developed and applied in the literature. However, this research has proposed an extended AHP-based decision model that decrease the subjectivity of the traditional method through a stochastic approach. The proposed decision model is considered dynamic since all the mathematical formulations are a function of time, which allows analysis of maintenance planning to be applied for any intervention time.

After its application in a numerical example, the dynamic decision-making model proposed has proven to be a helpful support tool for the maintenance planning of structures. This model allows determining the best alternative of intervention (i.e. inspection + repair) to be applied throughout the service life of the structure. The formulation of each criterion indexes allows introducing a stochastic analysis into the traditional AHP method. Thus, the study of probabilities enables to adapt the AHP method to the uncertainties given in the context of maintenance and the structural degradation.

The comparison matrix of criteria is influenced by the weights assignment that is given through the judgment of expert and/or stakeholders. This matrix determines, in a subjective way, which criterion is more important concerning another and it has a direct effect on the results of the final comparison matrix. Therefore, it is important that the weights assignments be carried out accurately and according to the conditions of each case.

It is recognised the need to develop a statistical analysis to be able to define the parameters that were assumed here as random variables. This is an analysis that will be time-consuming since must be collected an extended and reliable database to define the type of distribution for each parameter properly. However, these parameters, assumed as random variables for this study, may be updated and adjusted after each intervention to obtain a more suitable maintenance planning for each case.

Lastly, although the model has been applied to a case of corrosion-induced degradation in RC structures, it is important to note that the model can be adapted to other types of structures and other degradation mechanisms through an adjustment of the stochastic indexes. So, the equations formulated in this chapter should be reformulated for these cases. Furthermore, it can also be adapted for a greater number of alternatives as well as additional criteria and sub-criteria.

CHAPTER 6

Conclusions

CHAPTER 6

6 CONCLUSIONS

6.1 Final remarks

This thesis aimed to develop a decision model that can be easily applied for technicians and professionals to perform maintenance strategies for reinforced concrete structures under corrosion risk. The context of the study focuses on the carbonation-induced corrosion, that as has been demonstrated, is one of the most typical and expensive degradation mechanisms in infrastructures throughout the world. On the other hand, Paraguay does not have its structural concrete standards such as those of other countries in the region, namely CIRSOC:201 (Argentina), ABNT-NBR:6118 (Brazil), NSR-10 (Colombia), CBH-87 (Bolivia). Therefore, the results obtained in the degradation curves can be used as a reference to establish minimum cover thicknesses in the structures of the country considering the environmental exposition.

Reinforced concrete as a construction material has expanded around the world since the first half of the last century. This leads to suppose that a large part of the existing concrete infrastructures is near the end of their service life and need immediate intervention. Furthermore, considering the recurrent problem that climate change is having in all aspects of daily life, several researchers argue that the degradation of structures could be accentuated shortly as an effect of this climatic phenomenon. This motivated the realisation of this thesis for which fundamentals objectives have been established to address this problem. At the end of this thesis, it can be concluded that all these objectives outlined at the beginning of this work have been achieved through each chapter developed in this research.

Nonetheless, some limitations were felt when developing the activities proposed in this thesis. Firstly, considering that the research focuses on the need to preserve the durability of the structures in Paraguay, a significant limitation was the access to scientific information on this subject. The lack of digitalisation of the investigations developed in the country did not allow easy access to scientific information, for which the thesis does not include enough bibliographical reference in this subject area. The latter also applies to the case study, for which it was only possible to access cases of intervention in structures located in the city of Asunción.

Another limitation refers to the parameters used to obtain the degradation curve in the structures of Paraguay. The water/cement ratio, the air content in the structure and the current concentration of carbon dioxide in the urban area of the city of Asunción are some of the parameters that have been estimated due to the lack of real measurement of such parameters. Therefore, it is recommended that these data be recalibrated to obtain an even more accurate result on the degradation of these structures. The same circumstances have been found for the parameters adopted in Chapter 5. However, the values for such parameters have been assumed according to other several studies found in the literature that address the same subject.

6.2 Main Conclusions

After having conducted this research, some meaningful conclusions can be drawn from each developed chapter. Firstly, it has been shown that climate change is a global phenomenon that has an impact not only on the life quality of people but also on the durability of the structures. According to the climatic scenarios established by the IPCC, the perspective is not encouraging regarding the climatic variations expected until the end of this century. Regarding the climatic parameters associated to the degradation mechanism studied in this thesis, that is, the temperature and the atmospheric concentration of CO_2, the results show a rapid increase of their average values since the end of the past century.

This accelerated variation of the climatic parameters suggests that the best strategy to face this problem is to establish adaptive measures that allow stopping the early deterioration of concrete infrastructures. Studies regarding the degradation of structures against the climatic phenomena have shown that an increase of 2 °C above the current average temperature values could increase the corrosion rate by 15%. On the other hand, global warming could accelerate the time of failure in the structure by up to 31% or decrease the service life in up to 15 years for structures located in environments of moderate levels of aggressiveness. Regarding carbonation, the carbonation depth in the structures could increase by 45% by the end of 2100 if climate change continues to accentuate in the coming years. Therefore, the effect of climate change will force to return to the studies related to structural degradation to formulate the most appropriate strategies for the maintenance of the structures.

In the third chapter of this thesis, the degradation of concrete structures has been addressed. Although the structures can be degraded by different mechanisms, this study focused only on the corrosion of the reinforcement caused by carbonation. It has been seen that carbonation is a natural phenomenon directly associated with the environmental conditions surrounding the structure. This context allows relating the study to the effects of climate change easily. One of the main conclusions of this chapter is related to the degradation of concrete structures in Paraguay.

Some studies collected from the literature review have shown that one of the main problems in the structures of the country is the carbonation of concrete. Consequently, the low quality of infrastructures favours the premature degradation of concrete infrastructures, which implies an economic impact on their life cycle. Through the case study presented, it has been determined that one of the most influent parameters in the degradation of the structures is the insufficient thickness of the concrete cover. In the interventions carried out in the existing structures, it has been possible to verify even cover thicknesses between 1 and 10 mm. This deficiency has allowed the structures to present corrosion in the reinforcement in the first years of service, requiring an intensive repair to preserve its durability.

On the other hand, the cover thickness was not the only problem found in the structures of the case study. Some test results have shown structures with cover thickness above 25 mm whose reinforcement was completely corroded. This suggests a low quality in the concrete of the cover, that is, a high porosity that allows the passage of carbon dioxide into the interior facilitating the carbonation of the concrete. All these parameters must be considered at the execution time of the construction project to guarantee the durability of the structures. Thus, this goal must be achieved through a stricter control in the construction stage. Lastly, from the 206 carbonation tests performed in 38 different concrete structures of Paraguay, it was identified that the 49.07 % of these structures are under corrosion risk or imminent corrosion initiation. For this reason, considering the case study as a representative sample, it could be inferred that about half of the structures of Asunción need an immediate maintenance intervention.

Subsequently, in Chapter 4 an exhaustive research of the bibliography has been developed to choose a numerical model adapted to the approach of this thesis. This numerical model has been used to obtain the degradation curves by carbonation in the structures of Paraguay. From these curves has been found that, for a service life of 50 years, the maximum carbonation depth expected for under the considerations of the climatic scenarios RCP 4.5 and RCP 8.5 will be between 15 and 40 mm. These values depend on the concrete quality, i.e., the compressive strength of the concrete, and the foreseen climatic scenario. Therefore, considering a control scenario, it was found that for concrete structures in Paraguay is expected an average increase of carbonation depth up to 25 % for a worst climatic scenario until the period 2055-2065. On the other hand, the time to reach the same maximum carbonation depth of the control scenario can be reduced between 7 and 10 years for the best climate scenario, depending on the quality of the concrete.

Furthermore, the degradation curves obtained for Asunción also suggest that concrete with higher quality decreases the value of the ultimate carbonation depth. However, considering the results of Table 4.4, it was possible to notice that the time of corrosion onset increase more significantly for structures with a higher cover thickness (from 10 to 25 mm) than for structures with a better concrete quality (from 20 to 25 MPa). Hence, according to the study developed in Chapter 4, the cover thickness is referred as the most influential parameter to reduce the time to corrosion initiation.

Another result from Chapter 4 was the analysis of the time of corrosion initiation and the time of corrosion propagation for the structures on Paraguay. It has been shown the influence of cover thickness and concrete quality on these critical times under the corrosion perspective. Depending on the cover thickness and the concrete quality (i.e., 20, 25, 30 MPa), it is possible to define an increment in corrosion initiation times between 18 and 32 years in structures with a cover thickness of 25 mm regarding others with 10mm. This is a significant value considering that, in the case study, a large part of structures with a cover thickness less than or equal to 10 mm have been found.

Likewise, other interesting conclusion of Chapter 4 has been the validation of the model through real carbonation data. This is a very controversial topic in the elaboration of numerical models since the majority of them is validated only according to laboratory tests. The main limitation of this procedure is associated with the difference found between the properties of structures elaborated *in situ* and those that are controlled in the laboratory. The high complexity of comparing the results of the numeric model with real cases has also been experienced in this work. The main problem lies in the high variability of the parameters and properties between existing buildings, which makes it difficult to choose a specific value to obtain the degradation curves. For this research, these parameters have been averaged, and others have been assumed. However, an attempt for validation with real data has been presented.

The validation of the model has shown a low correlation whether it is considered that the same model has been validated previously through accelerated carbonation tests. Although it is an expected result, the analysis has concluded that the model has better reliability to determine the carbonation curves in sheltered structural elements from the weathering. Nonetheless, it should be noted that the results of the validation must be carefully analysed. That is, the values applied as input values in the model comprise average values of the real data whose standard deviation could influence the result obtained in the coefficient of determination.

In order to deal with the shortcoming mentioned above, the recommendable is to carry out the same analysis on an existing structure of which the precise data necessary to apply the numeric model of carbonation is available. In essence, the results of Chapter 4 have allowed determining that the structures of Paraguay are under a considerable degradation risk considering the expected carbonation depth under the effect of climate change. For this reason, it will be necessary to establish measures to deal with this degradation process through maintenance strategies. These strategies were formulated in the next chapter.

Chapter 5 comprises the main result of this thesis. The maintenance model has been structured in two processes. Firstly, the formulation of the optimal inspection times and the most appropriate inspection technique is proposed. Then, the maintenance planning considering the inspections together with the repairs is analysed. It should be emphasised that the main focus of these strategies has been preventive maintenance, whose reasons for this choice have already been established in the chapter.

For the first stage of the maintenance model, the significance of performing an efficiency analysis in the planning of inspections that is complemented by the optimisation process has been demonstrated. Although the efficiency analysis is mostly associated with studies in the area of the economy, this research has shown its applicability in maintenance management. The results of the analysis show that it is possible to use a smaller number of resources (inspection techniques) and a lower total time in the inspections planning to achieve the same or better results than those given only by the mathematical optimisation of the problem. This avoids the waste of resources that can collaborate indirectly with the reduction of the environmental impact associated with the construction sector and the sustainability of infrastructures.

For the second stage has been formulated a dynamic decision-making model. This model is considered dynamic since through the result obtained after its application; it is possible to determine the best alternative concerning the inspection technique and the repair method for any time over the service life of the structure. Likewise, this adaptability allows the model to be associated with the dynamic behaviour expected by the future effect of climate change on the degradation of structures. This model can be considered as an extension of the traditional AHP method. The new approach of the model addresses the uncertainty of the degradation process through the formulation of stochastic indexes that are used in the weight assignment of criteria. This approach leaves aside the subjectivity associated with the methodology proposed in the traditional AHP method, which allows having greater reliability on the results obtained with this method.

The main advantage of this decision model is that the decision maker gets to know in advance and in a simple way what are the resources (inspection techniques and repair methods) with which he needs to have to perform the intervention in the structure. Likewise, the repair in the structure is associated with the minimum damage degree detected during the inspection, which is a parameter that is also defined by the decision maker. In this way, once the input parameters required by the proposed model are established, the results obtained allow establishing the best maintenance strategy in the structure considering the expected damage rate throughout its service life. According to the results of a numerical example, it has been demonstrated that the application of this maintenance model collaborates in the reduction of the total life-cycle cost of the structure which is achieved through preventive maintenance.

The proposed decision model is partly based on mathematical formulations found in the literature. However, in the expression regarding the detectability of the inspection techniques, a problem of discontinuity in the probability function has been found. To address this problem, the probability at the limit points has been considered as polynomials of cubic splines to fulfil the monotonicity and concavity at the boundary lines. The results after the application of the proposed equation have shown that with the new expression allows representing the probability function more closely to reality.

Lastly, although the context of the study developed in this research is focused to the corrosion-induced degradation in concrete structures, it should be highlighted that the proposed decision model may be applied to other degradation mechanisms and other typology of structure to formulate the best maintenance strategy. Nevertheless, the parameters considered as the input data must be set regarding each degradation conditions. Furthermore, in this research, the cost analysis has been established according to referential cost that is an analysis method commonly adopted in other studies. This enables to perform the analysis to any construction market throughout the world.

6.3 Suggestions for further Research

This research has been limited by the scope foreseen at the beginning of this thesis and by the time established for the completion of the doctoral program. For this reason, some aspects related to this research topic can be deepened and developed in future work. Below are described some proposed recommendations regarding this topic.

Regarding the calibration of the numerical model used in Chapter 4, it is recommended that new interventions would be made in existing buildings periodically to generate an extensive database for real cases. This database could be used not only for the calibration of numerical models of carbonation but also for other degradation mechanisms. The importance of validating numerical models with real cases is associated with the need to formulate models that are more reliable and allow obtaining results that can be applied directly in real case studies.

Moreover, further research is necessary to improve the correlation of the model for existing structures considering their weathering exposure. It should be mentioned that the carbonation model considered in this thesis does not consider this aspect. Several studies have shown that there is a higher carbonation depth in concrete elements sheltered from weathering than in the unsheltered ones. New considerations regarding the effect of relative humidity in both indoor and outdoor environments on the diffusion coefficient could be made to consider the difference in carbonation depth between the concrete elements with both exposure conditions.

The proposed maintenance model establishes a stochastic analysis that is determined by the probability functions. However, from a strictly mathematical perspective, the numeric model is not entirely probabilistic. That is, certain parameters such as detectability or the probability of failure are determined stochastically by considering variables randomly with a given probabilistic distribution. However, when applying these functions within the model, they are established as deterministic and not probabilistic values. For this, it is necessary to perform a more exhaustive analysis about the oscillation of the values of these parameters over time to address the uncertainty of real cases in an even more accurate way. Then, through new probability theories and concepts such as the Bayesian update, the results of the model could be even more dynamic.

The parameters established for the inspection techniques and repair methods were assumed in this investigation based on other studies. However, it is necessary to carry out a detailed survey that validates these values since the results obtained depend directly on the capabilities of these techniques and methods. It is advisable to elaborate a report after each inspection and repair to be able to update the values of these parameters. That is, the reliability of these techniques must first be validated in conjunction with laboratory studies to determine the detectability of the inspection techniques and the real efficiency of the repair methods.

To determine the maintenance activities, the proposed model considers the damage degree in the structure to intervene before the presence of the structural failure, i.e., preventive maintenance. In existing structures, the carbonation depth commonly varies between elements of the same structure and even between different parts of the same component. This leads to a different level of corrosion damage in the structures that could be considered in the maintenance planning. Nonetheless, the maintenance model proposed in this research does not consider the spatial variation of the structural damage. These considerations could be introduced as an extension of the proposed model through spatial variability theories and Gaussian random fields in the degradation process. Consider the spatial variation of the damage in the formulation of maintenance strategies could be even cheaper and have better results from a perspective of structural reliability.

Finally, considering the increasing tendency of the computerisation of the activities related to the construction, the decision model proposed in this thesis can be linked through the programming to a Building Information Modelling (BIM) tool. In this way, the model can be converted into a support tool that allows defining, from the design phase of the construction project, the necessary maintenance activities in the structure over its whole service life. Actually, an informatics tool for the dynamic decision model presented in this thesis is already being under development to facilitate its application for the decision makers in the construction sector.

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