Universidade do Minho
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Risk-based railway infrastructure management systems

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Cofinanciado por:

STATEMENT OF INTEGRITY

I hereby declare having conducted this academic work with integrity. I confirm that I have not used plagiarism or any form of undue use of information or falsification of results along the process leading to its elaboration.

I further declare that I have fully acknowledged the Code of Ethical Conduct of the University of Minho.

Resumo

A gestão de ativos de infraestruturas envolve diversos processos relacionados com o seu ciclo de vida nomeadamente na avaliação do estado de conservação, modelação da degradação, manutenção e otimização. Entre os vários ativos que compõem a rede ferroviária, as pontes são ativos valiosos para a rede ferroviária, uma vez que permitem cruzar determinados obstáculos tais como rios, vales, entre outros. No entanto, esses tipos de estruturas estão expostos a várias ameaças que podem causar degradação severa a longo prazo ou a curto prazo em caso de eventos extremos. Dessa forma, a falta de manutenção pode resultar em grandes consequências indesejáveis tanto para a estrutura, colapso parcial ou total do sistema, como para as pessoas.

Além disso, diversos estudos têm sido propostos por diferentes autores relativo a análise pós-recuperação após um evento extremo por meio do cálculo da resiliência. A resiliência representa a capacidade da infraestrutura de se recuperar de eventos extremos e representa um indicador valioso para estimar as dimensões das consequências.

O principal objetivo desta investigação é contribuir para uma evolução da prática atual de monitorização de infraestruturas em Portugal, através da proposta de uma metodologia de gestão baseada no risco e na resiliência. A metodologia desenvolvida inclui a estimativa de modelos de degradação que contemplam a degradação devido a eventos como corrosão e eventos como terremotos, inundações, entre outros. Além disso, a metodologia desenvolvida sugere diferentes planos de manutenção para um determinado período do ciclo de vida. Algoritmos genéticos são aqui adotados para otimizar problemas com dois objetivos conflituosos entre si, desempenho e custos. Estes conceitos são validados individualmente através dos capítulos desta tese e validados no capítulo 7 num estudo de caso de uma ponte ferroviária de betão armado localizada em Portugal.

Palavras chave

Sistemas de Gestão de Pontes, Risco, Resiliência, Degradação, Eventos Extremos, Otimização multiobjetivo, Algoritmos genéticos.

Abstract

Asset management of infrastructures involves several processes related with its life cycle namely on the assessment of the condition state, degradation modelling, maintenance, and optimization. Among the several assets that compose the railway network, bridges are valuable assets for the rail network by providing cross critical links such as waterways, valleys, and other types of facilities. However, these types of structures are exposed to several threats that can cause severe degradation at long term or at short term in case of hazard events. In this way, the lack of maintenance can result in large undesirable consequences either for the structure, such as partial or total collapse of the system, or for the people.

Moreover, several studies have been proposed by different authors concerning the post recovery analysis after a hazard event through the calculation of the resilience. Resilience stands for the ability of the infrastructure to recovery from hazard events and represents a valuable indicator to estimate the dimensions of the consequences.

The main goal of this research is to contribute to an evolution of the current practice of infrastructure monitoring in Portugal by proposing a management methodology based on the risk and on the resilience.

The developed framework includes the estimation of degradation models that captures degradation due to events such as corrosion and events such as earthquakes, floods, among others. Moreover, the developed methodology is capable of suggesting different maintenance plans for a certain life cycle period. Genetic Algorithms are here employed to optimize problems with two different conflicting objectives, performance and costs.

These concepts are validated individually through the chapters of this thesis and validated in the chapter 7 in a case study of a reinforced concrete railway bridge located in Portugal.

Keywords

Bridge management systems, Risk, Resilience, Degradation, Hazard events, Multiobjective optimization, Genetic Algorithms.

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List of Abbreviations

AASHTO - American association of state highways transportation officials
AI - Artificial Intelligence
AM - Asset Management
ANN - Artificial Neural Network
BMS - Bridge Management System
BMS - Bridge Management Systems
BNs - Bayesian Networks
CBR - Case-Based Reasoning
COST - European Cooperation in Science and Technology
CPT - Conditional Probability Table
DBN - Dynamic Bayesian Network
DLS - Damage Limit State
DMV - Decision Making Vector
EA - Evolutionary Algorithm
EDP - Engineering Demanding Parameter
FE - Finite Element
FHWA - Federal Highway Administration
fib - international federation for structural concrete
FORM - First-Order Reliability method
GA- Genetic Algorithm
GOA - Gestão de Obras de Arte
HMM - Hidden Markov Models
IABMAS - International Association for Bridge Maintenance and Safety
IABSE - International Association for Bridge and Structural Engineering
IM - Intensity Measure
IP - Infraestruturas de Portugal
IPMA - Portuguese Institute for Sea and Atmosphere
ISO - International Organization for Standardization
JCSS - Joint Committee on Structural Safety
KPI - Key Performance Indicator
LCA - Life Cycle Assessment
LCAC - Life Cycle Agency Costs
LCSA - Life Cycle Social Assessment
LHS - Latin Hypercube Sampling
LM71 - Load Model 71
LSF - Limit State Function
M&R - Maintenance and Rehabilitation
MAB - Masonry Arch Bridge
MC - Markov Chain
MCS - Monte Carlo Simulation
MLE - Maximum Likelihood Estimation
MOEA - Multiobjective Evolucionary Algorithms
MOP - Multiobjective Optimization
NCHRP - National Cooperative Highway Research Programs
NPV - Net Present Value

NSGA II - Non-dominated-sorting genetic algorithm II
PDF - probability distribution function
PDV - Peak Ground Velocity
PG - Performance Goal
PGA - Peak Ground Acceleration
PGD - Peak Ground Displacement
PI - Performance Indicator
PMC - Probabilistic Model Code
PMS - Pavement Management Systems
PN - Petri Net
R&D - Research and Development
RC - Reinforcing Concrete
RV - Random Variable
SAMCO - Structural Assessment, Monitoring and Control
SHM - Structural Health Monitoring
SLS - Serviceability Limit State
SORM - Second-Order Reliability method
TD - Triangular Distribution
TPM - Transition Probability Matrix
TQI - Track Quality Index
ULS - Ultimate Limit State

Chapter 1

1 Introduction

1.1 Background and motivation

The transport network is extremely important to the social-economic development of a country. Particularly, the railway transport has been revealing crucial to a sustainable growth of the society. Railways, in their present form, made their first appearance at the beginning of the 19th century in British mines [1]. The golden age of the railways occurred during the first industrial revolution where it was introduced several developments such as the introduction of steam, and the extensive exploitation of coal and iron mines. The first railway lines operating in Europe began around 1830 being attained its maximum density at 20th century. In Portugal, according to Souza [2] the main development of railway infrastructures occurred between 1850 and 1870 with the construction of the railway lines in center and south of Portugal. With the First World War, the railway entities became economically weakened being extremely necessary a support from the state. Later, in the 50's, with the constant increasing of the population, the railway transport vision changed, in which it was abandoned the exclusive transport of goods being thus turned into a general transport of people and goods. In this way, there were created new programs for the management in the railway infrastructures for the new level of demand by the society. However, with the developments in road infrastructures, and with the large advantages that the road transport presented when comparing with the rail transport regarding door-to-door transport, higher comfort and flexibility led the users, in general, to opt by having their personal car. To overcome these problems, several investments among all the Europe, have been implemented in the field of railway infrastructures so the competitiveness can be assured. One of those investments was under Shift²Rail program, which aimed to contribute to: (i) cutting the life-cycle cost of railway transport; (ii) doubling the railway capacity and (iii) increase reliability and punctuality by as much as 50%. Also, other research projects, further discussed in chapter 2, have been financed by the European Commission on the assessment of existing railway infrastructures. Among railway infrastructures, bridges compose a crucial asset on the railways by providing critical links to cross waterways, valleys among other types of facilities. For example, in Portugal, the main companies responsible for the management of infrastructure are *Infraestruturas de Portugal* (IP), Brisa and Ascendi.

Moreover, during their life-cycle, bridges are exposed to natural hazards that can seriously compromise their functionality. In fact, several failures of bridges have been occurring over the last decades being the most common failures due to scour, floods, impact of vehicles and overloading. Because of these events, large amount of consequences either in terms of human lives or in terms of economy have been recorded. In this way, the engineering community has been dedicating some efforts at understanding bridge failures and to developing methodologies to provide a reliable behavior of the structure.

Moreover, the importance of knowing the process of degradation of a bridge is very critical to understand when the bridge should need maintenance actions. As the available budget is limited, the needs of establishing management systems to provide optimal maintenance schedules to the stakeholders become an important matter.

In Portugal, the actual management system comprises the following objectives [3]:

- Assure the safety of the structures, maintaining them at the level of capacity predicted in design.
- Assure that the railway traffic is done without restrictions and under the conditions of comfort and velocity predicted.
- Maintain the data records of bridges updated, organising the information to plan and optimize the interventions, minimising costs, and interferences with the circulation.

1.2 Research scope and objectives

The life-cycle analysis of bridges has been a topic widely studied for several researchers through these last decades in which some of them are worth it to mention. Neves and Frangopol [4] proposed a model that considered the uncertainties in the performance deterioration process, times of application of maintenance actions, and in the effects of maintenance actions on the condition, safety, and life-cycle cost of structures by defining all parameters involved in the model as random variables. Later, Okasha and Frangopol[5] proposed a computational framework including lifecycle performance prediction, maintenance optimization and updating lifecycle performance using structure health monitoring (SHM) data and controlled testing results. The framework was implemented in several roadway bridges. Du and Karoumi [6] presented a work in the field of life cycle assessment (LCA) of railway bridges by proposing a guideline to quantify the environmental burdens for the railway bridge structures. A comparison case study between two alternative designs of the Bridge under study was carried out through the whole life cycle, with the consideration of several key maintenance and end-of-life scenarios. Nielsen et al. [7] proposed a framework for life cycle management that consisted on the assessment, maintenance optimization of concrete and steel railway bridge in Australia. Safi et al. [8] introduced the Swedish bridge and tunnel management system following by an implementation life-cycle cost scheme. The study presented different alternatives for the final life cycle cost with the inclusion of several variables. Freire et al. [9] proposed a management system for bearing devices integrated in a bridge. The framework included a database module, inspection a decision making. Almeida et al [10] proposed a life cycle cost estimation and cost minimization through an optimization algorithm. The framework was composed of the degradation model, cost models and optimization. The costs models were divided into direct and indirect costs. Fernando et al. [11] proposed a model for the evaluation of intervention strategies based on progressive damage and hazard events. The model was based on the Markov theory. Denysiuk et al. [12] aimed to propose a computation framework applied to a highway bridge to help the project managers in the process of decision making. The analysis was conducted in an element level and a structural level. Yianni et al. [13] focused their study on the development of a Petri net as a degradation model based on the Network Rail company database to present a new degradation model as response to the limitations of the Markov chains. Xie et al [14] proposed a life time reliability-based maintenance analysis considering life cycle costs and life cycle environmental impact. The results were shown through optimization procedure with an implementation of an optimal schedule. Griffin and Patro [15] presented a paper describing the current approach regarding the railway bridge assessment and the innovative work that has been carried out to improve the efficiency of the assessment. Fernandes et al.[16] presented a framework for railway bridge maintenance affected by the corrosion effects. The proposed framework considered a multiobjective optimization by considering the reliability and costs of maintenance as the two objectives. Later, Fernandes et al. [17] presented a similar approach but based on risk analysis instead.

By analysing all these studies developed over the last two decades, it can be stated that a great amount of work has been conducted in the field of bridge management and decision making. However, digging into the literature, very few studies address issues of bridges being subjected to both progressive and hazard events. Here, progressive events stand for events such as reinforcing corrosion whereas hazard events stand events that suddenly force to bridge to an immediate repair or its closure, in case of very serious damages. Examples of studies that address these topics can be seen in [18-21].

Likewise, another very important issue that remains still a challenge in the bridge management field stands for the post hazard event analysis, wherein questions like how much time the system takes to recover to its original functionality as well as which consequences are involved, are crucial to fully comprehend the dimension of the problem. These questions have been answered by studies as referenced in [22-24].

Nevertheless, another issue pointed out relies on the lack of studies regarding the railway bridge management. Most of the railway works are more concerned to safety assessment, dynamic tests, and interaction between rail track and train rather than about life cycle analysis. Accounting all these challenges pointed out, the present thesis proposes a methodology for a bridge management system (BMS) applied to railway bridges through their lifecycle, which combines progressive degradation with hazard events. Several key performance indicators (KPIs) will be estimated with special focus on the estimation of the risk that the infrastructure is subjected during its life-cycle due to the several hazard events. Note that, although the focus of this thesis relies more on railway bridges, the proposed methodology is broadly applied to other types of infrastructures.

1.3 Thesis Outline

Following the current chapter, the chapter 2 is dedicated to the overview of the current infrastructure management systems applied worldwide as well as the most relevant research and development project, among the infrastructure management topic, developed ever since. Special focus is after given to the bridge management systems.

The chapter 3 focuses on the issues of assessment of existing bridges wherein it is discussed the current practices adopted on the assessment of bridges within the most common Key Performance Indicators (KPIs). The chapter is finalized with three practical applications on bridges composed of different typologies and materials. Special focus is given to the reliability, robustness, and risk KPIs.

The chapter 4 aims to address the topic related to the degradation modelling. It starts by conducting a thorough literature review among the most relevant degradation models applied in the field of infrastructures. After the literature review, two different types of models, mechanistic and stochastic, are applied to two practical applications.

The chapter 5 addresses to the life-cycle of infrastructures by presenting the literature review on the three main aspects that composes the sustainability concept, economy, social and environment. Moreover, an overview of the optimization algorithms is conducted with special focus on multi objective optimization algorithms based on genetic algorithms. To finish the chapter, two practical applications are conducted, the first to a single bridge, and the second to a hypothetical network to highlight the differences in terms of involved costs on the maintenance.

The chapter 6 is dedicated to the hazard analysis of bridges by conducting a literature review on the most common hazards that occur on a bridge. Moreover, a literature review on the resilience concept is conducted to discuss the issues related to the recovery of a system in the post hazard event. This concept is after applied to the same two practical applications presented in the chapter 5 with the inclusion of a possible hazard occurrence.

The chapter 7 aims to present a risk and resilience-based methodology on a railway bridge subjected to multiple hazards, a seismic hazard and corrosion effect over the time. A 2D Finite Element (FE) model for the bridge is developed using the software TNO DIANA.

To finalize, the chapter 8 concludes the thesis by drawing the final remarks as well as some future directions for further works to be developed.

Chapter 2

2 Infrastructure Management Systems

Asset Management (AM) is a multidisciplinary task that involves a lot of processes related to life cycle analysis, maintenance, risk analysis and optimization [25, 26]. As a formal approach to propose guidelines on the field of AM, the International Standard Organization (ISO) released in 2014 the ISO 55000 series. These series were composed of three documents: (i) ISO 55000 – Asset Management Overview, Principles and Terminology; (ii) ISO 55001 – Asset Management Systems: Requirements; and, (iii) ISO 55002 – Asset Management Systems: Guidelines for the application of ISO 55001. According to ISO 55000 [27], AM can be defined as a *"co-ordinated activity of an organization to realize value from assets"*. There are a wide range of definitions of AM depending on the field of evaluation. Infrastructure management systems (IMS) is the combination of engineering, management, financial and economic practices, applied over the life cycle of infrastructure systems, to provide an adequate level of service for users in the most cost-efficient way [28]. In order to have an integral and comprehensive life-cycle analysis, it is fundamental to assure the functionality of all the modules that, integrated on the system, will allow to carry out all the activities relating on managing the network of infrastructures. Normally, those modules are: (i) database; (ii) degradation and cost models, (iii) optimization models and (iv) updating modelling.

The database is considered the heart of the IMS. Normally this database is composed by an inventory, with information regarding the infrastructure ID, location, construction year, drawings and other documents, inspection component that stores all the information about inspections and reports that can include the condition state. Also, the database can compose the component regarding the maintenance actions which will inform the owner about the type of maintenance carried out in that period [29].

The degradation and cost models comprise another module of an IMS. They are supported on the database, where it is given all the historical information concerning the condition of the structure as well as the maintenance actions that will determine the performance of the models.

The optimization is considered the brain of the system since it consists in the development of an algorithm that defines the best strategies to apply based on the information obtained by the degradation models. Generally, the outcome is based on multi-objective problems where normally two conflicting objectives are defined (e.g., cost and condition rate) and multiple solutions are obtained giving to the project manager a set of optimal options to be considered on the decision-making process.

The update models aim to gather all the information obtained by the three previous components and to modify them through techniques that may be mathematical in nature. One of the most well-known techniques is the Bayesian inference. In field of the IMS, pavement and bridges are currently the infrastructures with more applications. Pavements are one of the most important road infrastructures wherein several pavement management systems (PMS) were developed to establish forecasting models as well as maintenance schedules to keep the infrastructure safe. Great part of the PMS employs as forecasting models the Markovian chain theory and regression analysis [30]. Examples of PMS real applications concerns to HDM-4, a PMS system applied in more than one hundred of countries [31]. The analytic framework of HDM-4 is applied for a life cycle analysis of 15-40 years which analyse four different modules [32]: (i) Road deterioration, concerning the prediction of the model; (ii) work effects, i.e. the maintenance activities and their effects on the road as well as the corresponding costs; (iii) road user effects, regarding vehicle operation costs, costs of travel time and costs to the economy of road accidents and (iv) social and environmental effects, i.e. vehicles emissions and noise.

Regarding railway IMS, there are already presented some studies such as in [33], wherein it is adopted the concept of artificial intelligence, through case based reasoning, to solve current problems based on similar previous problems. Furthermore, in [34] it is highlighted the importance of maintenance of railway IMS by proposing a condition-based maintenance chain composed of the following steps: (i) monitoring; (ii) analysis; (iii) warning/alerts generation; (iv) planning; (v) optimization; (vi) scheduling and execution and (vii) management.

Another field less explored, but already with some investigation, concerns the building management systems. For example in [35], the authors stated that although there is not a significant amount of investigation on this field, there were already some examples of expert knowledge systems focused on specific construction elements such as flat roofs, waterproof membranes, industrial floor epoxy coatings, wall ceramic tiling, wall gypsum plasters, gypsum plasterboard walls, wall nature stone claddings and wood floorings. Other different IMS concern the water waste management systems, see [36], electricity and gas distribution, see [37-39].

The following section states an overview of the most important research projects developed in the field of IMS, followed by an overview of the most important bridge management systems worldwide.

2.1 Research projects over the world

Due to a wide range in thematic of infrastructure management, a large amount research and development (R&D) projects over the world have been developed, namely in order to disclose, standardize and generalize the best research works and practices in these type of structures [40]. In the following section, it will be mentioned some of these works in the USA and Europe.

In the USA, one of the research programs was the NCHRP – National Cooperative Highway Research Programs which was developed presenting reports in the following areas such as (i) life-cycle cost analysis [41], multi-objective optimization [42] and decision-making process [43].

Also, in Europe, several R&D projects were developed, namely on roads, rail track, tunnels, railway and roadway bridges, among others, see Figure 2-1.

From Figure 2-1, it can be seen that several projects since 1998 were developed. BRIME - Bridge Management in Europe [44], the oldest in the field of highway infrastructures, had the main goal of cost-benefit analysis and structural reliability. Later, COST 345 [45] was focused on bridges, tunnels and retaining walls with the objective of assessing the costs of maintenance, inspection, and rebuilding. LIFECON - Life Cycle Management System [46] dedicated its work on bridges and tunnels. DARTS - Durable and Reliable Tunnel Structures [47] aimed to analyze the life cycle analysis on tunnels by performing structural assessment under hazard events such as fires, chloride ingress, and carbonation. SAMARIS - Sustainable and Advanced Materials for Road Infrastructures [48] focused on road and tunnel infrastructures giving their main contribution on the improvement of maintenance of highway infrastructures and implementation of innovative techniques to increase the performance of infrastructures. SAMCO - Structural Assessment, Monitoring and Control [49] aimed to investigate life cycle management and develop guidelines on structural health monitoring on structures such as buildings, power plants, and industries. Sustainable bridges [50], in the field of railway infrastructures, aimed to conduct an investigation on the increase of the load carrying capacity of bridges and the speed in the European rail network. Innotrack – Innovative Track Systems [51], presented in the field of the railway track an investigation work aiming at decreasing the life-cycle costs (30% until 2020) and increase the Reliability, Availability, Maintenance, Safety (RAMS). DuratiNet – Durable Transportation Infrastructures in Atlantic Space [52] focused their studies on the durability of transport infrastructures and deterioration of infrastructures near the sea. ETSI – Bridge Life Cycle Optimization [53] as well as COST TU1406 - Quality specifications for roadway bridges, standardization at a European level (BridgeSpec) [54] conducted their research works on roadway bridges with focus on life cycle assessment of bridges. Still on bridges, SBRI - Sustainable steel-composite bridges in built environment [55] developed a project of investigation for steel bridges focusing on life cycle cost and life cycle assessment. MAINLINE - Maintenance, renewal and improvement of rail transport infrastructure to reduce Economic and environmental impacts [56] investigated management tools to assess the environmental and economic impacts on its life-cycle and new technologies to extend life-cycle for railway infrastructures. SustIMS - Sustainable

Infrastructure Management System [57] developed an optimization tool that could integrate all the assets of a highway. INFRALERT – Liner infrastructure efficiency improvement by automated learning and optimized predictive maintenance techniques [58] focused its work on both road and railway infrastructure on the field of optimization and decision making in maintenance planning. Concerning transport network infrastructures, AM4INFRA - Common Framework for a European Life Cycle based Asset Management Approach for transport infrastructure networks [59] aimed to provide an easy comprehensive multi-modal management of mobility needs and expectations to the agencies by developing a framework for life cycle and risk-based Asset Management. SAFEWAY is the most recent project among IMS, currently on going, wherein the objective is to design, validate and implement strategies, tools and technical interventions to significantly increase the resilience of the transport infrastructure by reducing risk vulnerability and strengthening network systems to extreme events. More detailed information about the working program as well as the deliverables can be consulted in [60].

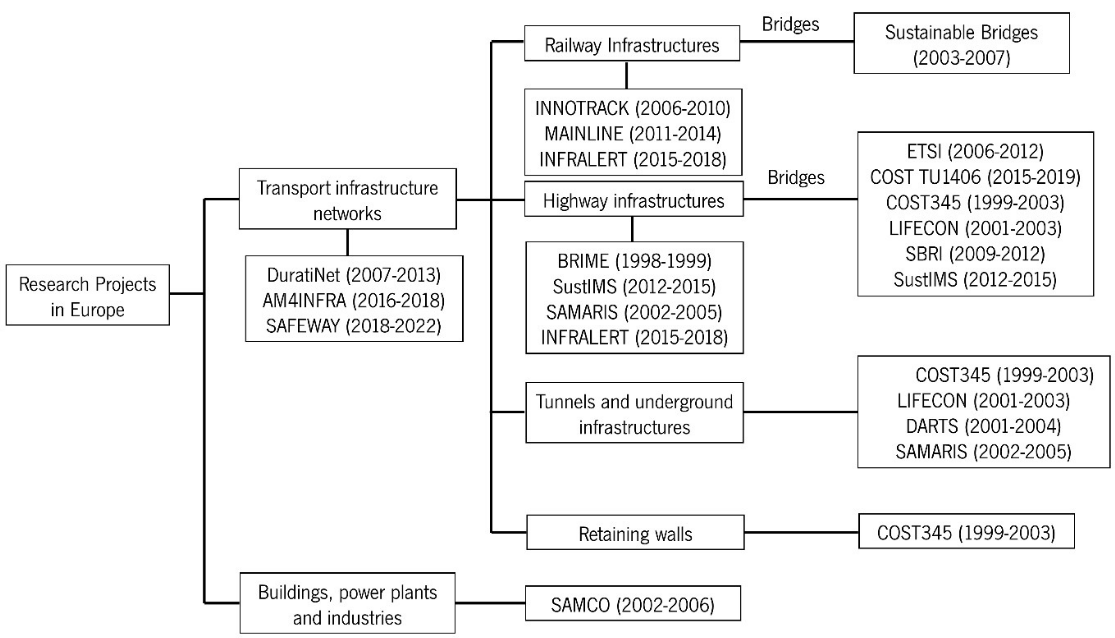


Figure 2-1 – Research projects in Europe in the field of assessment and management of infrastructures

2.2 Bridge management systems

Bridges are vital to the social economic development of a country since they are essential components that provide crossings at critical locations, that otherwise could add a significant amount of time travel [43]. Although, due to several processes of degradation, these infrastructures needed to be constantly monitored and maintained. Life cycle management of bridges comprises several activities during their life cycle including design, operation, and maintenance which will help to maintain the integrity of the bridge [7]. According to [29], the main stages of a life cycle for a bridge are depicted Figure 2-2:

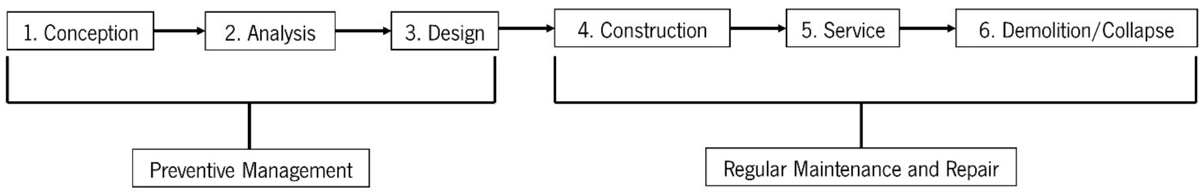


Figure 2-2 - Main stages of a life-cycle of a bridge (adapted from [29])

2.2.1 BMS over the countries

In a general way, infrastructures are vital to the growth of a society and to the well-being of the people. Therefore, it should be constantly managed by implementing procedures and practices to ensure an optimal strategy of intervention that can maximize its benefit. Each country presents their own strategies of management by developing their BMS. For example, in the USA, PONTIS and BRIDGIT were the developed BMS. PONTIS, now denominated as AASHTOWare, is one of the most well-known and complete BMS in the entire world. PONTIS has the capacity of "*express the engineering concerns of deterioration and structural performance in economic terms understandable to a broader audience*" [61]. BRIDGIT was a BMS released by NCRHP and considered ideal for smaller departments of transportation. This BMS *"facilitates the organization of bridge data, provides clear, accurate and timely reporting, can rank bridge populations by a number of user-specified criteria, allows the identification of critically deficient structures, and facilitates the tracking of deterioration trends and repair performance"* [62]. BRIDGIT elements are classified into seven categories: deck, superstructures, piers, abutments, joints, railing, and bearings. The main difference upon these two BMS developed in the US rely on the optimization module. Whilst BRIDGIT adopts a "bottom-up" approach, PONTIS goes for a "top-down" approach. In Switzerland, KUBA[63] is the road structure management system developed for the Swiss Federal Roads Authority. It is currently divided in four components, a data collection system, a management system, a special transport assessment system and a reporting system. More details about their functionality can be seen in [63]. In [64] it is reviewed several management systems applied in different locations of Canada. Examples of BMS in Canada are Alberta BMS, Ontario BMS and Quebec BMS. In Japan, Miyamoto et al. [65] presented a practical application of a BMS in which it is evaluated the performance of bridges offering also a rehabilitation strategy based in a combination of two performance indicators, cost minimization, and quality maximization. The framework developed was composed of: (i) inspection data; (ii) evaluation of the performance indicators such as load-carrying capacity and durability being ranked in a scale of 0-100 and then divided into five groups from 0-19, 20-39, 40-

59, 60-79, 80-100 in which the first group was the most dangerous and the fifth the safest; (iii) deterioration prediction in which the deterioration curves for load-carrying and durability were defined as biquadratic and cubic functions respectively; (iv) cost and effect on maintenance and (v) optimization of rehabilitation strategy by applying genetic algorithms (GA) to solve the multi-objective problem. In South Africa, Nordengen et al. [66] described the development of a BMS in Pretoria by the Center for Scientific and Industrial Research in partnership with Stewart Scott International. The system was composed of five modules: (i) Inventory; (ii) Condition; (iii) Maintenance; (iv) Budget and (v) Inspection. The inventory was composed of bridges and culverts. Inspection module contained detailed information about the structures regarding summaries of inspection, ratings, inspection photos and reports. The condition module was composed of weighting factors in order to prioritize repair/rehabilitation based on Degree (D), which ranged from 1 to 4 being 0 with no defect, Extent (E), from 1-local to 4-general, and Relevancy (R), from 1-minimum to 4-critical. These weighting factors were used to distinguish between structural and non-structural members. The budget module served to estimate the costs associated with each repair. The optimization was proceeded by using the cost ratio per defect and the limited budget per year. Repairs were allocated based on an Urgency ratio (U). In the UK, Network Rail manages all the railway infrastructures. The main goal of the asset management program is the improvement of the rail network capacity by reducing the maintenance costs. The areas of work cover on the asset optimization, asset information, risk management and the consideration of several optimization schedules to establish an equilibrium between the availability of the system and efficiency of work. The objectives of this management program have established a quality control plan with a duration of 5 years. The current quality control plan period started in 2015 with the main goal of performing maintenance based on risk [67]. Currently, the Network Rail management is composed of the database, where all the records of the structures are stored, the forecasting module, where the predictive model adopted is based on the Markovian approach, the intervention module, and the life-cycle cost module. In Portugal, The BMS GOA (Gestão de Obras de Arte) is currently used by several Portuguese companies. This system has the following objectives[68]: (i) to inventory all data related to all structures of the network; (ii) to register inspection results in order to define maintenance actions; (iii) to budget the costs of maintenance actions; (iv) to define a schedule of interventions depending on the available budget. In Denmark, DANBRO was the BMS implemented since the 1980s [69]. The DANBRO system is mainly organized by the inventory, inspection, optimization, and rehabilitation works, long term budget, price catalogue and heavy transport administration. The bridges are divided into fifteen elements where maintenance actions can be performed manually. The condition state of the bridge goes from 0 to 5 being 0 the best and 5 the worst condition [69]. Moreover, in 2008, The International Association for Bridge Maintenance and Safety (IABMAS) developed a questionnaire of several BMSs around the world [70].

Since then, three editions have been published in 2010, 2012 and 2014. The last report accounted with questionnaires on 25 BMSs from 18 countries. This report made a review of the several features adopted in the BMS namely: (i) used performance indicators; (ii) condition state measure; (iii) degradation models adopted per each BMS; (iv) planning frame time; (v) cost estimation. In annex A, a resume of the features of the several BMSs regarding the performance indicators used, prediction information and cost information are presented, based on IABMAS report.

Chapter 3

3 Assessment of existing bridges

As structures are ageing, there is an extreme need to assess them. The main reason concerns the load-carrying capacity. Because the structure deteriorates during its life-cycle and volume traffic has been increasing over the years, most of the structures, namely bridges, are submitted to higher loads than those from which they were designed. The safety evaluation of structures is dependent of many factors to assess its condition state. Since the absolute safety of structures cannot be reached at all, because of the many sources of uncertainty, there is a need of applying a probabilistic approach instead of a determinist approach. One of the main drawbacks in the assessment of existing structures is the fact that almost all codes are based on the design of new structures. Therefore, a certain structure could be classified as unsafe when in fact the structure is safe if a probabilistic approach is applied. Nowadays, in USA and Canada there are already probabilistic guidelines for the assessment of existing structures [71, 72]. In Europe, it can be mentioned the British guidelines [73, 74], the Swiss guideline [75] and the Danish guideline [76]. O'Connor [77] presents a paper where is pointed out the saved costs for adopting a probabilistic approach. Hence, it comes up a question such as: why do not apply these methodologies for the design of new structures? In fact, in spite of the benefits of a detailed approach, when compared to the associated costs of designing a new bridge, they are relatively low and do not justify its complexity [78, 79]. Structural Assessment and Monitoring Control (SAMCO) [80], an European funded research and development project, proposed different levels of structural assessment with an increasing of complexity, see Figure 3-1. The simplest methods are those based on partial coefficients, i.e. the semi-probabilistic approach used in codes[71, 81]. In this approach, all the materials are defined accounting with their class and using the characteristic and design values. The corresponding loads are obtained in a similar way to the materials. The load effects are obtained considering a linear-elastic, and for each element of the structure, a plastic behavior of materials is adopted[82].

Concerning the intermediate levels, additional information can be considered on the materials through some laboratory experiences and incorporated in the models through Bayesian inference. Concerning the loads, deterministic values are adopted, but based on real data.

The most advanced level combines the non-linear analysis with probabilistic analysis. All the mechanical parameters of the materials and the loads are considered as random variables with corresponding probabilistic laws.

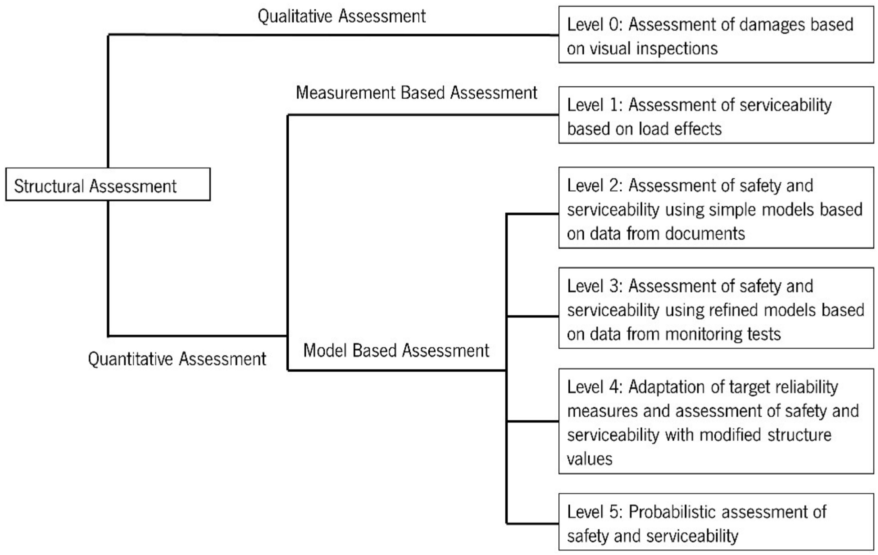


Figure 3-1 – Levels of assessment of structures (adapted from [80])

However, within the last years, significant research has been developed worldwide regarding the condition assessment of structures, namely by non-destructive tests, visual inspection techniques and Structural Health Monitoring (SHM). The later revealed to be a very important technique since it involves a wide range of activities which allow obtaining information about the actual performance of existing structures. COST TU1402 – "*Quantifying the value of Structural Health Monitoring*" was developed to assess the value of SHM and quantify it to improve the decision basis for the life-cycle management.

On these days, there are several ways of evaluating a structure condition. More recently, the concept of performance indicators was introduced, simplifying the communication between consultants, operators, and owners. Concerning bridge infrastructures, Strauss et al [83] proposed a categorization of the performance indicators (PIs) and interactions between the performance goals (PGs), that were crucial for optimal bridge management, by defining three levels. The first level was at the component level wherein the PIs are evaluated in terms of damages to specific components of a bridge. The second level concerned to the system level by evaluating the bridge condition, reliability, safety, and its functionality. The third and most complex concerned to the network level by evaluating the importance of the bridges to the system of infrastructures and the way they can influence the performance of the other assets. The literature presents a wide range of PIs that, combined with other PIs, can result in a key performance indicators (KPIs). These KPIs can be classified as qualitative and quantitative. Qualitative KPIs are represented as an arbitrary scale using linguistic terms such as high, moderate, low, and very low. Furthermore, they can be presented in a form of condition index being defined by a scale of integer values.

These type of KPIs are the most adopted across Europe on visual inspections [84]. However, the structural condition and safety levels are not explicitly accounted and thus resulting on the fact that discrete stochastic transitions between condition state may fail to account the actual structural performance [85]. Quantitative indicators are more applied to assess physical characteristics of a structure. They are very useful to measure since they provide more detailed information about ultimate, serviceability and fatigue limit states. Their major drawback accounts on the variety of quantitative indicators that the literature presents to assess the condition state of the infrastructure being thus difficult to state an universal measure to estimate its actual condition state. In the following sections, some of the most applied quantitative KPIs in the literature are hereafter discussed.

3.1 Reliability of structures

Reliability is a KPI widely studied in the field of structures and used to perform analysis under uncertainties. The main goal of this PI is to identify the failure probability of the system. The failure probability is defined as a violation of a limit state. Generally, that limit state is given by a Z function which is the difference between resistance demand (R) and load effects (S) being defined as:

$$\mathrm{Z} = \mathrm{R} - \mathrm{S} \tag{3-1}$$

According with Melchers [86], the probability of failure can be obtained by a convolution integral defined as follows:

$$p_f = P(Z \leq 0) = \int_0^{\infty} F_R(q).f_S(q)dq \cong \phi(-\beta) \tag{3-2}$$

where F_R is the cumulative resistance distribution function, f_S the probability density function of the load effect, ϕ is the standard normal distribution function and β the reliability index. The analytical determination of the integral in equation (3-2) is only possible for very simple cases. The most common case is for R and S as two independent variables with a mean of μ_R and μ_S and variances of σ_R^2 and σ_S^2. Considering the safety margin in equation $\mathrm{Z} = \mathrm{R} - \mathrm{S}$ and the addictive properties, the mean and variance of Z is given by:

$$\mu_z = \mu_R - \mu_S \tag{3-3}$$

$$\sigma_Z^2 = \sigma_R^2 + \sigma_S^2 \tag{3-4}$$

Replacing on the equation (3-2), the p_f is given by:

$$p_f = P(R - S \leq 0) = P(Z \leq 0) = \Phi(\frac{0-\mu_Z}{\sigma_Z}) \tag{3-5}$$

where Φ is the standard normal distribution function. Considering the equations (3-3), (3-4) and (3-5), the probability of failure is given by:

$$p_f = \Phi\left[\frac{-(\mu_R - \mu_S)}{\sqrt{\sigma_R^2 + \sigma_S^2}}\right] = \Phi(-\beta) \tag{3-6}$$

wherein β is given as:

$$\beta = \frac{\mu_Z}{\sigma_Z} = \frac{\mu_R - \mu_S}{\sqrt{\sigma_R^2 + \sigma_S^2}} \tag{3-7}$$

The obtained equation (3-7) is the reliability index proposed by Cornell [78, 87]. However, the Cornell formulation for the reliability index calculation does not solve the invariance problem once it depends on the approximation made for a non-linear Limit State function. Later, in 1974, Hasofer Lind proposed the calculation of the reliability index wherein the limit state function was transformed into a normal space allowing thus to overcome the invariance problem. Basically, this method aimed to find the design point, through an iterative process, that was given by the shortest distance between the origin and the failure surface. For a deeper explanation of the formulation see [87]. Another approximated analytical methods described in the literature refer to First-Order Reliability method (FORM) and Second-Order Reliability method (SORM), see [86-89]. These problems are solved by applying the following steps: (i) transformation of the variables into a standardized space; (ii) approximation by Taylor series expansion and (iii) obtain the design point which is given geometrically by the shortest distance to its origin.

3.1.1 Simulation Methods

Regarding the simulation methods, they present a high relevance when the problem is non-linear. Their greatest drawback relies on the computational time when assessing the probability of failure. There are two distinguished simulation methods: (i) pure simulation also known as crude Monte Carlo Simulation (MCS) and (ii) variance reduction techniques such as Latin Hypercube Sampling (LHS).

The MCS is an easy method to apply wherein the probability distribution functions (PDFs) of random variables are simulated by generating random values. The probability of failure is given by:

$$p_f = \frac{N_{g(X)\leq 0}}{N} \tag{3-8}$$

where $N_{g(X)\leq 0}$ is the number of simulations that represents the structural failure and N the total number of simulations. However, this method presents high computational cost when assessing the probability of failure, thereby being unpracticable on complex engineering problems. To overcome this drawback, variance reduction techniques were developed such as Latin Hypercube Sampling (LHS) [90, 91]. LHS is a variant of MCS that stratifies the sample in a way that the theoretical probability distribution function of the random variables is divided into m intervals with equal probability. This technique follows the steps: (i) divide the variable X_i into N intervals of equal probability; (ii) selection of representative values (e.g. middle points); (iii) combination by shuffling the vectors containing the representative values of each variable; (iv) evaluation of the limit state function for each combination and (v) estimation of the probability of failure. Despite the considerable reduction on the number of samples when comparing to the MCS, this method still comprises a considerable time computing.

To overcome the hassles provided by the simulation models, the literature proposes the definition of metamodeling approaches. Basically, the main idea is to replace the original limit state function by a meta model which is faster evaluated. One of the most common approaches relies on the polynomial response surface method. However, such approach will not be further discussed since it is out of the scope of the present thesis. Nevertheless, it can be consulted in [92-95].

3.1.2 Literature review on the application of reliability in civil engineering problems

In civil engineering, specifically on bridges, there are already applications among these approaches to solve reliability problems. Joan Casas et. al [82] presented on their work a simplified probability-based assessment for railway bridges aiming to be simple for the bridge evaluator, with a good accuracy and appropriate for the most common failure modes on bridges. This work carried out four different cases at the cross-section level. The same author also presented a methodology to calculate the reliability index of masonry arch bridges at the ultimate and serviceability limit state [96]. The non-linear analysis of the masonry bridge was performed by an LHS to obtain the reliability index at the ultimate limit state. Another interesting work reports to Nowak et al. [97] regarding an analysis of the reliability concrete bridges considering three different codes (Spain [98], Eurocode [99] and AASHTO [100]) aiming to compare the obtained results. The results have shown that the Eurocode is the most conservative being AASHTO the most permissive. The reliability indexes were calculated using FORM. Moreira et al. [101] proposed a probabilistic approach to obtain the reliability index of 5 masonry arch bridges from Portugal with different spans to assess their safety according to different codes. The reliability approach was obtained by applying LHS. Matos et al. [102] proposed a framework for structural probabilistic assessment composed of two main steps to compute the reliability index, the first wherein the numerical model was updated through a model identification procedure and the second wherein the deterministic model was converted to a probabilistic model. Each parameter was updated by Bayesian inference to give more accuracy to the results. The developed non-liner finite element model accounted with a sensitivity analysis and simulation techniques to compute de final reliability of the structure. Cavaco et al. [103] presented a framework to study the robustness of an existing reinforced concrete structure to define optimal and maintenance repair planning. The structural performance was computed by obtaining the reliability index combining FORM with response surface method. This work aimed to assess the corrosion on the reinforcement. Guimarães et al. [104] proposed an innovative framework based on the metamodeling techniques. His framework was validated and tested for several examples aiming to highlight the significant reduction of the number of simulations to obtain the reliability index when compared with other authors.

3.1.3 Target Reliability index

Besides the calculation of the reliability index, there are minimum values imposed by some standard codes. The target reliability index (β_{TARGET}), or threshold value, is defined as the minimum value that the structure should present to be considered safe. The optimum value of reliability index depends on the level of consequences when the limit state is violated and its cost. Since the cost of increasing the safety of an existing structure is higher than the cost of increasing the safety of structure during its construction phase, there are recommended different values for the reliability index [105]. The ISO 13822 [106] imposes three fundamental considerations to the establishment of minimum levels of structural performance: (i) economic considerations; (ii) social considerations and (iii) sustainability considerations. Table 3.1 presents the threshold values for several standards considering different reference periods and different degrees of consequence.

Table 3.1 – Threshold reliability values considering the different standards

Standard	Consequences			
	Very Low	**Low**	**Medium**	**High**
Reference Period : 1 year				
EN1990 [107]	-	4,2	4,7	5,2
ISO 2394 [108]	2,9	3,5	4,1	4,7
ISO 13822 [106]	-	-	-	-
PMC [109]	-	3,1	3,3	3,7
Reference Period : 50 years				
EN1990 [107]	-	3,3	3,8	4,3
ISO 2394 [108]	2,9	2,3	3,1	3,8
ISO 13822 [106]	2,3	3,1	3,8	4,3
PMC [109]	-	1,7	2,0	2,6

It is important to note that Table 3.1 defines the threshold values without considering the structural system (i.e., element level assessment) and non-linear considerations. On the other way, Fib standard [110] presents the threshold reliability values for a global level by considering the structural system and non-linearity, see Table 3.2.

Table 3.2 - Reliability index according with *fib* [110].

Safety Levels				
1	2	3	4	5
Reliability Index				
$\beta \geq 9{,}00$	$9{,}00 > \beta \geq 8{,}00$	$8{,}00 > \beta \geq 6{,}00$	$6{,}00 > \beta \geq 4{,}60$	$4{,}60 > \beta$
Qualitative Assessment				
Excellent	Very Good	Good	Satisfactory	Insufficient

3.2 Risk Assessment

Risk is a worldwide measure adopted in several different fields other than engineering. According with the book *Fundamentals of Risk Management* [111], risk is defined as "*An event with the ability to impact (inhibit, enhance or cause doubt about) the mission, strategy, projects, routine operations, objectives, core processes, key dependencies and/or the delivery of stakeholder expectations".* Moreover, in the same book, different organizations propose their own definition according with their own purposes, see Table 3.3.

Table 3.3 - Definition of risk according with different entities[111]

Organization	**Definition of Risk**
ISO Guide 73 ISO 31000	Effect of uncertainty on objectives. Note that an effect may be positive, negative, or a deviation from the expected. Also, risk is often described by an event, a change in circumstances or a consequences.
Institute of Risk Management (IRM)	Risk is the combination of the probability of an event and its consequence. Consequences can range from positive to negative.
Orange Book from HM Treasury	Uncertainty of outcome, within a range of exposure, arising from a combination of the impact and the probability of potential events.
Institute of Internal Auditors	The uncertainty of an event occurring that could have an impact on the achievement of the objectives. Risk is measured in terms of consequences and likelihood.

3.2.1 Risk Assessment in civil engineering

Risk is considered a more complex measure than reliability because it assembles the structural performance of the bridge at a probabilistic level with indicators related to the development of the society that can differ according to the region. In the works of Faber and Stewart [112] and Ellingwood [113] a comprehensive analysis of the risk indicator as well as its importance to facilitate a risk-informed assessment is discussed. Moreover, within the work of Faber and Stewart [112], a generic representation of the flow of the risk-based analysis is discussed, wherein the main stages are addressed to: (i) System representation; (ii) Exposures and hazards and (iii) Consequences. Its representation is also thoroughly explained in Joint Committee on Structural Safety (JCSS) [114]. Due to its complexity, risk has been recognized by the bridge engineering community as a topic of high interest as the literature can prove. For example, Daniel Imhof [115] proposed in his PhD thesis a risk assessment of existing bridges wherein it was suggested five risk indicators: (1) current safety; (2) future safety; (3) warning level; (4) condition; (5) importance. Adel Al-Wezzer [116] presented in his PhD thesis a risk-based bridge maintenance strategies in which a five-step methodology is proposed: (1) system definition; (2) system breakdown; (3) element-level modelling; (4) risk assessment; (5) risk management. Padgett et al. [117] proposed a cost benefit analysis due to seismic event by applying retrofit measures on non-seismic design bridges. Decò and Frangopol [18, 118] investigated a risk assessment of a bridge under multiple hazards such as abnormal traffic loads, environmental attacks, scour and earthquakes that were considered for a computation of time-dependent reliability, hazards function and survivor function. Zhu and Frangopol [119] studied issues related to deterioration through the performance indicators of reliability, redundancy and risk. An event-tree model was applied to assess the direct, indirect, and total risk associated with the failure of a component due to corrosion and traffic loads. The same authors [20] proposed a risk-based optimal maintenance of a bridge subjected to multi-hazard scenarios, traffic and earthquake loads, to analyze different failure modes. Kameshwar and Padgett [21] proposed a parameterized fragility assessment based on multi hazard risk assessment to assess bridges subjected to earthquakes and hurricanes. Their framework covered three stages: (i) Characterization of multiple hazards; (ii) Assessment of a parameterized multi-hazard fragility and (iii) Evaluation of the multi-hazard risk. Saydam et. al [120] proposed a five-state Markov model to predict the time-dependent performance of bridges. In this methodology, the direct consequences were computed according to the transitions to out-of-service states. Indirect consequences were computed considering the state probability in out-of-service states. The same author [121, 122] presented a reliability-based approach to component and system failure probabilities. Here, the risk was quantified in terms of expected direct and indirect losses on a component failure. Barone and Frangopol [123] presented two approaches to solve maintenance problems: (i) annual reliability-index and annual risk performance indicators; (ii) lifetime functions such as availability and hazard-based. The results showed for the first approach that reliability-index and risk were correlated once they were associated with the same system failure mode. However, lifetime functions were associated with extreme cases and there was no need of correlation between these two variables once they control different aspects of the problem. Dong and Frangopol [124] studied the fatigue crack growth on the deck considering risk updating based on the inspection of the crack size. The results were presented at the element and system level and with different levels of correlation effects. Zhu and Frangopol [125] considered risk analysis considering partial and full closed lanes on a bridge subjected to traffic loading and scour. Their proposed framework took into consideration six stages: (i) hazard identification; (ii) identification of lane closure scenarios; (iii) annual failure probabilities of structural components; (iv) occurrence of the probabilities of the identified scenarios; (v) estimation of the consequences for the different scenarios and (vi) risk assessment caused by the lane closure. Zanini et al. [126] proposed a fully time dependent probabilistic framework for seismic risk assessment of bridges with an application to a case study of 500 bridges. Yilmaz et al. [127] proposed a methodology for the assessment of the risk of the bridge subject to both earthquake and flood scenarios. Mondoro and Frangopol,[128] conducted a study of a bridge applying risk analysis due to multiple extreme events, floods, hurricanes, and tsunamis. Several failure modes and their respective consequences were studied at the deck, pier, and foundation level. This study was important to assess the effectiveness of management strategies.

3.2.2 Risk quantification

As for the field of structural analysis, a definition of risk considers a probability of occurrence of an event (likelihood) and the corresponding consequences as follows:

$$Risk = p_f \times C_f \tag{3-9}$$

where p_f is the failure probability and C_f the associated consequences. The consequences estimation can be direct, i.e. the direct damage on the infrastructure, and indirect, i.e. the damages surrounding the infrastructure system. Hence, the calculation of the risk can be divided into direct and indirect risk and re-written as follows [129]:

$$R_{Dir} = P(E) \times P(D|E) \times P(\bar{C}|D) \times C_{Dir} \tag{3-10}$$

$$R_{Ind} = P(E) \times P(D|E) \times P(C|D) \times C_{Ind} \tag{3-11}$$

where C_{Dir} and C_{Ind} are the direct and indirect consequences, respectively, $P(E)$ is the probability of occurrence of an event susceptible of causing any damage, $P(D|E)$ is the conditional probability of the structure being damaged considering it is subjected to the exposure, E and $P(C|D)$ is the collapse probability conditional on damage, D.

Considering a multi-hazard scenario and different type of damages, the equations mentioned above can be re-written as:

$$R_{Dir} = \int_x \int_y C_{Dir} \,.\, f_{D|E}(y|x) f_E(x)\, dy\, dx \tag{3-12}$$

$$R_{Ind} = \int_x \int_y C_{Ind} \,.\, P(C|D = y) f_{D|E}(y|x) f_E(x)\, dy\, dx \tag{3-13}$$

where $f_{D|E}$ is the probability of damage conditional on a given exposure, f_E is the exposure probability density function and $P(C|D = y)$ is the failure probability given a certain damage.

3.3 Robustness of Structures

Robustness of structures has been recognized over the years as a theme of high interest due to the collapse of big structural systems wherein consequences were considered unacceptable concerning its initial damage. Over the years, several system failures have been occurring, such as: (1) Ronan Point Building in 1968, London; (2) New World Hotel in 1985, Singapura; (3) The Highland towers in 1993, Kuala Lumpur; (4) Sampoong Department Store in 1995, Seul; (5) Hintze Ribeiro Bridge in 2001, Portugal; (6) World Trade Center in 2001, New York; (7) The Torch Tower in 2015, Dubai; (8) The Adress Hotel in 2015, Dubai; (9) Morandi Bridge in 2018, Italy; among others. Structural robustness began to be seriously studied after the massive disaster of World Trade Centre collapse. Another reason for a renewed interest of robustness analysis were derived from failures due to unexpected loads, design errors, errors during execution, unforeseen deterioration and poor maintenance [130]. In this context, a workshop carried by JCSS in collaboration with International Association for Bridge and Structural Engineering (IABSE) at the Building Research Establishment in London, UK (December 2005) gathered 50 experts, from research institutions, companies, and government, to discuss issues related with robustness. The conclusions led to a consensus that the present situation about ensuring sufficient structural robustness through codes and standards was highly unsatisfactorily. Therefore, a joint European project in Robustness was created, the COST action TU06010 – Robustness of Structures.

In general, robustness can be defined as a capacity that a certain structure has to support a certain amount of damage without occurring global collapse. Starossek and Haberland [131] presented several definitions of robustness in civil engineering domain by different authors including the Eurocode documents EC 0 [107] and EC 1-7 [132], which stated that robustness is *"the ability of a structure to withstand events like fire, explosions, impact or the consequences of human error, without being damaged to an extent disproportionate to the original cause".* The authors also discussed several terms related with robustness, such as exposure, vulnerability, damage tolerance, redundancy, ductility, and reliability.

Furthermore, several works were proposed in the field of robustness and classified by different natures, deterministic, probabilistic and risk based. Regarding deterministic indicators, the study of Frangopol and Curley [133] proposed redundancy as performance indicator comparing the intact structure with damage. This indicator ranged from one to infinity presenting thus a drawback in terms on the difficulty on quantifying the safety of the structure. Starossek and Haberland [134] presented an assessment of robustness based on the progressive collapse dividing into impact and redistribution. The proposed performance indicators ranged from zero to one being zero the worst situation and one the best in terms of robustness. Biondini and Restelli [135] proposed a robustness measure associated to accidental actions. The performance indicator assessed was obtained by comparing a situation of a damaged and undamaged solution wherein the robustness varied from zero to one. Cavaco [136] proposed in his PhD thesis a quantification of robustness somehow hard to define, to measure and to introduce in standards once the author considers that robustness is dependent on the structure environment. According with this author, robustness was presented as a performance evaluator since it evaluated the variation of structural performance indicator under a certain damage spectrum. The robustness ranged from zero and one, for null and full robustness, respectively. Concerning probabilistic indicators, Frangopol and Curley and Fu and Frangopol [133, 137] proposed probabilistic indexes to measure structural redundancy concerning the probability of failure and reliability index. However, as the deterministic indicator above presented by the same author, the range varied from zero to infinity being uncertain to measure how robust the structure was. Lind [138] proposed a quantitative measure for vulnerability and damage tolerance to compare scenarios of damaged and undamaged structures. Ghosn and Moses [139] focused their work on bridges, wherein the redundancy was defined as the capacity to redistribute the applied loads after reaching the ultimate capacity of the members. To assure adequate bridge redundancy and system safety, the authors proposed the verification of four limit states: (i) member failure limit state; (ii) serviceability limit state; (iii) ultimate limit state; (iv) damaged condition limit state.

Concerning risk-based performance indicators, Baker et. al [129] proposed a risk-based robustness assessment, based on direct and indirect risk. This index takes values between zero and one wherein a very robust structure corresponds to the value of one and zero to a structure which is highly dependent of the indirect risk. Table 3.4 resumes all the approaches as well as the involved variables.

3.4 Strengths and weaknesses of the key performance indicators

After a review of several types of KPIs accounted to assess the performance of infrastructures in general, Table 3.5 presents their strengths and weakness. The next section aims apply those different presented KPIs and provide a practical and insightful discussion on three case studies: (i) simply supported reinforced concrete (RC) bridge; (ii) steel bridge and (iii) masonry arch bridge. Those three case studies are located in different regions and different networks.

Table 3.4 – Robustness measures according to different authors

Authors, date	Measure	Description
Frangopol and Curley (1987)	$R = \frac{L_{intact}}{L_{intact} - L_{damaged}}$	L_{intact} and $L_{damaged}$ are the undamaged and damaged measures, respectively
Starossek and Haberland (2011)	$R_d = 1 - \frac{p}{p_{lim}}$ (Based on damage tolerance)	R_d is the robustness index, p is the damage extension; p_{lim} is the degree of acceptable progressive damage.
	$R_d = 1 - 2.\int_0^1 [d(i) - i]\, di$ (Based on redundancy)	$d(i)$ the maximum total damage resulting from and including the initial damage of extent *i*.
Biondini and Restelli (2008)	$Rob(\delta) = \frac{f(\delta = 0)}{f(\delta)}$	$Rob(\delta)$ is the robustness measure, $f(\delta = 0)$ and $f(\delta)$ are the undamaged and damaged measure
Cavaco (2013)	$R = \int_{D=0}^{D=1} f(x)dx$	$f(x)$ is the structural performance
Frangopol and Curley (1987)	$RI = \frac{P_{f(damaged)} - P_{f(intact)}}{P_{f(intact)}}$	$P_{f(damaged)}$ and $P_{f(intact)}$ are the failure probability for damaged and intact scenario, respectively
Fu and Frangopol (1990)	$\beta_R = \frac{\beta_{intact}}{\beta_{intact} - \beta_{damaged}}$	β_{intact} and $\beta_{damaged}$ are the reliability index for an undamaged and damaged scenario, respectively
Lind (1995)	$V = \frac{P(r_0, S)}{P(r_d, S)}$	r_0 and r_d are the system resistance for intact and damaged conditions damaged conditions; $P(r, S)$ is the probability of failure of the system either as function of the loads and resistance
Ghosn and Moses (1998)	-	The method is based on verification of four different limit states and then the redundancy is calculated by the difference between structure and the member level
Baker et al. (2008)	$I_{rob} = \frac{R_{Dir}}{R_{Dir} + R_{Ind}}$	R_{Dir} is the direct risk and R_{Ind} is the indirect risk.

Table 3.5 - Strengths and Weakness of the KPIs

Type of KPI	Measure	Strengths	Weaknesses
Qualitative	Condition State	✓ Most adopted across Europe [84]; ✓ Easy scale to implement on a system classification as well as it is useful for the decision-making process [40].	✓ Not accurate enough about the structural performance [84]; ✓ Each system adopts their own scale [70].
Quantitative	Reliability	✓ There is already great amount of literature on how to obtain such indicator as well as guidelines and codes; ✓ It enables to address problems such as ultimate, serviceability and fatigue analysis at analytical level [140, 141]; ✓ It provides a new level of assessment that can provide to be better at the economic level when compared with the partial safety factor analysis [142]; ✓ This indicator involves probabilistic analysis and combined with non-linear analysis can reflect more accuracy on the real behavior of the structure [143];	✓ Most of the cases, the probability of failure is not easy to obtain [104]; ✓ Among non-linear application, it can reveal problems regarding the computational time especially when applying MC simulation [144]; ✓ A great number of uncertainties involved [86].
	Robustness	✓ Great amount of research already on this topic in the field of bridges; ✓ A good measure to assess the redundancy of the structure when subjected to abnormal loads that affects the overall performance [145].	✓ Several different formulations by the authors with different scales to measure as well as several definitions [145]; ✓ Dependency of several parameters (Risk-based analysis).
	Risk	✓ Already some guidelines as well as several research studies developed to solve the problem of risk analysis; ✓ Very complete measure of assessment the condition state of the infrastructure and the surrounding environment.	✓ The calculation of the risk indicator involves the calculation of direct and indirect consequences which can involve proabilitiy density functions that can present a complex problem [129]; ✓ Risk analysis are based on information which can be uncertain or incomplete [112].

3.5 Case A – Simply supported RC bridge

The simply supported RC railway bridge is located in "Beira Baixa" line and inserted in a segment that links the cities of "Guarda" and "Covilhã", Portugal. This bridge was re-built in 2009 due some needs of extending its span. The whole system is divided into the infrastructure, which is subdivided mainly into the abutments, bearings, and foundations, and the superstructure, that is mainly composed of the beams and the slab, see Figure 3-2. This practical case is focused on the deck. The primary solution adopted for the deck was a composite cross section composed by steel profiles embedded within reinforced concrete. Due to the need of extending the bridge span and the typology of abutments, the final solution was a deck with a slab monolithically connected to two pre-stressed beams casted in situ. The deck is composed of two pre-stressed beams, each one with 0,90m width and 0,70m height and a slab with a mean thickness of 0,325m. In what concerns the pre-stress, a post-tensioned solution was adopted. The bridge is composed of one single span of 13.20m. Figure 3-2 and Figure 3-3, illustrate the bridge view and the cross section, respectively. The reinforcement for each beam is composed of 13Ø25, positive steel reinforcing, 4Ø20 and 6Ø16, negative steel reinforcement, and 40 cm^2 of pre-stressing steel, see Figure 3-4. Still concerning the deck, a longitudinal analysis will be conducted focusing the maximum flexural response at midspan.

Figure 3-2 – Railway bridge view

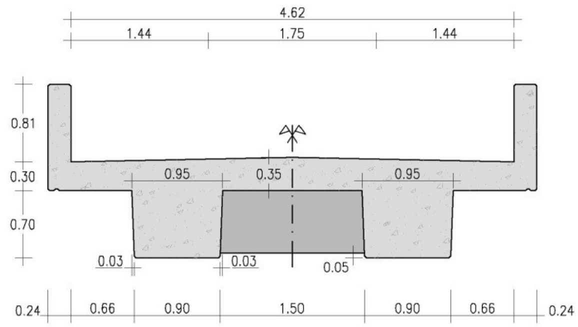
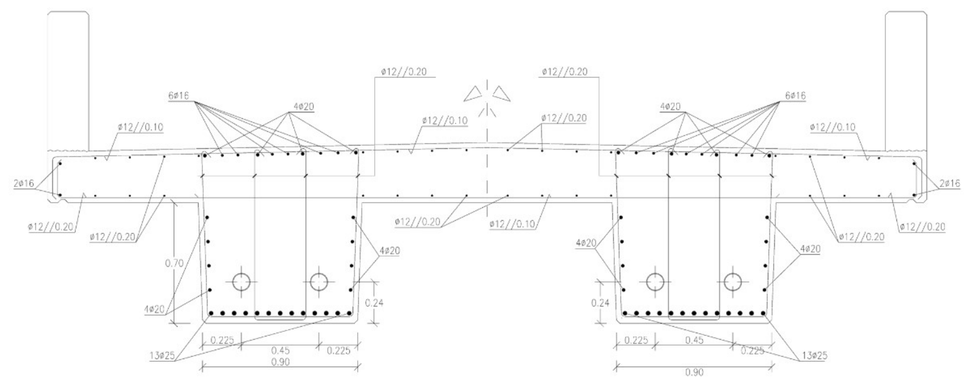
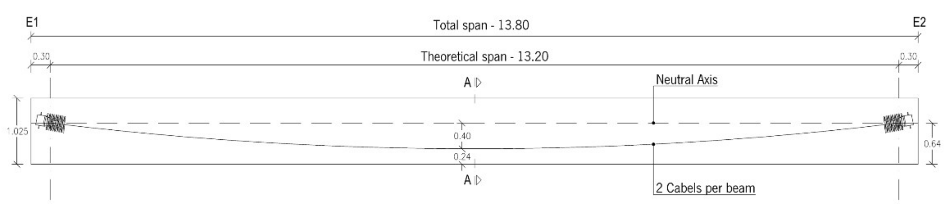


Figure 3-3 – Cross section of the deck



a)



b)

Figure 3-4 – Bridge Reinforcing: a) Reinforcing steel; b) Pre-stressing, dimensions in [m]

3.5.1 Reliability index calculation

The following section is dedicated to the calculation of the reliability index for the ultimate limit state at midspan at an initial age due to traffic loads. The limit state function adopted was given as stated in equation (3-1). Although a great part of real problems require numerical models to be solved due to its highly non-linearity, this problem was solved by an analytical expression once it dealt with a simply supported beam model. For this analysis, self-weight and additional permanent loads were considered as dead loads. Regarding the live loads, a load model provided by Portuguese code [146] was applied, see Table 3.7, being the mean values obtained from the characteristic values considering the percentile of

98% as suggested in [82]. Moreover, those loads were affected by a dynamic coefficient that according to the Portuguese code is given by:

$$\varphi = 1 + (\frac{2.16}{\sqrt{l}-0.2} - 0.27) \tag{3-14}$$

As for the resistance properties, their mean values and coefficient of variation were based on [109, 147, 148], see Table 3.6.

Table 3.6 - Considered variables for Reliability Calculation

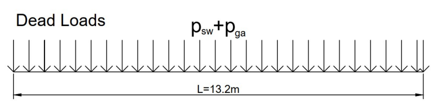
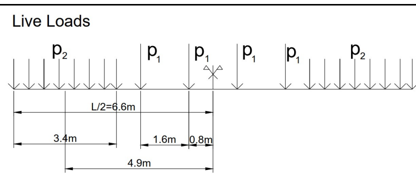
Description	Random Variable	Distr. Type	Mean Value	COV
Resistance	Steel yielding strength (f_{sy})	Normal	560 MPa	5.35%
	Reinforcing steel young modulus (E_s)	Normal	200 GPa	2.5%
	Prestressing steel young modulus (E_p)	Normal	195 GPa	2%
	Reinforcing steel Area (A_s)	Normal	Nominal	time-dependent
	Prestressing steel Area (A_p)	Normal	Nominal	2%
	Concrete compressive strength (f_{cm})	Normal	38 MPa	10%
	Neutral Axis depth (x)	Normal	0.26 m	5.5%
	Model uncertainties (θ)	Lognormal	1.2	15%
Loads	Self-weight (S_w)	Normal	72.94 kN/m	8%
	Additional dead loads (G_a)	Normal	33.4 kN/m	10%
	Load due the train (Q_{train})	Normal	p1=207.4 kN p2=63.4 kN/m	10%
	Centrifugal force (Q_{cent})	Normal	p1=74.76kN p2=23.86kN/m	10%

In this work, the reliability index was based on bending and resistance moment calculation, see Table 3.7. For the resistance, the flexural strength of the beam was assumed to be reached when the strain in the extreme compression fiber attained the maximum compressive strain (ε_{cu}=0.0035). Thereby, the strains in the reinforcing and prestressing steel were computed from a linear strain distribution by straight-line proportion. The limit state function (LSF) can be re-written as:

$$Z = \theta \times M_R - (M_{DL} + M_{Qtrain} + M_{Qcent}) \tag{3-15}$$

where M_R, M_{DL}, M_{Qtrain} and M_{Qcent} are respectively the resistance moment, moment due to dead loads, moment due to train load and moment due to centrifugal force. Note that the centrifugal force does not have a visible influence on this bridge since the railway line is practically straight. However, this force was considered in this case since the same was took into account on the design reports provided by the owners of the company. For sake of simplicity, equation (3-15) is shown in its compact form. Its computation was conducted through a FORM analysis. Furthermore, the reliability index value obtained at the initial age was 5.60. Comparing these values with Table 3.1 and Table 3.2, according with EN 1990 [107], the minimum value for the reliability index for a reference period of one year is 5.2 for the highest consequences. Once the obtained reliability index was greater than the threshold value, the bridge verified the safety at ultimate limit state. Concerning ISO 2394 [108], the threshold value is 4.7, leading to the conclusion that the bridge verified once again the safety regarding the ultimate limit state. Comparing the target reliability index of the PMC [109], also it was concluded that the safety of the bridge was satisfied at ultimate limit state. According with fib[110], the obtained reliability index is comprised between the values of 8 and 6, revealing that the bridge was in a good condition.

Table 3.7 – Bending Moment Calculation

		Load Scheme	**Moment Calculation (middle span)**
Dead loads	Self-weight Additional permanent loads		$M_{DL} = \frac{p_{sw} \times l^2}{8} + \frac{p_{Ga} \times l^2}{8}$
Live loads	Load model		$M_{Qtrain} = \left[(3.4 \times p_2 + 2 \times p_1) \times \frac{l}{2} - p_1 \times (1.6 + 0.8) - p_2 \times 3.4 \times 4.9\right] \times \varphi^*$
	Centrifugal force		$M_{Qcent} = 0.36 \times M_{Qtrain}$
Resistance demand	Reinforcing and pre-stressing steel	-	$M_r = f_{ys} \times A_s^+ \times d_s^+ + A_p \times d_p \times \left(E_p \times \left(\frac{\varepsilon_{cu}(d_{pi} - x)}{x}\right)\right) - (f_{cm} \times b \times \lambda^2 \times \frac{x^2}{2} - f_{ys} \times A_s^- \times d_s^-)$

* φ is the affecting dynamic coefficient of 1.36. This value was obtained according with Portuguese code

3.5.2 Risk Analysis

Following the formulation presented in section 3.2.2, the probability of failure, given a certain event, was obtained by the reliability index previous calculated in 3.5.1. Note that for this application the event was the traffic overloading. The direct consequences considered in this study were associated to the rebuilding of the RC deck. Regarding the indirect consequences, they can be of different nature like loss of human lives, injuries, and time loss due to detour of the vehicles. Since there was no information about human lives and injuries, this application focused on the time loss due to detour of the vehicles. The rebuilding costs (direct consequences) point to the replacement of the deck. According to the literature, see [18, 118], the rebuilding of the deck is given by:

$$C_{reb} = c_{reb} \times A_{reb} \tag{3-16}$$

where c_{reb} is the cost per square meter ($€/m^2$) and A_{reb} is the rebuilding area, i.e. the deck in m^2. As for the time loss (indirect consequences), alternative routes were defined for the vehicles crossing under the bridge and for the train during the time of rebuilding of the system. An alternative route for the vehicles is shown in Figure 3-5. The red line shows an alternative route to the users until the next train stop. The points marked with the letter A and B denotes the beginning and the ending of the detour, respectively and the red circle the location of the bridge. The detour route (km) and the detour costs due to the time loss (min) for this scenario can be expressed as follows [149]:

$$C_{detour} = DUR \times PER \times \sum_{v=1}^{2} TMD \times \left[CK \times (LD - LP) + CH \times \left(\frac{LD}{S_r} - \frac{LP}{S_n}\right)\right] \tag{3-17}$$

where DUR is the duration of the activity (days), PER is the conditioned traffic percentage, v is a variable that considers the calculation for cars (v=1) and for trucks (v=2), TMD is the average daily traffic, LD is the detour route (km), LP is the normal route (km), S_n is the normal speed (km/h) and S_r the restricted speed (km/h), CK is the unit cost per kilometer (€/km) and CH the unit cost per hour (€/h). Table 3.8 shows the quantification of the considered variables.

Legend:
Point A: Begin of the detour; Point B: End of the detour
Red Circle: Bridge Location; Black Line: Railway line; Red Line: Alternative detour

Figure 3-5 – Alternative detour map

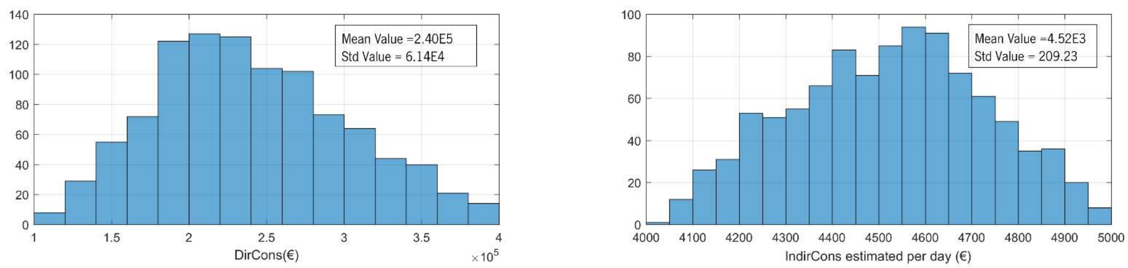
Table 3.8 - Consequence parameter estimation (parameters adapted from [149])

Description	Notation	Value	
Rebuilding of the deck (€/m²)	c_{reb}	TD* (680,1360,2550)	
Rebuilding area (m²)	A_{reb}	157**	
Conditioned traffic percentage	PER	TD* (80%-90%-100%)	
Average daily traffic	TMD	Cars	Trucks
		950	50
Cost per kilometer (€/km)	CK	0.18	0.68
Cost per hour (€/h)	CH	8.4	10.1
Normal Speed (km/h)	S_n	120**	
Restricted Speed for cars (km/h)	S_r	70**	50**
Detour route (km)	LD	18.108**	
Normal route (km)	LP	2.640**	

*TD stands for triangular distribution

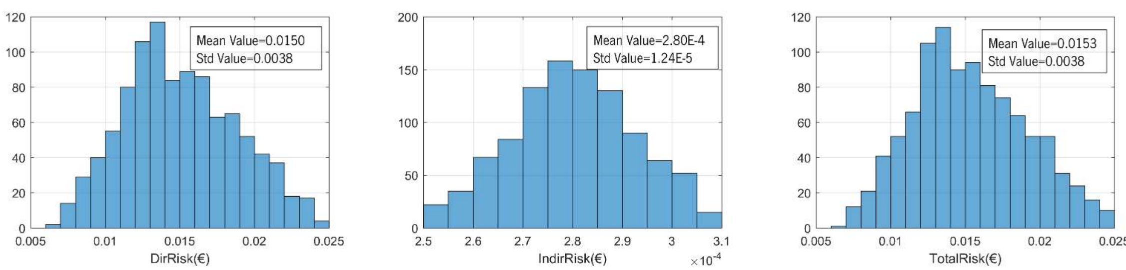
**Values measured for the present practical application

The duration of the maintenance activity (DUR) is related to the time the system takes to recover to its full capacity normally given in days. Such variable is an unknown parameter due the lack of information for the case. To tackle this problem, the presented results for the indirect consequences were presented per day of reconstruction of the deck. Note that the consequences were assumed to be probabilistic due to c_{reb} and *PER* variables. Thus, a triangular distribution was generated in MATLAB [150] considering 1000 samples. Considering the following assumptions for the consequences, Figure 3-6 shows the estimated direct and indirect consequences, respectively. Having estimated the consequences, the corresponding risk is depicted in Figure 3-7



a) b)

Figure 3-6 – Estimated consequences: a) direct; b)indirect



a) b) c)

Figure 3-7 . Risk estimation: a) Direct, b) Indirect c) Total

From Figure 3-7c), it was observed that the total risk to the RC deck was practically inexistent. This result was somehow expected given the high value for the obtained reliability and a corresponding very low probability of failure(6.183e-8). Despite of the initial risk value being very low, its exposure, i.e. bridge deck, to several threats such as environmental attacks, collisions of vehicles, can lead to a considerable increase on the risk metric. Furthermore, the obtained total risk shown in Figure 3-7 accounted with a scenario that the recovery of the system is occurred in 1 day. However, by increasing the number of days to RC deck takes to recover, the total risk indicator will increase its dependency on the indirect consequences. Next section aims to investigate the influence on those increases, i.e. on the indirect consequences by computing the risk-based robustness measure function of the recovery days.

3.5.3 Risk-based robustness indicator

As stated in the previous section, a risk-based robustness index will be estimated based on the formulation proposed in [129]. Recalling the measures presented in Table 3.4, the risk-based robustness is given as follows:

$$I_{rob} = \frac{R_{Dir}}{R_{Dir}+R_{Ind}} \tag{3-18}$$

where R_{Dir} and R_{Ind} are the direct and indirect risks, respectively. This index decreases as the indirect consequences increases. As mentioned in section 3.3, an index close to the value of 1 is very low dependent on the indirect consequences whereas value of 0 is highly dependent of the indirect consequences. To understand the daily growth of the indirect consequences, Figure 3-8 presents the time-dependent cumulative daily indirect consequences for 1 year.

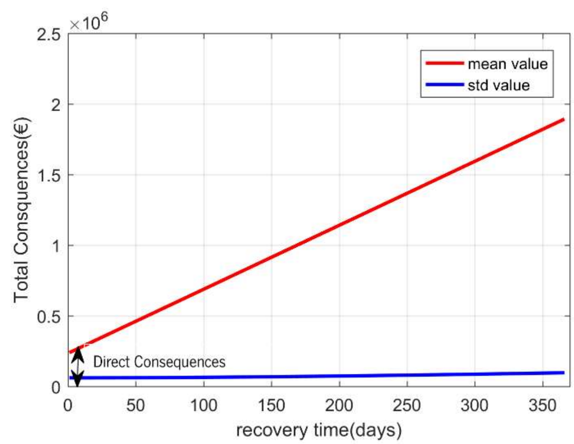


Figure 3-8 – Cumulative time-dependent indirect daily consequences

Considering this time dependency for the bespoke indirect consequences, the time dependent robustness index as well as the total risk are shown in Figure 3-9.

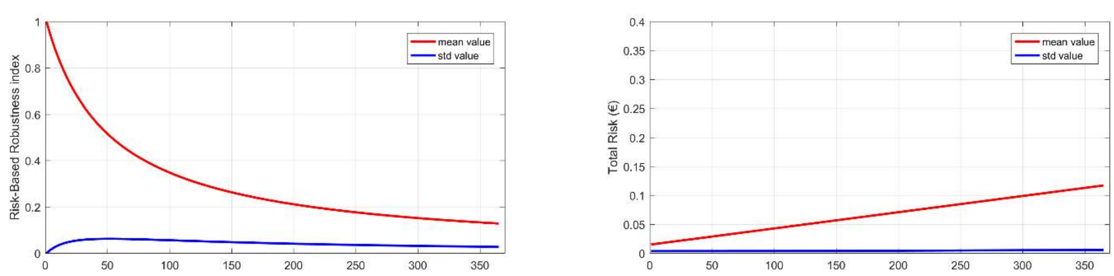


Figure 3-9 – Risk based robustness index and total risk

Considering the obtained results in Figure 3-9 it was concluded by the calculation of the risk-based robustness index that the structure decreased at the end of 1 year by 90% assuming that the recovery time was assumed to be one year. However, in the reality, assuming a recovery time of 1 year for a simply supported RC deck would be totally unrealistic. More realistic values for a problem like this, and considering there is a road crossing under the bridge, should be around 5 to 10 days of full recovery. That would led to a robustness index of almost 100%. Moreover, looking only for this indicator might be insufficient since the total risk involved was completely null. In this way, although the risk-based robustness allows to estimate a normalized result between 0 and 1 accordingly with the indirect risks for the system, it should not be forgotten the order of the risk under analysis. In this case, for the initial age, the RC deck presented very reduced values of risk. However, for the scenarios wherein the bridge is subjected to other hazards such as environmental attacks, earthquakes, bridge strikes, the risk could be much higher and carefully attention should be paid. Such scenarios will be carefully analyzed in furthers chapters.

3.6 Case B – Steel railway bridge

This railway bridge is located in Óbidos region, Portugal at the 99,805 km of “Linha Oeste” (Western Line) of the Portuguese rail network. The bridge connects Óbidos town with Caldas da Rainha city, crossing the Rio Arnóia. The bridge was originally built in 1886. However, due to the need to modernize the rail line, the bridge was renewed in 1990. The studied bridge is made of steel, see Figure 3-10, with a total length of 27.25m and a width of 5.3m. For this practical application, the analysis is focused on the superstructure.

Figure 3-10 – Bridge overview

3.6.1 Bridge geometry

The structural scheme adopted for this application was based on a truss bridge, see Figure 3-11. Given that the different trusses present a different geometry, for the sake of simplicity, different letters were attributed to each point. Note that the truss bridge is symmetric wherein the distance between adjacent points is 4.30m with a corresponding height of 6.2m. Table 3.9 and Figure 3-12 the geometry of the bars and the cross section dimensions, respectively.

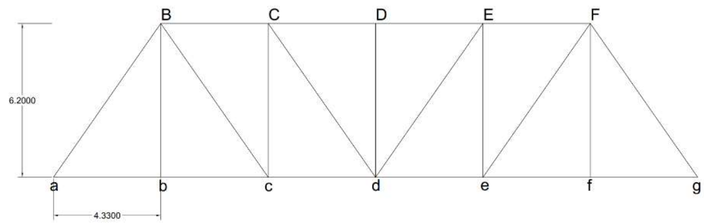
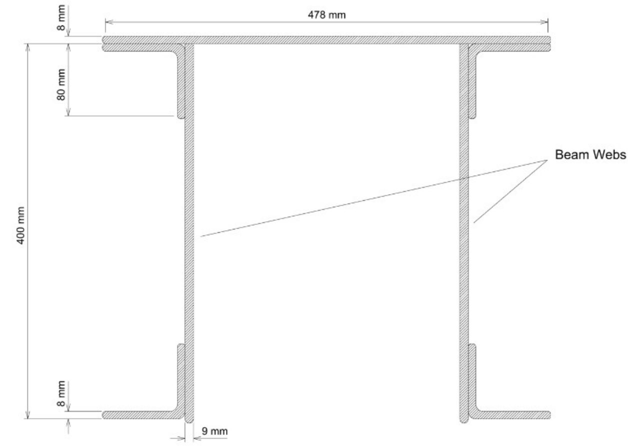


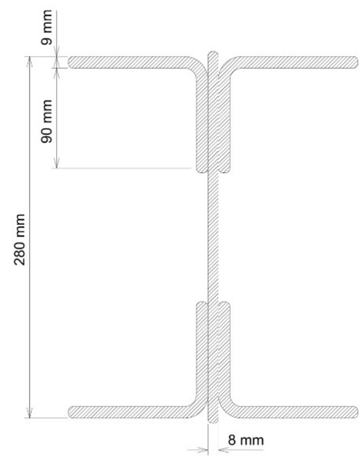
Figure 3-11 – Structural scheme adopted for the calculation

Table 3.9 – Geometry of the bars

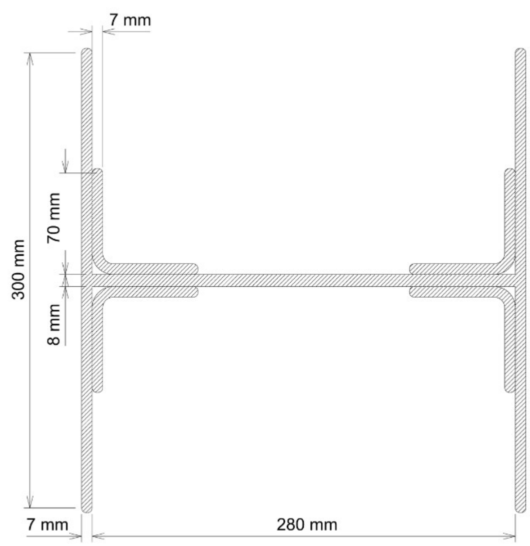
Bar	Composition	Area of the section (mm^2)	Total Area (mm^2)
[BC] [CD]	1 edge 478 x 8	3824	16144
	2 webs 400 x 9	7200	
	4L (8x80x80)	5120	
[bB] [cC] [dD]	1 web (280x8)	2240	8720
	4L (90x90x9)	6480	
[ab] [bc]	1 web (280x8)	2240	10360
	2 bars (300x7)	4200	
	4L (70x70x7)	3920	
[cd]	1 web (280x8)	2240	13360
	2 bars (400x9)	7200	
	4L (70x70x7)	3920	
[aB]	1 edge (500x7)	3500	17820
	2 webs (400x7)	5600	
	4L (80x80x8)	5120	
	2 bars (100x18)	3600	
[Bc] [Cd]	2 edges (454x8)	7264	15384
	2 webs(300x7)	4200	
	4L (70x70x7)	3920	


478 mm
8 mm
80 mm
400 mm
Beam Webs
8 mm
9 mm

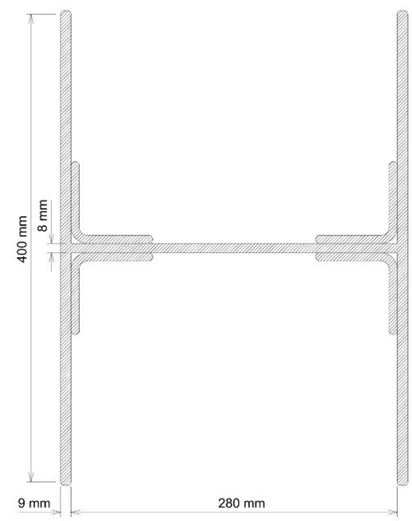
a)


9 mm
90 mm
280 mm
8 mm

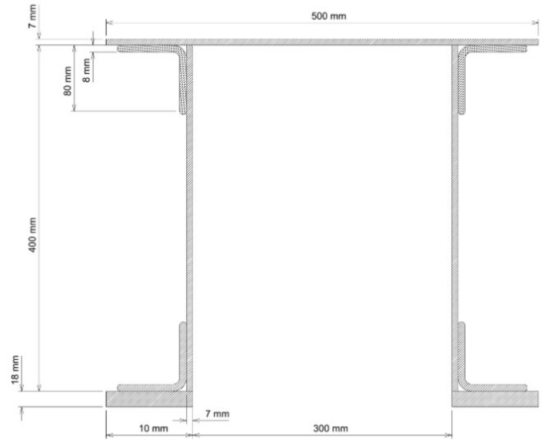
b)


7 mm
70 mm
300 mm
8 mm
7 mm
280 mm

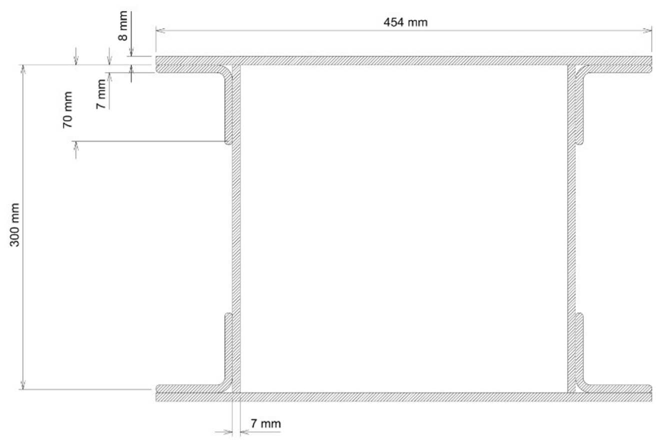
c)


400 mm
8 mm
9 mm
280 mm

d)



e)



f)

Figure 3-12 - Cross sections: a)[BC] and [CD]; b) [bB], [cC] and [dD]; c) [ab], [bc], d) [cd]; e)[aB]; f) [Bc] and [Cd]

3.6.2 Load modelling

For the present application, the load model to apply was based on the load model 71 (LM71) given by [151], see Figure 3-13. Note that the presented values were presented as characteristic values.

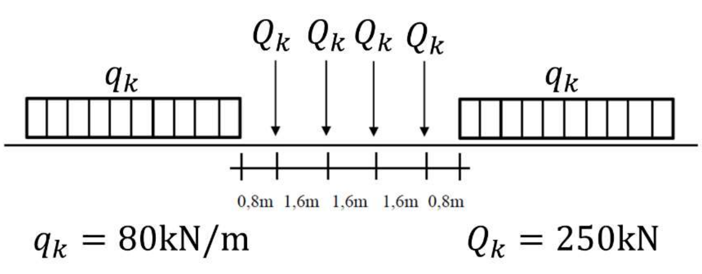


Figure 3-13 – LM71 (adapted from [151])

Moreover, the present application accounted with the dynamic effects. According with EN 1991-2 [151], the dynamic factor, ϕ_3, is given as follows:

$$\phi_3 = \frac{2.16}{\sqrt{L_\phi} - 0.2} + 0.73 \tag{3-19}$$

where L_ϕ is the determinant length, i.e., the span length L for simply supported bridges or L_ϕ for other bridge types, allowing this factor to be used for other structural members with different support conditions. Several determinant length values are provided in [151]. In this case (simply supported bridge), the determinant length was the bridge span. Hence, for this case the obtained dynamic factor was 1.16.

3.6.3 Structural Analysis

For the structural analysis of the truss bridge, to estimate the axial force in each bar, the software FTOOL [152] was used in here. Note that this software aims to estimate the axial forces due to the actions provided by the LM71 and its self-weight and therefore to proceed to the reliability index calculation. Figure 3-14 shows the adopted model. The axial forces are hereafter presented in Table 3.10.

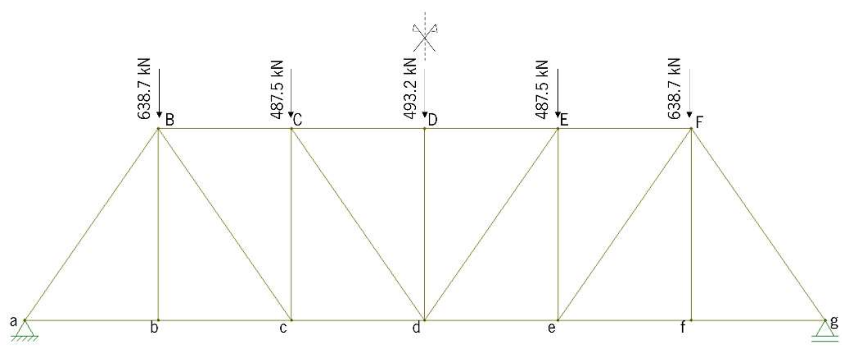


Figure 3-14 – FTOOL adopted model

Table 3.10 – Axial forces

Bar	Forces $[\mathrm{kN}]$	Bar	Forces $[\mathrm{kN}]$	Bar	Forces $[\mathrm{kN}]$
[aB]	-1675.0	[Bc]	895.7	[Cd]	301.0
[ab]	959.5	[BC]	-1472.6	[CD]	-1645.0
[bB]	0	[Cc]	-734.2	[Dd]	-493.2
[bc]	959.5	[cd]	1472.6	-	-

It was observed from the axial forces that some bars are submitted to compression leading therefore to possible buckling issues. In this way, to proper a quantification of the resistance of these bars, a coefficient

χ should be calculated and addressed to the resistance estimation of the bar. Next section is dedicated to the estimation of this coefficient for the compressed bars.

3.6.4 Buckling estimation coefficient on the resistance measure

The estimation of the buckling reduction coefficient, χ, followed the suggestions of EN 1993-1-1[153] given by the following steps depicted in Figure 3-15.

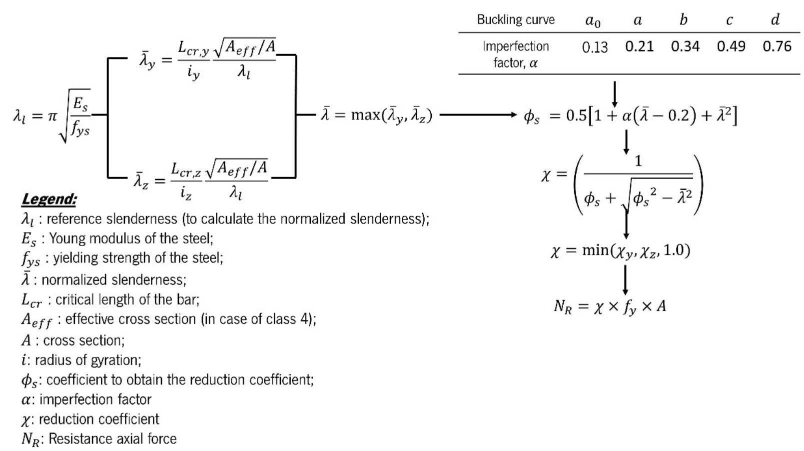


Figure 3-15 – Reduction coefficient calculation steps (based on [153])

Following the steps of the Figure 3-15, the reduction coefficients are presented in Table 3.11. The remaining steps for the coefficient calculation are addressed on the annex B. Note that, for the present application, the cross sections were not considered as class 4. Thus the effective area is equal to the total cross section.

Table 3.11 – Reduction coeffients

Bar	**Reduction coefficient, χ**
[aB]	0.99
[BC]	1
[Cc]	0.50
[CD]	1
[Dd]	0.50

3.6.5 Reliability index calculation

For the calculation of the reliability index in this practical application, as mentioned in section 3.6 only the superstructure will be considered, i.e. the deck. Such consideration relied on the information provided on the drawings for this bridge. Likewise the previous case, the reliability calculation related to a LSF definition. Thus, for the truss bridge, the LSF was calculated by considering the difference between the resistance axial strength and the axial action as expressed in equation (3-20).

$$Z = f_y \times A - N_S(PL, LM71) \tag{3-20}$$

where f_y is the yield strength of steel in MPa and A is the cross section in mm^2. The axial load was given as a function of the permanent loads (PL) and the live loads. Considering that uncertainty quantification was needed to have a proper definition of the reliability index, the variables were defined by probabilistic normal distributions with mean and coefficient of variation except for the model uncertainties, which was considered lognormally distributed, see Table 3.12.

Table 3.12 – Random variables quantification

	Variable	Mean Value	Coefficient of Variation (CoV)	Reference
Resistance	Cross Section, A (mm^2)	Nominal	4%	[109]
	Yielding strength, f_{ym} (MPa)	202.16 MPa 101.08MPa*	7%	[109]
Actions	Permanent Loads (PL)	23 kN/m	10%	-
	Live loads (LM71)	207.4 kN 63.4 kN/m	10%	[151]

*Value of the yielding strength affected by the buckling coefficient

After having defined all the resistance and demand variables, a structural analysis was made, and the limit state equations were chosen and entered within a FORM analysis to calculate a reliability index. Once the structure is isostatic, the obtained global reliability index was given by the minimum value obtained for each bar element. Table 3.13 shows the obtained reliability indexes. Accordingly, the considered reliability index for the whole truss system was 4.87 corresponding to the [Dd] element bar located at the middle span. According with the threshold values defined in Table 3.1 and Table 3.2, the structure was above the threshold values of ISO 2394 and the PMC and it was considered to be in a satisfactory level according with fib. However, according with EN 1990, for the minimum value of the reliability index, considering that a reference period of one year is 5.2 for the highest consequences, the structure did not meet the requirements. However, considering the medium consequences, the structure met the requirement. Once the structure was not located in a principal railway line of Portugal, the consequences normally are of lower dimension, therefore the threshold value could be reasonably checked according with the medium value. However, a warning should be placed for this bridge once with the deterioration process and natural hazards, the structure could present a serious threat.

Table 3.13 – Reliability indexes for each bar element

Bar	[aB]	[ab]	[bB]	[bc]	[Bc]	[BC]	[Cc]	[cd]	[Cd]	[CD]	[Dd]
Reliability index	9.87	10.00	14.29	10.00	11.53	9.96	5.83	8.98	12.70	8.87	4.87

3.6.6 Risk Analysis

The risk analysis calculation followed the same procedure as section 3.5.2 with the calculation of the consequences for the same scenarios, i.e. rebuilding of the deck and detour costs. For the detour costs, Figure 3-16 shows an example of an alternative detour route considering that the bridge is out of service. Given the redundancy of the roadway network, there were several options. However, the fastest detour was selected to minimize the consequences. Following the equations (3-16) and (3-17), given the parameters presented in Table 3.14, the direct and indirect consequences are depicted in Table 3.15 for the initial age. Note that here the rebuilding costs were assumed as deterministic per length of the deck (€/m), due to the report information about reconstruction of steel bridges provided by the Portuguese infrastructure company *"Infrastruturas de Portugal (IP)"*. Moreover, the *PER* variable is considered deterministic measured as 100 % since the presented application differs from the previous one by crossing a river and not a roadway thus resulting on a complete closure.

Table 3.14 - Variable quantification for indirect consequences (parameters adapted from [149])

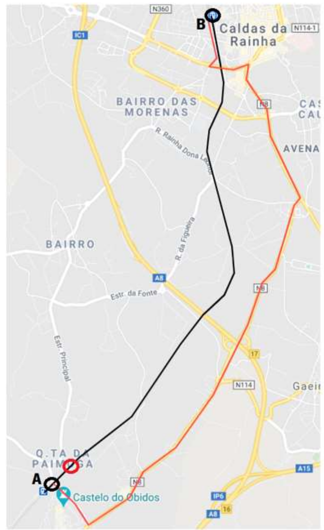
Description	**Notation**	**Value**	
Rebuilding of the deck (€/m)*	c_{reb}	8000	
Length (m)***	L	27.25	
Traffic conditioned percentage	PER	100%	
Average daily traffic	TMD	Cars	Trucks
		9000	1000
Cost per kilometer (€/km)	CK	0.18	0.68
Cost per hour (€/h)	CH	8.4	10.1
Normal Speed (km/h)**	S_n	160	
Restricted Speed (km/h)***	S_r	70	50
Detour route (km)***	LD	7.000	

Normal route (km)***	LP	5.000

* value assumed according with bridge repairing reports from IP

** normal speed of the train for that zone of the line

*** measured values for the present application



Legend:
Point A: Begin of the detour; Point B: End of the detour
Red Circle: Bridge Location; Black Line: Railway line; Red Line: Alternative detour

Figure 3-16 – Detour route

Table 3.15 – Consequences and Risk

	Consequences	**Risk**
Direct (€)	218000	0.616
Indirect (€)*	404	0.001
Total (€)	218404	0.617

*estimated per day

Analogously to section 3.5.2, the obtained risk was practically zero due to the high value of the reliability index and the corresponding very low probability of failure (5.58e-7) for the overloading hazard scenario previously defined at the initial age. The next section aims to estimate the risk-based robustness index applying the same procedure as section 3.5.3.

3.6.7 Risk-based robustness indicator

The previous section aimed to estimate the risk considering the daily indirect consequences. Nevertheless, those consequences can be higher once the system can take more time to recover. Figure 3-17 illustrates the cumulative time-dependent daily estimated consequences.

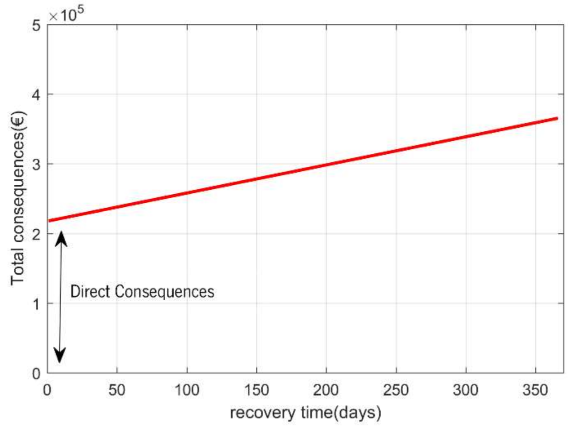


Figure 3-17 – Cumulative time dependent consequences

Considering the equation (3.18) for the robustness indicator calculation, Figure 3-18 shows the robustness indicator and the associated total consequences considering the cumulative daily recovery

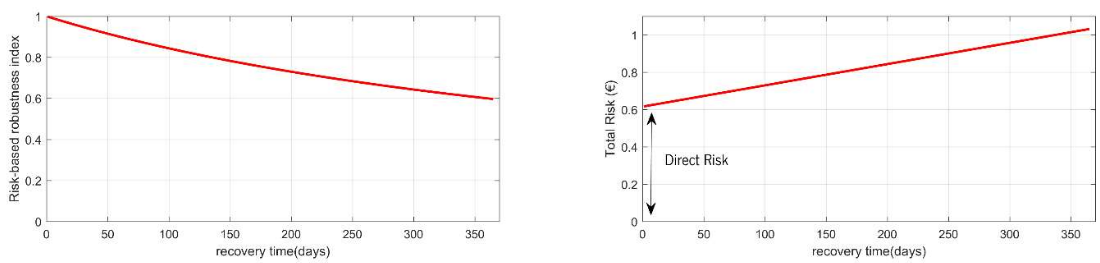


Figure 3-18 - Risk based robustness index and total risk

Considering the obtained results, similar insights could be drawn comparing to the previous application. The total risk was very low therefore not presenting any threat to the system. Analyzing the robustness indicator, compared to the previous application, it could be concluded that the system was not as dependent as the indirect risk since the dropping rate of the curve was smoother. In fact, the detour route was almost the same length as the original route which made the indirect consequences smaller in this case. Nevertheless, it constitutes a noteworthy information to mention that the recovery of the system here is totally theoretical once there is no information available regarding real recovery times for this application. However, more realistic values could be appointed to 5 to 10 days to totally replace of the steel bridge.

3.7 Case C – Masonry railway arch bridge: Canharda viaduct

Canharda Viaduct, see Figure 3-19, was built in 1882 and it is located in Beira Alta railway line. It is composed by five full-centered arches, each one with a free span of 12 m and a maximum height of 20 m. Its total extension is of 86,5m with a top width of 4m and it is all built of rough dry joint masonry. All piers have the same geometry, with exception of their heights, being the cross section in longitudinal and transversal directions, variable (linear variation).

Figure 3-19 – Canharda Viaduct

3.7.1 Bridge geometry

Due to the complexity of a masonry arch bridge (MAB), the present application dealt with a detailed modelling of the structural elements and a simplified modelling of the non-structural elements. According with the [154, 155], the most important elements concern: (i) typology of the arches, the length and thickness of the span; (ii) the width of the arches; (iii) backfill material and backing height and (iv) geometry of the piers and abutments.

Concerning the bridge, both abutments were modelled considering a backing height of 3000mm. The five arches of the bridges were similar and considered with a ring thickness of 870mm (considered to be uniform), a total span length of 12000mm and a rise of 6000mm, i.e. a span to rise ratio of 0.5, see Figure 3-20. Regarding the 4 piers, Table 3.16 presents the adopted dimensions.

Table 3.16 – Geometry of the piers

	Pier 1	Pier 2	Pier 3	Pier 4
Total Height (mm)	4600	11350	11350	6400
Width at top (mm)	2500	2500	2500	2500
Width at base (mm)	2700	3000	3000	2800
Number of blocks (mm)	11	27	27	15
Backing height over the pier (mm)	3000	3000	3000	3000

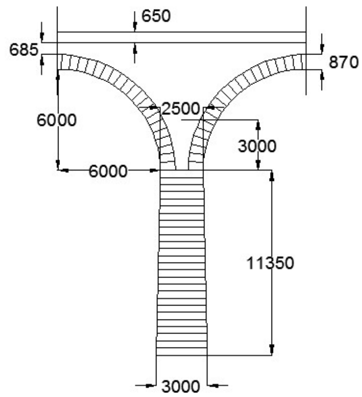


Figure 3-20 – Geometry adopted for the main arches [mm]

3.7.2 Bridge materials

The MAB accounted with the quantification of the following materials for the model: (i) masonry; (ii) Fill and (iii) ballast. Table 3.17 shows their quantification. Regarding the masonry, the density was adopted to assume the value of 25 kN/m^3 with a compressive strength of 20 MPa. The friction coefficient was considered of 0.58 (corresponding friction angle of $30°$). For the fill, a density of 15 kN/m^3 was considered based on the measurements on the bridge span. As for the internal friction, an angle of $30°$and a null cohesion were adopted. The null cohesion was an adopted value as a conservative measure for the model calculation. Moreover, the fill properties were hypothetical considered to be uniform, the same hypothesis considered in the studies of [96, 156].The ballast density was adopted as 17.66 kN/m^3 based on [157]. For the track, a value of 1.42 kN/m^2 based on [158].

Table 3.17 – Parameters considered for the analysis

Materials	**Parameters**	**Quantification**
Masonry	Density, γ_m (kN/m^3)	25
	Compressive Strength, f_c (MPa)	20
	Friction Coefficient, μ $(-)$	0.58
Fill	Density, γ_f (kN/m^3)	15
	Internal friction angle, ϕ $(\circ)$	30
	Cohesion, c (kPa)	0
Ballast	Density, γ_b (kN/m^3)	17.66
Track	Track load per unit area, T_L (kN/m^2)	1.42

3.7.3 Load modelling

For the present application, the load model to apply was based on the LM71 presented by [151], the same as presented by the previous applications. Moreover, the present application accounted with the dynamic effects, ϕ_3 following the equation (3-19). In this case (series of arches) and according to EN 1991-2 [151], the determinant length, L_ϕ, was twice the clear opening. Additionally, if the assessed structure is an arch bridge, the dynamic amplification may be reduced through a reduction factor [151], given by:

$$red\ \phi_3 = \phi_3 - \frac{h-1}{10} \tag{3-21}$$

being $red\ \phi_3$ greater or equal to 1,00 and h is the height of cover including the ballast from the top of the deck to the top of the sleeper given (in meters). The load PDF will be multiplied by the reduced dynamic amplification factor, given by equation (7), to increase the static load magnitude as to consider the static response amplification due to dynamic phenomena. The corresponding obtained value was of 1.18. Another important aspect regarding the LM71 recalls to the inclusion of the uniform distributed loads since these loads are optional depending or not of their favourable effects on the bridge performance calculation. This aspect was studied by [159] on MAB wherein three different load combinations were studied. Santis [159] concluded that the presence of the optional distributed loads has a favourable effect on the performance of bridge. Bearing this in mind, the present application considered the LM71 composed by the four concentrated loads.

3.7.4 Structural modelling

The present application was modelled by the software Limit State RING [160]. RING software idealizes MAB as an in-plane assemblage of rigid units, separated by contact interfaces along masonry joints, with a rigid-plastic constitutive behaviour [161]. MAB may collapse due to crushing, sliding along interfaces or a mix of these two failures mechanisms. The spandrel walls are not explicitly considered and the fill's influence is indirectly modelled through its effects (density, dispersion of the loads applied at the surface and horizontal passive pressure) [161]. The passive pressure, provided by the fill, is obtained through the Rankine theory. Since passive pressure is fully mobilised only for high deformations and displacements of sections of an arch into surrounding fill [161, 162], a reduction factor of 50% was applied to passive pressure, in order to considerer this fact. The live loads are dispersed according to the Boussinesq theory, with a limiting distribution angle of 30°, according to performed laboratorial tests [161]. The arch effective bridge width, i.e., the width of masonry arch in transversal direction that resists the applied loads, was considered equal to the bridge width, since no cracks were found. Regarding the collapse mode, the assessed bridge was entirely modelled in RING, once all failure modes involved more than one span (seven hinges mechanism), leading to being only obtained multi-span failure modes. For a more detailed description of MAB, address to [101].

3.7.5 Deterministic model output

Considering the assumptions on the model, through RING software [160], the mechanism of collapse that led to the minimum load factor obtained is given in Figure 3-21. The minimum load factor obtained was of 4.10. Note that this value is the multiplier on factored vehicle loads required to cause collapse. It was observed from Figure 3-21 a global mechanism with an interaction mainly of the arches 2 and 3. Also it was noticed that the global mechanism also affects the pier 2 and 3 on the base. More detailed information about the different types of collapse mechanisms in MAB, refer to [101, 163, 164].

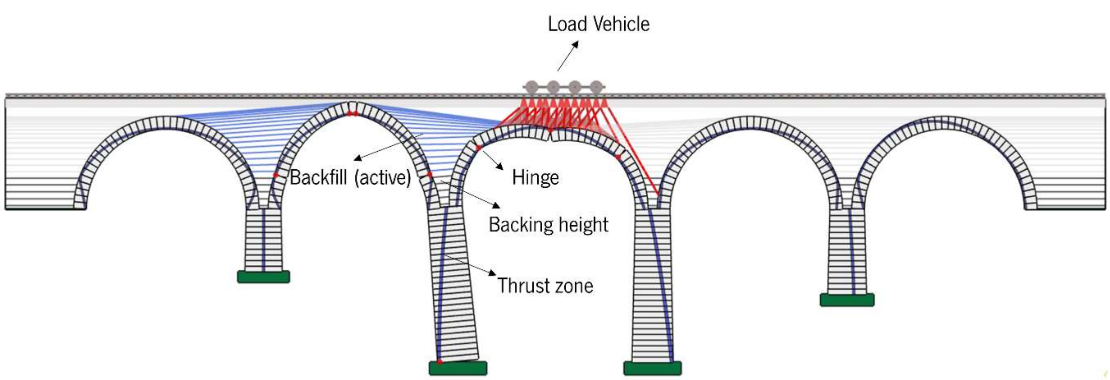


Figure 3-21 – Global mechanism of the MAB for the deterministic model

3.7.6 Reliability index calculation

The following section addresses a probabilistic analysis and the calculation of the reliability index. Before carrying out a probabilistic analysis, a sensitivity analysis was performed to minimize the number of probabilistic parameters for the analysis. For more detailed sensitivity analysis calculation, see [101]. According with [101], for the present application, the following parameters were considered: (i) density of the masonry; (ii) density of the fill; (iii) friction angle of the fill; (iv) pier width and (v) arch thickness. Table 3.18 shows its quantification for the mean value and coefficient of variation.

Table 3.18 – Random variables adopted

Materials	Parameters	Mean Value	COV (%)

Masonry	Density, γ_m (kN/m^3)	25	10
Fill	Density, γ_f (kN/m^3)	15	10
	Internal friction angle, ϕ (∘)	30	20
Geometry	Pier width, W_p (mm)	*	10
	Arch Thickness, t (mm)	870	10

*see Table 3.16

The LSF is given as follows:

$$Z\ (\gamma_m, \gamma_f, \phi, W_p, t, LM71) = R - S \tag{3-22}$$

The definition of an analytical expression, as seen on the previous applications, was not possible here to obtain due to the complexity of the problem. Hence, the calculation was performed through LHS method. The analysis considered 90 samples for the reliability calculation. From each simulation, an adequacy factor was obtained, to which a curve fitting process was performed to adjust the most suitable PDF. Considering that this application assumed all the resistance parameters behave according with a normal distribution, the curve fitting was validated through the central limit theorem thus resulting on a normal distribution. Hence the resistance parameter was represented by a normal distribution with the following parameters:

$$R \sim N(4.10, 0.47^2)$$

As for the actions, considering the LM71 affected by the dynamic coefficient, a normal distribution for the actions was considered as given as:

$$S \sim N(1.18, 0.11^2)$$

Note that the resistance and the actions were herein presented with normalized values. Moreover, a convergency test was performed to check the adequacy number of simulations adopted by obtaining the ratio between the cumulative mean on the *n* simulation over the overall mean for the 90 simulations, see Figure 3-22.

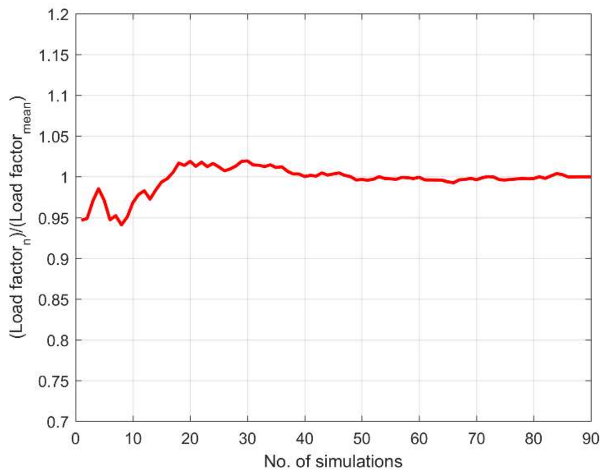
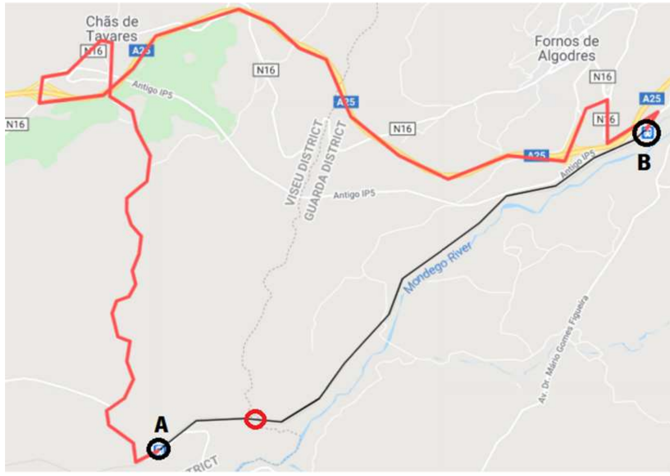


Figure 3-22 – Convergency of the simulations

The reliability index was obtained through FreET software [165] with a corresponding value of 6.10. The obtained value was very high for this arch bridge. Indeed, for arch bridges with medium span (10-15m), the fill internal friction angle is important, since it provides more passive pressure, resulting in an additional restrains, and thus a higher collapse load. Another important issue is the ballast and the fill height in the arch crown zone of the arch. This process allows a higher dispersion of the lives loads resulting therefore in less stresses in the arch and in a higher collapse load.

3.7.7 Risk analysis

The risk calculation followed the same procedure as the previous applications. Figure 3-23 shows an example of an alternative detour route considering that the bridge is out of service. Again, following equations (3-16) and (3-17), given the parameters presented in Table 3.19, the direct and indirect consequences are depicted in Table 3.20 for the initial age. Note that the rebuilding constructions were compared to a big intervention wherein the condition state of the bridge was considered as minimum and close to collapse. Its quantification was based on the report information about reconstruction of masonry bridges provided by *IP* company.



Legend:
Point A: Begin of the detour; Point B: End of the detour
Red Circle: Bridge Location; Black Line: Railway line; Red Line: Alternative detour

Figure 3-23 – Detour route

Table 3.19 – Variable quantification for the direct and indirect consequences

Description	Notation	Value	
Rebuilding (€/m)*	c_{reb}	1000	
Length (m)***	L	86.50	
Traffic conditioned percentage	PER	100%	
Average daily traffic	TMD	Cars	Trucks
		9000	1000
Cost per kilometer (€/km)	CK	0.18	0.68
Cost per hour (€/h)	CH	8.4	10.1
Normal Speed (km/h)**	S_n	200	
Restricted Speed (km/h)***	S_r	120	100
Detour route (km)***	LD	19.000	
Normal route (km)***	LP	8.100	

* value assumed according with bridge repairing reports from IP

** normal speed of the train for that zone of the line

*** measured values for the present application

Table 3.20 – Consequences and Risk

	Consequences	Risk
Direct (€)	86500	6.45e-5
Indirect (€)*	29971	2.23e-5
Total (€)	116471	8.683-5

*estimated per day

3.7.8 Risk-based robustness indicator

Likewise the previous sections of risk-based robustness calculation, Figure 3-24 shows the cumulative total consequences function of the number of days to recover.

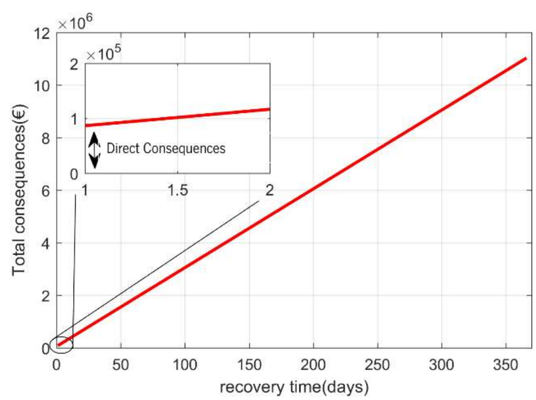


Figure 3-24 – Total consequences estimation

Considering the equation (3.18) for the robustness indicator calculation, Figure 3-25 shows the robustness indicator and the associated total consequences considering the cumulative daily recovery

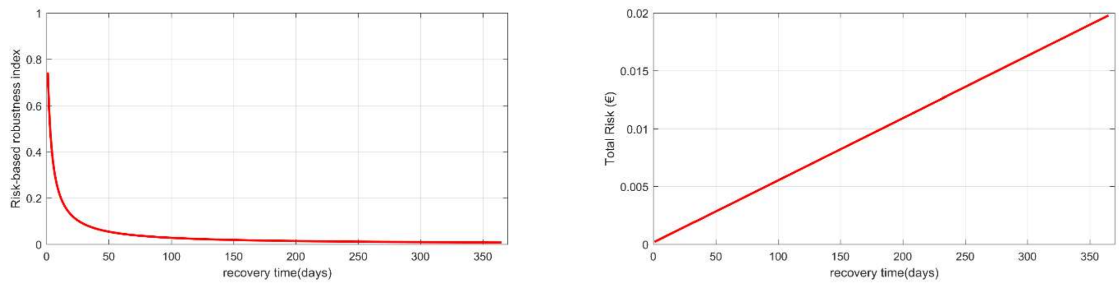


Figure 3-25 – Risk based robustness index and total risk

The obtained results concluded that the risk-based robustness indicator was highly dependent on the indirect consequences as there was a considerable drop on the values for low days of recovering. As for the total risk, as the probability of failure was very low due to the high reliability index, the values were practically zero concluding that due to the overloading effects, there was a minimal risk involved.

3.8 Final remarks

The present chapter aimed to introduce the theoretical formulations adopted in the literature for the calculation of the KPIs in existent structures, reliability, risk, and robustness. Moreover, an overview of the most relevant works applied in civil engineering adopting these KPIs was herein presented as well as a brief table about the comparison among those KPIs in terms of strengths and weaknesses. This chapter ended with an application of those KPIs to three different practical applications: (i) simply supported RC bridge; (ii) steel bridge and (iii) masonry arch bridge. Regarding the reliability index, the structures revealed to be in good condition state for problems related with overloading. Moreover, a comparison was made with the threshold values provided by the codes wherein the practical cases A and C revealed to meet the requirements for the threshold values. In what concerns the practical case B a special attention should be paid for the threshold values preconized by the EN 1990 for a reference period of 1 year and considering the level high for the consequences.

Regarding the estimation of the risk, all the three cases revealed to present a very minimal threat to the system given the variables introduced in the problem and the hazard considered, i.e. the overloading highlighting the fact that the bridge are over-designed in most of the cases. However, it should be noted here that consideration of different hazards such as bridge strikes, floods, earthquakes, among others, or even the degradation process of the structure could lead to different levels of risk and consequently to higher levels of consequences.

To finalize, an estimation of the risk-based robustness index was carried out based on the formulation of Baker et. al [129] to measure the robustness of the system. This measure was slightly different from the other robustness measures since the risk-robustness is highly dependent on the indirect consequences of the system, i.e. variables that depend on the location of the structure, development of the society and the network around the structure. Hence, a lower robustness in here might not mean that the structure has a very low performance, as observed in the present applications. Moreover, this measure estimated alone with any other calculations could lead to naïve and erroneous conclusions. The next chapter aims to focus on degradation modelling of structures wherein the state of the art of the most relevant degradation models on infrastructures are discussed. The chapter ends with an application of the degradation models into practical applications.

Chapter 4

4 Degradation modelling

Degradation models are an important part of a Management System since they predict the future condition of the infrastructures, being therefore an essential tool for the decision-making process. The process of developing a degradation models is not an easy task due to several uncertainties associated to the process. Degradation can be defined as *"the decrease in capacity of an engineered system over time, as measured by one or more performance indicators" [166]*.

According with the formulation of degradation stated in [166], its basic formulation considers the initial capacity, denoted as V_0, the degradation over the time, such as $D(t)$ and the remaining capacity, $V(t)$, resulting on the following equation:

$$V(t) = V_0 - D(t) \tag{4-1}$$

According with equation (4-1), the failure occurs when the remaining capacity drops to zero. However, in the practical cases there are always a threshold value, denoted by *k*, which defines the limit value allowable to be reached for an infrastructure. In this way, the expression is re-written as:

$$V(t) = \max(V_0 - D(t), k) \tag{4-2}$$

Degradation models are divided into (i) Mechanistic models (ii) Deterministic models; (iii) Random-variable models; (iv) Stochastic models; (v) Petri-net models, (vi) Artificial intelligence models and (vii) Bayesian networks. Note that the notation herein used for the classification of the degradation models are based on the literature review. Figure 4-1 shows a resume of the different degradation models presented in the literature.

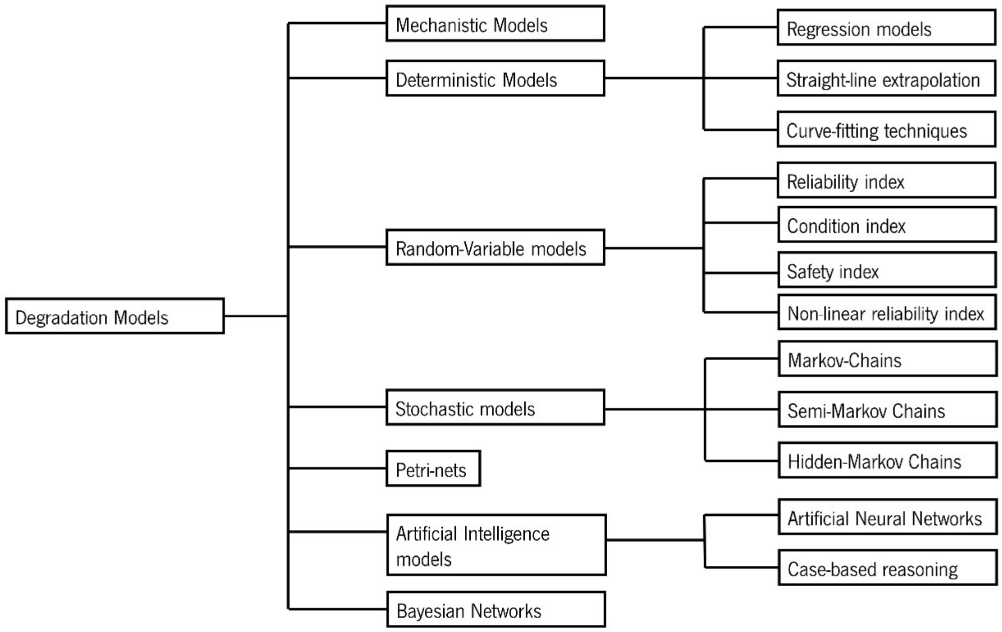


Figure 4-1 - Degradation models for bridges

4.1 Mechanistic models

Mechanistic models are based on theoretical models and they are usually applied to describe specific deterioration of specific components [167]. The main drawback of these models concerns its application to a higher level (e.g., network level). The literature presents several theoretical models to model the degradation of an infrastructure. For example, in the studies of [168] and [169], a corrosion damage model for steel bridges is proposed. The mechanism model based on this study follows an exponential function given by:

$$C(t) = At^{B} \tag{4-3}$$

where C is the average corrosion penetration, t is the number of years and A and B are parameters derived from observations. With this model, rates of corrosion for steel bridges can be estimated and consequently the remaining resistance capacity.

Another example of application relies on reinforcing concrete (RC) structures wherein most of the models presented in the literature are corrosion-based models. Typically, the loss of structural strength in aging RC bridges is mainly attributed to the chloride-induced corrosion of reinforcing bars [170]. In this process, various mechanisms are evidenced such as loss of the reinforcement area, accumulation of corrosion products leading to volumetric expansion that results in concrete cracking and spalling, loss of the steel–concrete bond, and variation of mechanical properties of steel and concrete [170, 171]. The process of reinforcement corrosion can be roughly divided into an initiation and a propagation phase. During the initiation phase, chloride ions diffuse through the concrete cover until the chloride content on the surface of the reinforcement exceeds a critical threshold value, which dissolves an oxide protective layer (depassivation) [172, 173]. The transport of chloride ions through concrete is typically modelled using Fick's second law [171]. Based on this model and accounting for the impact of influencing parameters, the probabilistic model for the time for corrosion initiation is expressed as [174, 175]:

$$\boldsymbol{T_{corr}} = \boldsymbol{\theta_d}\left[\frac{\boldsymbol{x^2}}{\boldsymbol{4k_e k_c k_t D_0 t_0}^{\boldsymbol{n}}}\left[\mathbf{erf}^{-1}\left(\mathbf{1}-\frac{\boldsymbol{C_{cr}}}{\boldsymbol{C_s}}\right)\right]^{-2}\right]^{1/(\mathbf{1}-\boldsymbol{n})} \tag{4-4}$$

where θ_d is a model uncertainty coefficient to account for the idealization on Fick's second law, x is the concrete cover depth, k_e is an environmental factor, k_c is a factor that accounts for the influence of curing, D_0 is the chloride diffusion coefficient which describes the resistance against the ingress of chlorides, k_t is a factor describing the effect of the test method to determine D_0, t_0 is the reference period for D_0, n is an age factor that incorporates the densification of cement paste due to further hydration chloride, erf is the error function, C_{cr} is the critical chloride concentration, and C_s is the surface chloride content determined from the expression $C_s = A_{cs}(w/b) + \varepsilon_{cs}$, where A_{cs} is a regression parameter between C_s and the water-binder ratio (w/b), and ε_{cs} is the error term [174]. During the propagation phase, various mechanisms are evidenced such as loss of the reinforcement area. To its estimation, the diameter of the reinforcement bars d_b at time t can be expressed as [175]:

$$d_b(t) = \left\{\begin{matrix} d_{bi} & t \le T_{corr} \\ d_{bi} - 2\int_{T_{corr}}^{t} \lambda(t)\, dt & T_{corr} < t \le T_f \\ 0 & t > T_f \end{matrix}\right\} \tag{4-5}$$

Where d_{bi} is the initial bar diameter, T_{corr}, is the time for the corrosion initiation, $\lambda(t)$ is the corrosion rate represented as $\lambda(t) = 0.0116 i_{corr}(t)$, where $i_{corr,t}$ is the corrosion current density at time t [171], and T_f corresponds to the time when $d_b(t)$ theoretically reaches zero [175].

4.2 Deterministic models

In deterministic models the condition assessment of the infrastructures is obtained by a mathematical formula. Regression analysis is the most basic degradation model that consists in establishing a relationship between the actual data of a bridge's condition with the form of a mechanical model or a selected arbitrarily form, by using statistical techniques. The output of this model is expressed by deterministic values that represent the average predicted condition [176]. Several studies have applied different regression methods such as linear, non-linear and stepwise regressions to model the infrastructure degradation [177, 178].

The linear regression presents the general form $y(t,\beta) = \beta_0 + \beta_1 t + \varepsilon_i$ where β_0 and β_1 are regression parameters which can be obtained by using the method of least squares and ε_i the prediction error. There is also the multiple linear regression that considers more parameters being defined as: $y(t,\beta) = \beta_0 + \beta_1 t_1 + \beta_2 t_2 + \cdots + \varepsilon_i$ [179]. Further details of its calculation can be seen in [166]. However, in most of the practical applications, the problems are not linear. The basic idea is the same as the one applied on linear regression. Some examples of practical applications can be found in [180, 181]. Usually, in a way to simplify the process, those equations are transformed into linear problem. One example can be seen in [166] wherein the fatigue data of the two asphalt mixtures was simulated using a degradation model via non-linear regression analysis, being transformed to linear analysis. Stepwise regression can be used to estimate the statistical importance of the performance indicators (PIs) in a model. For example, according with [182], within this model, the prediction of the condition state was dependent of several variables such as infrastructure type, condition rating, nondestructive testing results, infrastructure traffic, infrastructure age and surrounding environment. Another example of its application can be seen in [183]. In this work, the performance indicator assessed was the deflection of the bridge based on monitoring data wherein the results showed a high accuracy proving to be an effective method for bridge deformation analysis. Yet, these type of models presents some drawbacks such as the difficulty to incorporate additional data and the estimation of R^2 as well as the residual error. Moreover, regression extrapolation techniques disregard the physical nature of degradation and the uncertainty involved on the variables, they do not account for the current condition of a facility and its history to predict the future condition. They ignore also the interaction between degradation of different infrastructure components [184]. Although these models are acceptable for predicting a short period of time, when these functions are predicted beyond the bounds of the data for long time periods (which is required in the case of systems with expected long lifetimes such as civil infrastructures), they can contain high levels of inaccuracy [177]. Straight-line extrapolation and curve-fitting techniques are additional deterministic models that have been used to develop degradation models. An example of this technique can be applied to predict the condition rating of the bridge given the assumption that traffic loading and maintenance history follows a straight line [185]. This method requires that one condition measurement has been performed after construction [179]. Nonetheless, they suffer from the same limitations as regression models.

4.3 Random-variable models

Modelling of degradation is strongly influenced by sampling and temporal uncertainties [186]. Random variable (RV) models account for the sample variability by assuming that one or more parameters associated with the degradation model are random, i.e. a probability distribution is assigned to the uncertain parameters [187]. Among the RV distributions, a significant amount of research has been carried out on reliability index models where the lifetime distribution is associated to a limit state which is a function of one or more random variables [187]. The variation of the reliability index with time is defined as the reliability profile, and it has been extensively used to model linear degradation of structures under different maintenance scenarios (no maintenance, preventive and essential maintenance) [188-191], considering uncertainties related with the degradation process, the maintenance actions effects and their application times. The degradation model is given by a bi-linear relation as follows:

$$\beta(t) = \begin{cases} \beta_0 \, for \; 0 \leq t \leq t_I \\ \beta_0 - \alpha_1(t - t_I) for \; t > t_I \end{cases} \tag{4-6}$$

Later, another linear model was proposed by Neves and Frangopol [4]. In this study, they defined two key performance indicators (KPIs): (i) condition state; (ii) safety index. The former had into consideration the results of visual inspections while the later overcomes some limitations of the condition index in order to cover effects such as degradation processes that are not visible to the inspectors. In this way, the degradation model is defined as follows:

$$C(t) = \begin{cases} C_0 \, for \; t \leq t_{ic} \\ C_0 - \alpha_L(t - t_{ic}) \;\; t > t_{ic} \end{cases} \tag{4-7}$$

$$S(t) = \begin{cases} S_0 \; t \leq t_i \\ S_0 - \alpha(t - t_I) \; t > t_i \end{cases} \tag{4-8}$$

Being C_0 and S_0 the initial condition and safety states, α_L and α the degradation rates, t_{ic} and t_i are the times of initiation of degradation.

The consideration of linear degradation could be an idealized assumption limited for various failure modes, so it has been extended in some studies into higher order functions to consider the non-linear degradation effect with time. The study of Gaal [192] showed that the validation of the prediction model regarding the spalling assessment fitted better on a second order polynomial function. Another study was carried out by Petcherdchoo [187] in which a non-linear Reliability index degradation model was proposed. The formula was defined as follows:

$$\beta(t) = \begin{cases} \beta_0 \; for \; 0 \leq t \leq t_I \\ \beta_0 - \alpha_2(t - t_I) - \alpha_3(t - t_I)^p \; for \; t > t_I \end{cases} \tag{4-9}$$

being α_2 and α_3 the degradation rates. Later, Frangopol and Neves [193] also proposed non-linear degradation models where the safety and condition index were stated as follows:

$$C(t) = \begin{cases} C_0 \; for \; t \leq t_{ic} \\ C_0 - A(t^2 - t_{ic}^2) \;\; t > t_{ic} \end{cases} \tag{4-10}$$

$$S(t) = \begin{cases} S_0 \; t \leq t_i \\ S_0 - B(t^2 - t_i^2) \; t > t_i \end{cases} \tag{4-11}$$

where A and B are the degradation rates.

4.4 Stochastic models

4.4.1 Markov chains

As the infrastructures ages the deterioration rate grows, which is not accounted in reliability approaches based on lifetime distributions. For this reason, several research papers have recommended to model deterioration as a time-dependent stochastic process that incorporates the temporal uncertainty [187].

Markov chain (MC) approach is the most widespread stochastic deterioration modelling technique that has been used for predicting the performance of infrastructure facilities [194]. MC models capture the uncertainties and randomness of the deterioration process by accumulating the probability of transition from one condition state to another over multiple discrete time intervals [194]. Transition probabilities are estimated using the relative transition frequency of the condition states, which is determined by visual inspections carried out [195]. There are different methods to estimate the transition probability matrix (TPM). However, obtaining and validating reliable TPM is one of the most difficult matters in developing MC deterioration models [196]. Within the civil engineering field, their first application were in Pavement Management Systems (PMS) to describe their condition over the time. Nowadays, there are several studies using Markov chains to model the deterioration over time in different infrastructures such as bridges, railways, pavements, and wastewater systems. The following paragraphs highlight some of the studies based on MC.

The early studies rely on Golabi et al. [30] in which a maintenance system for roads in Arizona was proposed. The MC model considered both short and long-term management objectives as well as factors such as physical road conditions, traffic densities, environmental characteristics, and types of roads.

Later, Butt et. al [197] proposed to use a combination of homogeneous and non-homogeneous Markov chain to develop the model. The Pavement condition index ranged from 0 to 100 with a division of ten intervals to define 10 condition states.

Yang et.al [198], adopted a MC model to predict the pavement crack performance by employing a logistic model to estimate the transition probabilities. The model accounted with crack conditions, pavement parameters and traffic as variables.

Kobayashi et. al [199] presented a methodology to estimate Markov transition probabilities to predict the deterioration of the pavement. The transitions were defined by exponential hazard modes. Due to the irregularity of data, the transition probabilities had many uncertainties. At the end, the applied model revealed to be useful once it was positively applied to an empirical example. Abaza [200] presented a procedure to evaluate the distress rate having into account two defects, cracking and deformation. The calculation of the transition probabilities was based on "back calculation". Hassan et al. [201] presented a probabilistic model based on Markov chains in which two variables were assessed, the distress rate and surface age. Moreira et. al [202] developed an application based on Markov chains to predict the evolution of 5 PIs: cracking, skid resistance, bear capacity, longitudinal and transverse evenness. The results showed a slow deterioration for all the PI's and they were useful for the development of optimization methodologies for PMS.

Although these types of models were applied first in pavements, applications of MC models were also developed to Bridge Management Systems (BMS). Jiang et. al [203] presented in his study the efficiency of the Markov chain approach to estimate the future bridge conditions. The bridge rating system was the one adopted by the Federal Highway Administration (FHWA) in which the bridge inspectors employed a range from 0 to 9 being the 9 the best condition and 0 the worst one.

Cesare et. al [204] provided another study related with Markov chains applied to BMS. The MC models were used on the assessment of highway bridges deterioration with a database of 850 bridges in the state of New York. The transition matrices were obtained to the overall condition of the structure. Therefore, a TPM was developed for each component. Also, a discussion of the importance on the correlation between the elements was discussed. The results showed that the estimation of transition probabilities was affected by the lack data.

Orcesi and Cremona [205] proposed an approach to obtain the condition state by using MC models. The transition matrices for several types of bridges were obtained and combined with an advanced traffic assignment that considered congestion phenomenon's and enabled to quantify the levels of service of the bridge network in an accurate way. The same authors [206] also presented a Markov chain framework for a reinforced concrete bridge aiming to a global vision of the condition of the bridge stock to assess the optimal funds at national level. One of the drawbacks was the assumption of a homogenous database with the same condition for all the bridges.

Sobreiro [207] presented his work based on a Markov process approach to develop a predicting model for deterioration of existing bridges. The TPM was obtained by the data referred to visual inspections. Such data was important to establish plans of maintenance actions

Wellage et al. [208] pointed out that bad estimation of transition probability matrices would lead to invalid future predictions. In this way, the authors presented an algorithm to overcome this limitation and calibrate the state-based MC model of railway bridge components. Those transition probability matrices were estimated based on MC Monte Carlo, regression-based non-linear optimization, and Bayesian maximum likelihood. The results showed better accuracy for MC Monte Carlo approach.

Denysiuk et al. [12] proposed a computational framework for optimization of maintenance activities on bridges based on MC model at the element and system levels. The classification adopted was defined by 1 to 5 being 1 the best and 5 the worst state.

Li et al. [209] proposed on their study a MC model to predict the performance degradation tendency of different components of bridges in different areas. Bridge condition rating records of Shanghai were used to predict the deterioration process of local bridge on a network level. Two maintenance actions were considered: (i) routine maintenance; (ii) minor, medium and major repair. The results showed that routine and minor repair seemed to present a faster degradation on the infrastructure than medium repairs. The condition rate adopted ranged from 5 to 1 being the best and the worst, respectively. The database considered for this study was based on records of 16623 bridges.

In the field of the railway track, there were also some developed works regarding degradation models adopting MC. Bai et al. [210] proposed an application of MC model to evaluate the degradation of track maintenance unit between two consecutive maintenance activities. The states of Markov to assess the track quality index (TQI) were divided into 4 states. It was also considered heterogenous factors while obtaining the transition probability matrix, resulting on different degradation behavior for two maintenance units in the same mileage.

Shafahi and Hakhamaneshi [211] applied a two-stage model to optimize track maintenance. In the first stage, the authors used Markov chain to model track degradation behavior obtaining the transition probability matrix based on the data of Iranian railways. In the second stage, they applied dynamic programming optimization to obtain optimal decisions for maintenance activities.

Zakeei et al. [212] proposed a model based on MC to predict rail wear. The model was built based on Weibull distribution to define the hazard rate function and therefore the transition probabilities. Regado [213] presented a study to model track railway degradation based on MC model. The model was based on inspections of the geometrical parameters of the line: (i) longitudinal level; (ii) alignment.

Although the MC models present several advantages regarding data manipulation and consideration of uncertainties on the problem, Morcous [194] states some limitations such as the memoryless property, wherein the next state only depends the current state disregarding all the information concerning the previous states.

Regarding its theoretical formulation, MC model is a random process that undergoes transitions from one state to another on a state space. The transition between states is defined by:

$$P^{\Delta t} = \begin{bmatrix} p_{11} & p_{12} & \dots & p_{1n} \\ 0 & p_{22} & \dots & p_{2n} \\ \vdots & \vdots & \ddots & \vdots \\ 0 & 0 & \dots & p_{nn} \end{bmatrix}^{\Delta t} \tag{4-12}$$

where p_{ij} is the transition probability between the states *i* and *j* from instant t to $t + \Delta t$. When the interval between the inspections are not regular, the P matrix is related to Q matrix through the following equation:

$$\frac{d}{dx} P = PQ \tag{4-13}$$

where P is the transition matrix and Q the intensity matrix. Solving the equation, the transition matrix can be given by:

$$P = e^{Q\Delta t} \tag{4-14}$$

The intensity matrix Q represents the instantaneous transition probability between state *i* and the state *j*. It is assumed that, in each time interval, assets can only advance between adjacent condition states. Therefore, the elements of Q are null except for the main diagonal and the diagonal above as it is shown in:

$$Q = \begin{bmatrix} -\theta_1 & \theta_1 & \dots & 0 \\ \vdots & \ddots & \ddots & \vdots \\ 0 & \dots & -\theta_i & \theta_i \\ 0 & \dots & 0 & 0 \end{bmatrix} \tag{4-15}$$

where θ_i is the transition rate between adjacent states. Such rate calculation is based in the record of database inspections that builds the MC model. Its expression is given by:

$$\theta_i = \frac{n_{ij}}{\sum \Delta t_i} \tag{4-16}$$

where n_{ij} is the number of elements that moved from state *i* to state *j* and $\sum \Delta t_i$ is the sum of time intervals between observations, whose initial state is *i*. To improve the quality of fit, the initial Markov model is trained and improved through an optimization process by minimizing the following expression:

$$\sum_{m=1}^{M} \sum_{n=1}^{N} \log{(p_{ij})} \tag{4-17}$$

where m is the number of transitions observed, n is the number of analyzed elements and p_{ij} is the probability of occurrence of observed transition, as predicted by MC model. The probability of a structure being in a certain state *i* at a specific time *t* is given by:

$$p_i(t) = p_0 \times P \tag{4-18}$$

where p_0 is the vector for the initial condition state of the structure and P is given by equation (4.14). The final time dependent condition state is obtained by the average condition state in a certain year and given as follows:

$$\overline{CS}(t) = p_i(t) \times \begin{bmatrix} 1 \\ \vdots \\ \vdots \\ \vdots \\ k \end{bmatrix} \tag{4-19}$$

where $\overline{CS}$ is the time dependent average condition state and 1 to *k*, the total number of condition states. Note that those condition states are integers values being 1 and *k* the best and worst condition state, respectively.

4.4.2 Semi-Markov chains

Semi-Markov chains represents an extension of the Markov processes and generalizes the continuous time MC by allowing the distribution of the time between changes of state of the system to have an arbitrary distribution (non-exponential) [166]. Bridge's real degradation process is closer to semi-Markov chains than to MC, because several factors affect the length of time that elapses before a bridge component deteriorates to its next lower condition state [214], which causes the time between transitions (waiting/sojourn time) to be a random variable and its distribution is governed by the next state the process will enter [215].

There is a considerable amount of literature checking the suitability of different distributions for the sojourn times [216]. The results have demonstrated that the Weibull distribution is the best fit for the waiting time for superstructure components. Additional research in semi-Markov models [215, 217] has shown that the benefit of this approach might be questionable because they tend to shift preservation actions in worse condition state compared to MC models. Furthermore, they pointed out that bridges are rarely allowed to deteriorate into the worst condition states, so the lack of data makes difficult the estimation of sojourn time distribution for those states. As its application is out of the scope of this thesis, a deeper explanation of this formulation can be consulted in [218].

4.4.3 Hidden Markov chains

Another group of Markov chains that are gaining interest on the prediction degradation of infrastructures are the Hidden Markov models (HMM). During these last few years, HMM have been applied in several areas such as voice recognizing, language modelling, recognizing of manuscript words, on-line signature validation, learning of human actions and fail detects in dynamic systems [219]. Normally, an HMM is generally defined by five variables: (i) number of states n, (ii) number of observable symbols m, (iii) state transition probability distribution A, i.e. the transition matrix, (iv) observation symbol probability distribution B; and (v) initial state distribution π. Generally, HMM follows three key problems: (i) what is the probability that a model can generate a sequence of observations?; (ii) what sequence of states best explains the sequence of observations? and (iii) given a set of sequence of observations, how it is learned the model of probabilities that would generate them? For each key problem, different algorithms are applied to solve them. For the first step, to estimate the number of sequences for probability estimation, a forward-backward algorithm is applied to solve the problem to reduce that number of calculations. On the step two, the Viterbi algorithm is employed to determine the best sequence of states that explains better the sequence of observations. As for step three, the calibration of the parameters so the model can learn properly is given by the Baum-Welch algorithm. For the detailed mathematical procedure of these algorithms, the reader is referred to the following works [220, 221].

These models present some applications on civil engineering field being great part of them to solve problem where there are incomplete data. Kobayashi et. al [222] stated that the accuracy of prediction of deteriorating models highly depends on the monitoring data. However, this kind of data contains errors that tend to weaken the prediction. In this way, they presented on their study an HMM approach of a

Japanese national road system to tackle this problem. They also compared the results with the conventional multi-stage exponential Markov Model stating a great improvement when using HMM. Another similar study, where it is investigated the use of exponential HMM, is provided by [223].

Lethan et al. [224] assessed the pavement deteriorating process through HMM, more precisely, to evaluate the current frequency of local damage as well as the degradation of other pavement indicators (e.g., crack, roughness). Later, Monica et al. [225] presented a work of prediction on the condition rating of the deck concrete elements of bridges from the National Bridge Inventory managed by the U.S. department of transportation. From all the models evaluated, HMM revealed to predict a better condition state at the end of the life cycle. The authors claimed that such fact is related to the misclassification by the inspectors of the states, given by condition probability of observations.

4.5 Petri-Nets

Petri-Nets (PNs), originally developed by Carl Petri, are bipartite graphs built with two types of nodes: places and transitions, which are linked by arcs. The execution of a PN is controlled by the position and movement of tokens (denoted by dots) among the places and within the PN [226]. Bridge models have been developed based on PN approach, where tokens represent the bridge or element, places represent its condition, and transitions model the changing state of the system thus replicating the deterioration process.

Research works have demonstrated the appropriateness of PNs to model bridge deterioration [227, 228]. For instance, [227] applied a PN approach for railway bridge management. The model comprised several modules with different purposes, namely element deterioration, inspection and maintenance, and the interaction among them. The deterioration module permitted to model the failure modes independently for each individual component, to consider the several degradation mechanisms that affect the bridges and the different deterioration rates for each defect. On that premise, one of the main advantages of the PN modelling technique is that allows to indicate the maintenance activity required to rectify each condition [228]. Moreover, in the work of Ferreira [229], several applications of petri nets to different types of infrastructures is presented. For further details regarding the approach, see [229].

4.6 Artificial neural networks

Artificial intelligence (AI) techniques have been developed to overcome some of the limitations of the current bridge deterioration models used in BMS, by making the best use of the large database that is periodically updated and proportionate valuable knowledge about bridges that can be employed in predicting their future condition [184]. AI models establish the relationship between the bridge deterioration and the influencing parameters that may affect the bridge condition through computer techniques that aim to automate intelligent behaviors [184]. AI techniques comprise different methods such as artificial neural networks (ANN) and case-based reasoning (CBR).

Many publications have investigated the feasibility of using ANN in modelling infrastructure's deterioration and have proven the superiority in the prediction accuracy [230, 231]. Likewise, CBR approaches have addressed some limitations by making use of a library of previously solved problems when solving a new problem [184]. This implies that the method searches for previous cases similar to the current problem, in their physical features, environmental and operating conditions, inspection and maintenance records, and reuses them to solve the problem [184]. Regarding its formulation, see [184] as its implementation is out of the scope of this thesis.

4.7 Bayesian networks

Bayesian networks (BNs) are probabilistic models in the form of directed acyclic graphs where vertices (nodes) stand for random variables and edges (arcs) symbolize direct influences between nodes [232, 233]. The direct predecessors of a node are called "parents" and conversely "children" are immediate successors of a particular node [232]. For each child variable, a conditional probability table (CPT) needs to be defined linking condition states of the child to the parent variables [234]. Recently, BNs have been employed for modelling bridge deterioration. For instance, [234] represented the overall group condition of a sample group of UK's railway masonry arch bridges through a BN. The methodology was extended to Dynamic Bayesian Networks (DBNs) to introduce a time dependent degradation model. Likewise, [233] developed a DBNs model for predicting the condition of a steel bridge main girder. The applicability of this model for the purpose of deterioration modelling relies on the improved predictive results, the possibility of estimating the CPTs based on expert knowledge, the Bayesian updating capabilities, and the ability to considerate influencing factors of the degradation process (environmental effects, traffic density, maintenance actions) [232, 233].

4.8 Strengths and weaknesses of the degradation models

To summarize, the main advantages and disadvantages of the different approaches that have been proposed for modelling the degradation of infrastructures are hereafter presented in Table 4.1. Analogously to chapter 3, the following sections aim to apply the degradation modelling theory in practical applications: (i) simply supported RC bridge and (ii) steel bridge.

Table 4.1 – Strengths and weaknesses of the degradation models

Type of analysis	Degradation Model	Strengths	Weaknesses
Mechanistic	Steel Corrosion	✓ Formulation simplicity	✓ Only explains the degradation process at the element level
	RC corrosion	✓ Computational efficiency	✓ Definition of the parameters not possible for some cases
Deterministic	Regression models	✓ Simplicity in the formulation [177]	✓ Predict average condition [184]
	Straight-line extrapolation	✓ Based on data from field observations [177]	✓ Neglect the randomness of the variables [184]
	Curve-fitting techniques	✓ Computational efficiency [177]	✓ Inaccuracy for long-term predictions [177]
			✓ No interaction between component's degradation [184]
Random Variable	Reliability index	✓ Parameters associated with deterioration are random [187]	✓ Limited data on the performance of deteriorating structures and the effects of maintenance actions [191]
		✓ Incorporates explicitly reliability assessment [191]	
	Condition index	✓ Consider entire deterioration and maintenance history [4]	✓ Condition and safety are not fully correlated [4]
	Safety index	✓ Incorporates structural assessment [4]	✓ Large data variability and poor reliability [191]
	Non-linear reliability index	✓ Consider the non-linear deterioration effect with time [235]	✓ Due to dispersion in results, it is not sufficient to make the most reliable and cost-effective decisions [4]
Stochastic	Markov Chains	✓ Capture the uncertainties of the deterioration process [194]	✓ Assume discrete condition states, constant inspection period, fixed bridge population, and stationary transition probabilities [236]
		✓ Computational efficiency to manage a large number of facilities, simplicity of use, effectiveness to identify the most deteriorated bridges [194]	✓ Do not consider condition history (memoryless property) [187]
			✓ Transition probabilities are subjective and require updating [184]
		✓ Adequacy to employ the available condition rating data	✓ Severity of element deterioration is described in visual terms only
	Semi Markov Chains	✓ Sojourn time is a random variable [215]	✓ Lack of data makes difficult to estimate sojourn time for worst states [215]
		✓ Capable of modelling the realistic increasing deterioration rates [228]	✓ Preservation actions specified for worse condition state compared to Markov models are questionable [215, 217]
			✓ Exponential increase number of model states as the model complexity increases [237]
	Hidden Markov	✓ Good to incorporate the biases of the information into the model [222]	✓ The formulation of those models is very dense and complex

		✓ They do not hold on the Markov property, i.e. they account the historical information ✓ Good to overcome problems when the dataset is incomplete	✓ On the training process, the optimum result is not global but local instead
Petri-Nets		✓ Manageable size of the analytical model [228] ✓ Considers interaction and dependency between different element deterioration processes [228] ✓ Accounts for previous maintenance actions [237]	✓ Understand the complex deterioration process had to be obtained before calibrating the model [227]
Artificial Intelligence	Artificial neural networks	✓ Superiority in the prediction accuracy ✓ Identify the degree of order to generate the best fitting curve for the data [184]	✓ Large amount of data required ✓ Need to process data to remove effects from previous maintenance actions [184]
	Case-based reasoning	✓ Accounts for past improvement actions [184] ✓ Accounts for past conditions on the predicted ones ✓ Considers the interaction between bridge elements	✓ Issues when the size and coverage of the case library are inadequate [184] ✓ The degree of matching suffers from subjectivity [184]
Bayesian Networks		✓ Expert knowledge can be implemented when historical condition data are insufficient [233] ✓ Possibility to update with new data from inspections, monitoring or maintenance activities [234] ✓ Ability to consider multiple deterioration factors jointly [233]	✓ Their quantification grows rapidly with the number of nodes and states [81]

4.9 Practical case A: simply supported RC bridge

This application regards to the same bridge referred in section 3.5, chapter 3, with focus again on the deck element. For this practical application, two different types of models are going to be implemented: (i) degradation model based on corrosion of the reinforcing bars and (ii) MC degradation model. MC models will be developed based on a database containing inspections about condition state of other bridges. Note that this approach is more qualitative based when compared to the corrosion-based degradation. Thereby the evaluated metrics, i.e. the KPIs, may differ among them. However, it is noteworthy to explore both approaches and draw some insightful comparisons.

4.9.1 Corrosion of the reinforcing bars

The corrosion of RC structures is divided into initiation and propagation phase. However, for this application, the focus was on the propagation phase. Hence, the time zero of the degradation curve was considered as the initiation time for corrosion. The propagation phase was governed by a corrosion rate that, according to Vu and Stewart [171], is given by:

$$i_{corr}(t_p) = i_{corr}(t_0)\ 0.85\ t_p{}^{-0.29}\ [\mu A/cm^2] \tag{4-20}$$

where $i_{corr}(t_0)$ is the corrosion rate at the start of corrosion propagation, t_p is the time since corrosion initiation. The initial corrosion rate is dependent on the environmental conditions. According with Lu et al. [238]:

$$i_{corr}(t_0) = \frac{T_K RH(w/c)}{cover}\ [\mu A/cm^2] \tag{4-21}$$

where T_K is the average temperature in Kelvin, RH is the average relative humidity, w/c is the water-cement ratio, and the concrete cover.

Thus, the loss of reinforcing steel area, based on corrosion rates, according to [239] can be obtained through:

$$A_s(t_p) = \frac{n\pi D_0}{4}\left(D_0 - \left[\frac{(t+0.62)^{0.71} - 0.712}{17.59}\right] i_{corr}(t_p)\right) \tag{4-22}$$

where n is the number of rebars, D_0 is the initial diameter. The previous equation (4-22) was applied assuming the ionic charge z=2 [239].

The corresponding lifetime considered for the analysis was 100 years. A uniform reduction of the reinforcing cross-section area was assumed. Since the pre-stressing tendons have a ticker concrete cover than the reinforcing steel, corrosion in the formers is not governed by carbonation or chloride penetration through the concrete cover. The propagation phase was expressed according to equations (4.20), (4.21) and (4.22). Table 4.2 shows the considered parameters for the propagation phase. The air temperature, as well as the relative humidity, were obtained through the Portuguese Institute for Sea and Atmosphere (IPMA). Based on the parameters obtained for the propagation phase, the time-dependent reliability index is illustrated in Figure 4-2.

Table 4.2 – Parameters considered for corrosion

Parameter	**Distribution**	**Mean Value**	**Standard Deviation**
Air temperature in Kelvin (T_K)	Normal	288	7
Relative Humidity (RH)	Normal	72%	2.2%
Concrete cover (mm)	Normal	30	6
ϕ(mm)	Deterministic	25	-

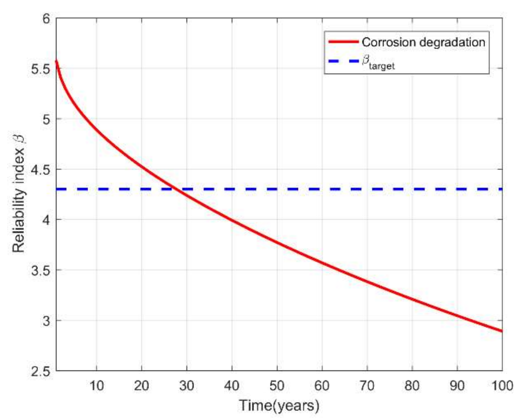


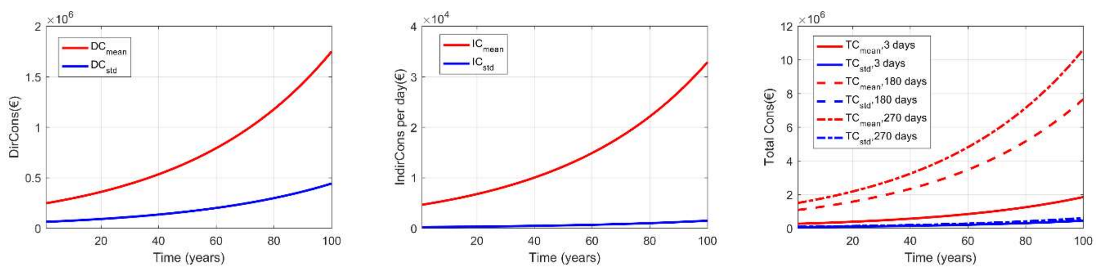
Figure 4-2 – Time-dependent reliability index

The model obtained based on corrosion degradation led to a considerable decay on the reliability index. At the end of year 100 the corresponding reliability was around 2.7, corresponding to a loss of almost 50% of its initial value. Nevertheless, it was also noticed that the threshold reliability index of 4.3, according to EN1990 [107] for a safety period of 50 years with high level of consequences, was crossed at the age of 30 years. Moreover, the following time-dependent risk was computed to investigate the obtained values for the time that the reliability crosses the threshold value. Considering the same conditions as in section 3.5.2, see Table 3.8, the following direct and indirect consequences were again re-estimated. The estimation of the time-dependent consequences in terms of monetary losses must consider the future risk. Accordingly, the future risk is obtained as [18]:

$$FV(t) = PV(1 + r)^t \qquad (4\text{-}23)$$

where *FV* is the future risk value, *PV* the present value and r the annual discount rate, herein assumed as 2%.

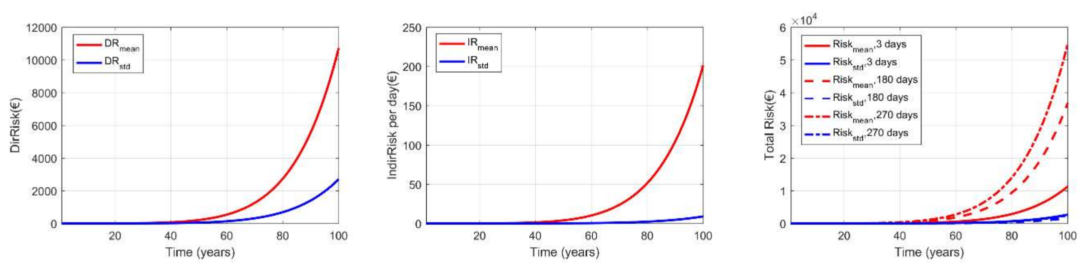
Nevertheless, the obtained indirect consequences were estimated per day. Since the recovery time for certain situations can take longer than one day, for this application, it was assumed three different times of recovery, 3 days for a slight scenario, 180 days for a moderate scenario, and 270 days for a severe scenario. Those recovery times were based on the study of [240] assuming the affected area was local. Accordingly, the direct, indirect, and total consequences are assembled in Figure 4-3



a) b) c)

Figure 4-3 – consequences estimation: a) direct consequences; b) indirect consequences per day; c) total consequences for three different recovery days

Accordingly, by applying the equation (3-9) , the time dependent risk was obtained. Considering the different times of recovery, Figure 4-4 shows the estimated risk for the bridge.



a) b) c)

Figure 4-4 – Risk estimation: a) direct risk; b) indirect risk per day; c) total risk for three different recovery days

It was concluded from the obtained results that the risk started to present serious threats after the age of around 50 years for this application. This corresponded to a reliability index of around 3.50. In this way, despite the threshold value proposed by EN1990 [107], the bridge still presented a very low risk assuming these formulation conditions. Another important aspect that was noteworthy to mention here was the comparison of those risk values with the initial construction value of this bridge. According with the initial design reports, this bridge presented an initial cost of around 630000 €. The comparison in terms of the ratio between the total risk of the bridge and the initial investment comprises a useful metric for the project manager and thus for the process of decision-making. However, it should be considered in here the discount rate over the time on the initial investment for a more realistic approach. Considering the discount rate previously adopted, Figure 4-5 shows those estimations considering the three different periods of recovery. It was observed from Figure 4-5 that such metric started to present serious concerns after the age of 80 years old indicating that some measures such as maintenance or even risk mitigation should be considered to decrease those risk levels. Furthermore, it was noticed that for the slight scenario, the time-dependent ratio was not as serious as the other two scenarios. Once this system, as concluded in the previous section, was highly dependent on the indirect consequences, the recovery time should not be carefully controled.

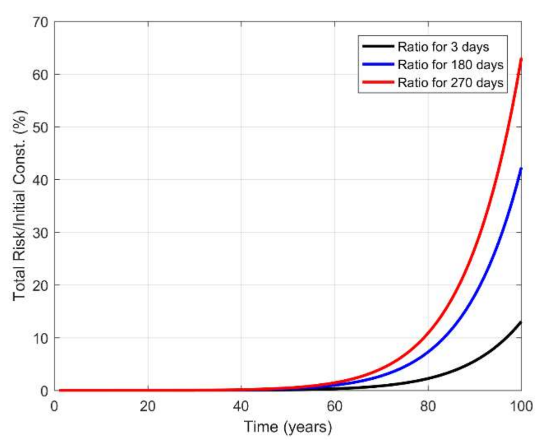


Figure 4-5 – Comparison with total risk and initial construction value for three different recovery times

4.9.2 Markov chain-based degradation model

The following section follows the theoretical formulation described in the section 4.4.1. To build the rate degradation matrix, i.e. the intensity matrix, a database with several records of the condition state of the deck with the same material of the present application was considered. This application adopted the classification systems currently adopted in Portugal [241] wherein the condition state was described by a set of integers from 1 to 5, being condition state 1 and 5 the best and worst, respectively. The present database was composed of the construction year of the element as well as a set of records of inspection containing the date and the actual condition state.

The first step to obtain the MC model was to compute the intensity the matrix given by the transition rates from state *i* to *j*. Those transition rates were obtained through equation (4.16). Note that those rates were optimized through maximum likelihood estimation, see equation (4.17). Accordingly, the obtained transition rates were given as follows:

$$\begin{bmatrix} \theta_1 \\ \theta_2 \\ \theta_3 \\ \theta_4 \end{bmatrix} = \begin{bmatrix} q_{12} \\ q_{23} \\ q_{34} \\ q_{45} \end{bmatrix} = \begin{bmatrix} 0.051 \\ 0.067 \\ 0.623 \\ 0.066 \end{bmatrix}$$

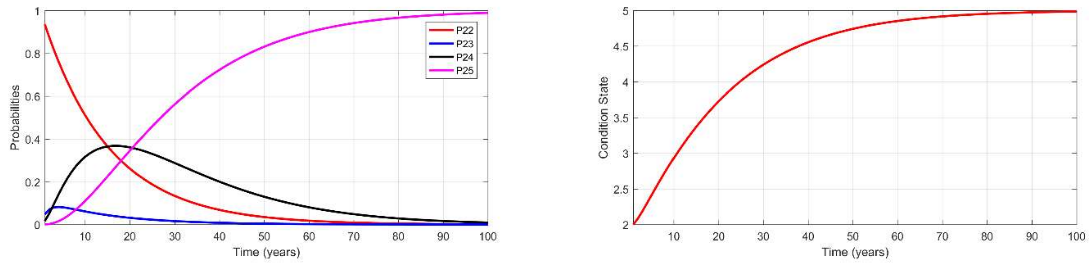
Hence, the intensity matrix was populated as follows:

$$Q = \begin{bmatrix} -0.051 & 0.051 & 0 & 0 & 0 \\ 0 & -0.067 & 0.067 & 0 & 0 \\ 0 & 0 & -0.623 & 0.623 & 0 \\ 0 & 0 & 0 & -0.066 & 0.066 \\ 0 & 0 & 0 & 0 & 0 \end{bmatrix}$$

The second step concerned the estimation of the probability of the element being in a certain state. To obtain that, it was important to know the real condition state of the element considered for this application. As mentioned in section 3.5, chapter 3, the bridge was re-built as new in 2009. The latest report on this bridge on 2013 showed the element deck was already with some sign of efflorescence and few cracks indicating that the corrosion process might have already started. Thus, the condition state value of 2 was given. In the absence of another type of information, the actual condition state of the element was considered to start at the year of 2013. Hence, the initial probability was given as follows:

$$p_0 = [0\ 1\ 0\ 0\ 0]$$

The time dependent probability for the structure being in a specific state *i* and transit to state *j* as well as the average condition state, given by equations (4.18) and (4.19), respectively, are depicted in Figure 4-6.



a) b)

Figure 4-6 – Final condition state for the RC deck: a) transition probabilities considering the initial condition state 2; b) final average condition state

It was observed from Figure 4-6a) that the probability for the structure being in state 2 decreased for almost 100% to 25% in 20 years. As the probability of being in state 2 decreased, the probability to transit for state 4 increased. It was noticed from this figure that the probability transition to state 2 for state 3 was very reduced when compared to the others. This fact was given by the short number of records on the database from state 2 to state 3. For this reason, the average condition state of this element deck considering the historical inspection to other similar decks reached the state 4 at the age of 30 years old. This should raise a warning for the project managers to carefully analyze this situation and consider establishing some maintenance activities. However, it is worthwhile to mention here that this condition state estimation is purely stochastic and is highly dependent on other records. Thus, as the database increases, new conditions states, resulting from new inspections, are added influencing then the degradation rate.

4.10 Practical case B: steel bridge

The same procedure, as previously presented in section 4.9, is herein applied, based on the steel bridge previously described in chapter 3. The first analysis concerns the empirical formulation proposed by [168] and [169], based on corrosion of steel structures, and the second concerns on the application of the MC models, based on inspection reports about similar steel bridges as the presented practical application.

4.10.1 Corrosion of the steel elements

For this application, the corrosion calculation was obtained by estimating the corrosion penetration given by the equation (4.3). As stated in section 4.1, this equation was dependent of two parameters that accounted the different type of environment. Table 4.3 shows those different values according with [168]:

Table 4.3 – Parameters for steel corrosion according with the environmental conditions

Environment	Unprotected carbon steel		Weathering steel	
	A	B	A	B
Rural	34,0	0,65	33,3	0,50
Urban	80,2	0,59	50,7	0,57
Marine	70,6	0,79	40,2	0,56

For the present study, the bridge was assumed to be exposed in a rural environment and unprotected carbon steel. Therefore, equation (4.3) was given as follows:

$$C(t) = 34.0t^{0.65} \tag{4-24}$$

According with section 3.6.5, chapter 3, the reliability index obtained was given by the minimum reliability index given the truss system is isostatic, i.e. the bar [dD] with a corresponding value of 4.87. Thus, the corrosion estimation for the system was represented by the bar [dD] for further analysis. Note that the calculations were assumed by considering the beginning of the degradation for 1990, i.e. the renewal year. Like the previous application, this one was focused on the propagation phase rather than the initiation phase. In this way, the degradation curve was developed assuming the initial year as the year that corrosion started. Although the initiation phase was not addressed in here, it was assumed a time for the initiation of the process. In the study of [242], the authors claimed a rate of corrosion of practically zero between 10 and 15 years for steel bridges. Bearing this in mind, a value ranging that interval could be reasonable assumed in here. Considering all these assumptions, Figure 4-7 shows the time-dependent reliability index. The estimated reliability index showed a considerable drop over the time. It was observed that the threshold value, preconized by the EN 1990 [107], was crossed after 20 years of starting the corrosion. The final obtained value, after 100 years, was 2.50, around 50% of its initial value. This issue appoints for some potential problems that the bridge might present in terms of performance and safety.

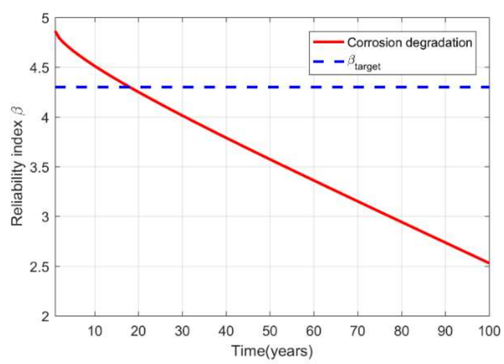
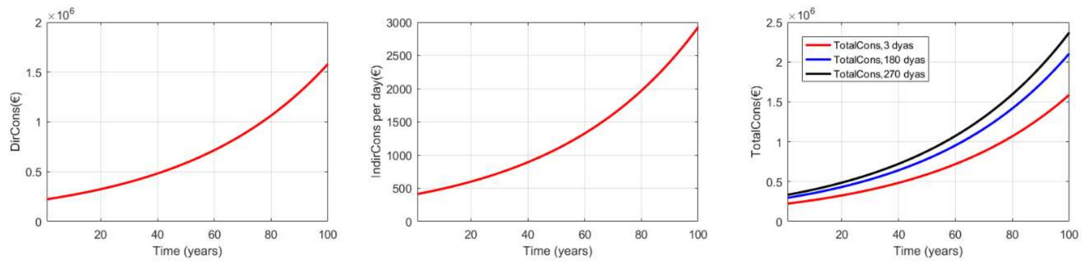


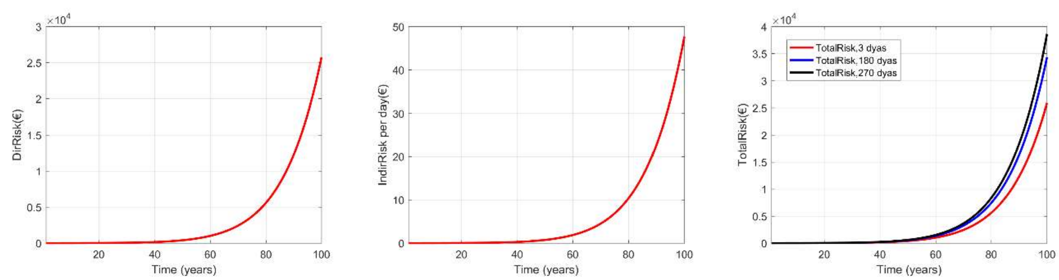
Figure 4-7 – Time dependent reliability index

Furthermore, the time-dependent risk was herein obtained considering the same procedure as the previous section. According with the parameters defined in Table 3.14, equation (3-9) and equation (4.23), Figure 4-8 and Figure 4-9 show the consequences and the obtained risk, respectively.



a) b) c)

Figure 4-8 - consequences estimation: a) direct consequences; b) indirect consequences per day; c) total consequences for three different recovery days



a) b) c)

Figure 4-9 – Risk estimation: a) direct risk; b) indirect risk per day; c) total risk for three different recovery days

The risk curves depicted in Figure 4-9a), b) and c) showed a very low value until the age of 60 years. Moreover, the total risk observed in Figure 4-9 showed a low dependency on the indirect risk in comparison with the previous application. For this reason, the values for the total risk considering different types of recovery were somehow similar among each other. This outcome was consistent with the calculation of the robustness index in section 3.6.7. Furthermore, a comparison of the total risk and the initial cost of the bridge, considering the reconstruction year of 1990, was made. According with the bridge reports, the renewal of the bridge was roughly estimated on 205000 euros. Figure 4-10 depicts the time dependent ratio estimation. From the Figure 4-10, it was observed that those ratio values presented a considerable increase after the age of 90 years. In fact, for cases with higher recovery time, the ratio started to present values over 100% denoting that the risk started to be considerably higher that the value invested on the initial reconstruction. Hence, establishing maintenance activities constitutes an essential measure to avoid reaching those values.

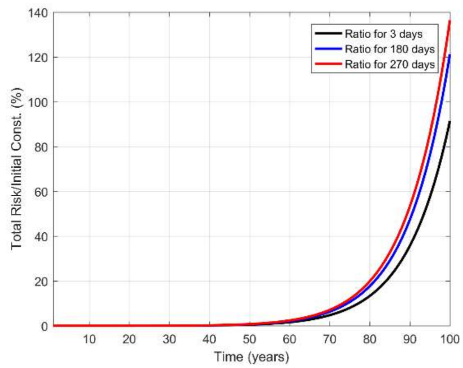


Figure 4-10 - Comparison with total risk and initial construction value
for three different recovery times

4.10.2 Markov chain-based degradation model

The same formulation described on section 4.9.2 was applied in here. Note that the database adopted for this application followed the same principles as the database presented in the previous practical application but considering steel bridge element records. The intensity matrix was given by:

$$\begin{bmatrix}\theta_1\\\theta_2\\\theta_3\\\theta_4\end{bmatrix}=\begin{bmatrix}q_{12}\\q_{23}\\q_{34}\\q_{45}\end{bmatrix}=\begin{bmatrix}0.187\\0.169\\0.330\\0.01\end{bmatrix}$$

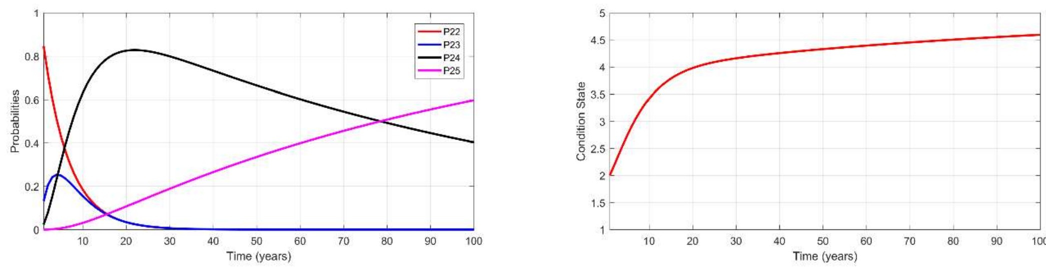
The intensity matrix was populated as follows:

$$Q=\begin{bmatrix}-0.187 & 0.187 & 0 & 0 & 0\\ 0 & -0.169 & 0.169 & 0 & 0\\ 0 & 0 & -0.330 & 0.330 & 0\\ 0 & 0 & 0 & -0.01 & 0.01\\ 0 & 0 & 0 & 0 & 0\end{bmatrix}$$

The reports of the bridge inspection showed signs of degradation in many points of the elements being attributed a current condition state of 2 at the year of 2016. In the absence of further information, the degradation curve considered the initial year of 2016. Thus, the initial probability condition vector was given by:

$$p_0 = [0\ 1\ 0\ 0\ 0]$$

The corresponding time dependent probability of transition as well as the average condition state are assembled, according with equations (4.18) and (4.19), in Figure 4-11.



a) b)

Figure 4-11 - Final condition state for the steel bars: a) transition probabilities considering the initial condition state 2; b) final average condition state

Observing the Figure 4-11 a), the probability of structure being in the states 2 and 3 lasted approximately 10 years. This fact was also observed in the Figure 4-11 b) wherein at the end of the year 10, the average condition state was around 3.50. The condition state increased rapidly to 4 (around year 20) once the probability of being in that state was very high (around of 80%). Considering that very few inspections verified the transition of state 4 to state 5, this highlights the fact of the curve never reached properly that condition state. Nevertheless, as stated in the previous application, those curves could suffer continuous variations on the condition as the database is fed with more inspections that lead to a more accurate measure.

4.11 Discussion of the obtained models

Two different models were obtained on the previous practical applications aiming to estimate the time dependent condition state. Figure 4-12 resumes the final obtained models for this practical application.

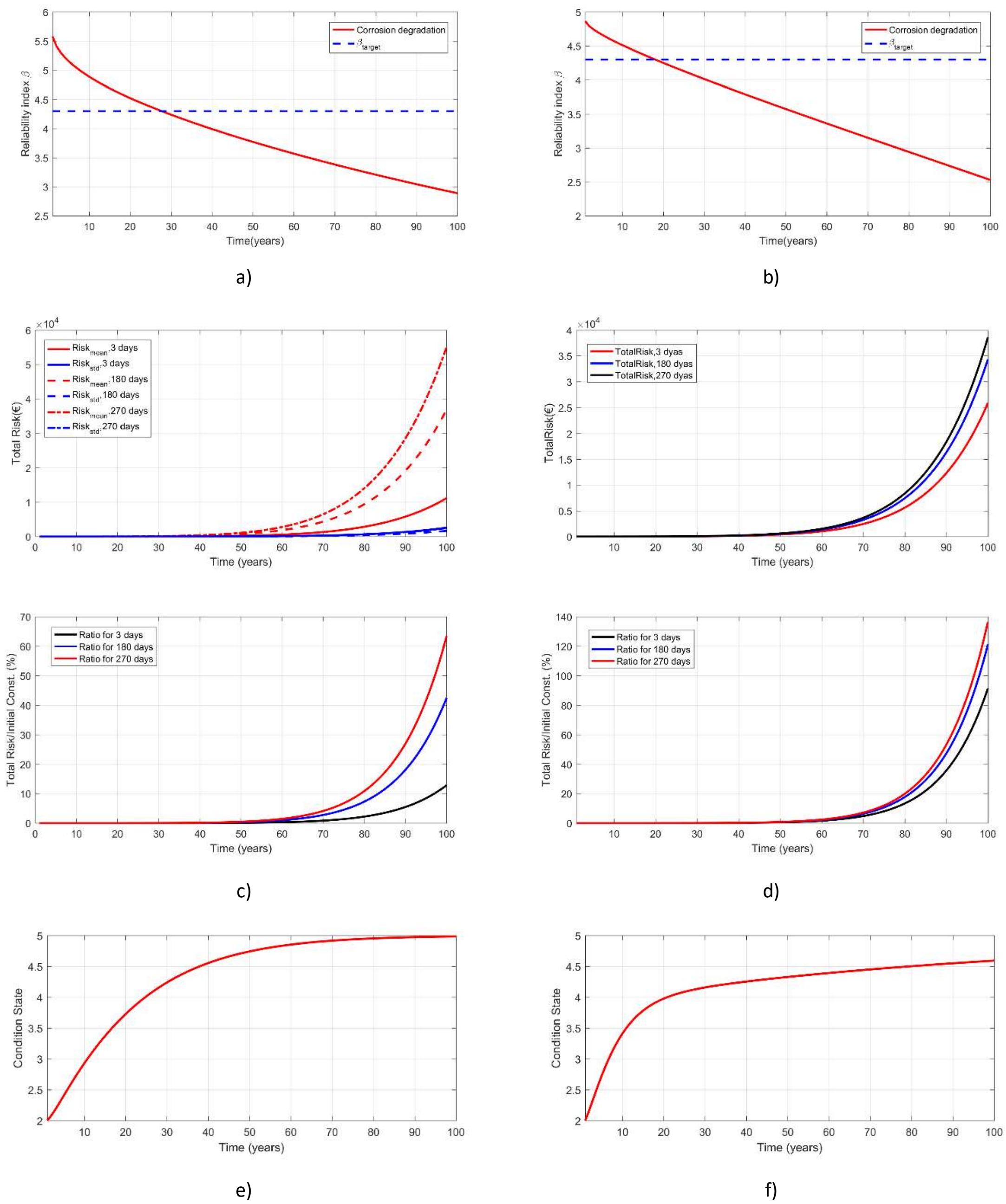


Figure 4-12 – Resume of the models considering the two presented fornulations: a) reliability for RC deck; b) reliability for steel bars; c) risk and risk ratio for RC deck; d) risk and risk ratio for steel bars; e) qualitative condition state for RC deck; f) qualitative condition state for steel bars

The results revealed interesting insights by pointing issues regarding the definition of strategies of maintenance and mitigation after reaching certain levels of performance. However, those models are difficult to compare in terms of being better or worse on predicting the actual state. Such fact relies on its formulation and the level of uncertainty. While reliability and risk calculation rely on empirical models, the qualitative estimation of the condition state relies on stochastic models based on inspection reports.

Regarding the later, the inspection reports are very uncertain once they are dependent on the skill of the inspector and on what he is concerned to assess. In other words, such estimation is based on qualitative assessment belonging to level zero scale according with SAMCO [80]. Concerning the former, the empirical formulations combined with probabilistic assessment tend to provide a more exact and reliable result. Yet, those results can lead to some uncertainties once the conditions measured to build the empirical formulations might not match exactly the reality of the practical application. An alternative for a better assessment would be the combination of those models for a more accurate consideration when defining the maintenance and mitigation actions. This approach was suggested by Neves and Frangopol [193] as stated in section 4.3. However, this application goes a bit further by proposing an analysis not only based on the condition state and safety index but also by considering the risk measure on the problem formulation providing thus a more detailed approach.

4.12 Final remarks

The present chapter aimed to describe the state of the art on the degradation modelling of infrastructures. Several models were discussed in terms of formulation as well as its applications by several authors over the literature. It was concluded that the most employed degradation model on infrastructures still relies on the MC models despite their limitations as presented in Table 4.1. Regarding the practical applications, two different models were applied, an empirical and qualitative based, respectively. The results provided a very useful information to understand the process of degradation that the element was subjected as well as how that degradation could influence on the decision of maintenance activities.

Regarding the first model applied, two different KPIs were estimated, reliability and risk. Concerning the reliability, it was observed that the threshold value according with EN 1990 [107] was crossed at the age of 30 and 20 years since the starting time of corrosion, respectively for the two practical applications. However, according with the assumptions preconized for these applications, the risk values showed to be practically zero at those ages. In fact, the risk started to present a considerable threat at the age of 70 years old. Moreover, to understand how serious the risk for those practical applications could be, a comparison was made between the obtained risk and the initial investment on the bridge renewal.

In what concerns the second model, a qualitative KPI based on the condition rating from 1 to 5, was estimated. Unlike the first type of model, the second was built based on reports provided by visual inspections. Such assumptions raised uncertainties on the model, mainly related to the skill of the inspector and on the number of records registered, i.e. the length of the dataset. In fact, those uncertainties were clearly observed when the calculation of the transition probabilities to build the average condition state. Concerning the uncertainties of both models, an alternative proposal of assessment relied on the combination of those two models, and visualize the degradation rate considering both quantitative, i.e. reliability and risk based models, and qualitative, i.e. condition index models based on expert opinion. Nevertheless, it should be noted that quantitative models are also associated to uncertainties and it should be accounted on the analysis of the degradation models. The next chapter aims to discuss the topics related to the life-cycle cost analysis of infrastructures, maintenance activities and optimization of the costs while performing those maintenance.

Chapter 5

5 Life cycle analysis

Life-cycle analysis of an infrastructure is a financial supporting tool since it allows to the project manager an estimation of costs at a long term. This analysis can be used for several phases of the lifetime of the system: (i) design; (ii) maintenance; (iii) end-of-life. According to Sanchez-Silva [243], life-cycle analysis can be defined as *"project evaluation strategy directed to assess the environmental and/or economic impacts of a product or service throughout its lifetime".* To achieve sustainability on the infrastructures, the life-cycle analysis combines three different areas: economic, social, and environmental. Sustainability is a very important indicator to have into account in a life cycle of large infrastructures systems since they have a strong impact on the social-economic growth of a society. It has become an important issue in the 1980s after the release of a report by the World Commission on Environmental and Development [244]. In this report, sustainability was defined as *"Development that meets the needs of the present without compromising the ability of future generations to meet their own needs".*

Furthermore, several works regarding the developments on the sustainability area have been developed. For example, Zavadskas et al. [245] presented an overview of sustainable decision making in civil engineering referring that between 2013 and 2017, there were a great number of contributions on the topic. Kloepffer [246] summarized the life cycle assessment state of the art and proposed different approaches of calculating the sustainability by the three areas, above mentioned, in a separate way. In Elkington [247] it was proposed seven performance measures:(i) public safety, public health, public security, social equity for social area; (ii) lifecycle cost, asset value, service level and functionality for economical area and (iii) emissions, resource conservation and climate change adoption for environmental area. In Bocchini et al. [248], there were defined the aspects of sustainability by four levels being the level 4 composed by single indicators, level 3 the categorization of indicator, level 2 the three dimensions for the areas and the level 1 the sustainability itself as a role model. Concerning the studies on the infrastructures, for example, Gervásio et al. [249] proposed a work of comparison of alternative for bridge taking into account economic, social and environmental analysis. Dong et al. [19] presented a work to assess the sustainability over a life cycle of a bridge under multi-hazards. The approach included hazards such as floods and earthquake. For the sustainability, it was considered social, environmental, and economic including fatalities, CO_2 emissions and energy waste and expected losses in terms of money. Dong et al. [250] presented a work to a bridge network to study the sustainability under a seismic region to assess the hazard effects. The same authors also developed a work on the multi-objective retrofit optimization of bridge network having into account in the sustainability and cost of retrofit actions in the conflictive objectives. Sabrina Engert [251] presented a literature review on the most important issues for incorporating sustainability into a strategic management. Zoubir Lounis [252] discussed also the topic of sustainability by proposing a work with two examples of application, the first based on risk and the second based on resilience to a seismic event. In the following Table 5.1, an overview of the three areas of sustainability is presented.

Table 5.1 – Life cycle measures

		Definition	**Variables/indicators involved**	
Life cycle costs	Agency costs	Agency costs are the costs that are concerned to the owner during the entire life of an asset.	Construction/Design costs: ✓ Material acquisition and transportation; ✓ Construction equipment; ✓ Man-power cost	Maintenance costs ✓ Inspection costs ✓ Preventive and essential maintenance costs End-of-life costs ✓ Demolition and removal costs
	User Costs	"The consequences on human populations of any public or private actions that alter the ways in which people live, work, play, relate to one another, organize themselves so as to meet their needs and generally cope as members of society"[253]	Vehicle Operating Costs Traffic Delay Costs Traffic Accident Costs	
	Society, Environmental costs/ Life cycle Assessment (LCA)	"LCA is a standardized and systematic method that evaluates the potential environmental impacts of a product or a service throughout its whole life cycle, from raw material acquisition, manufacture, use and maintenance till the end of the life of its function" [254]	Noise and Aesthetic Costs Abiotic depletion Global warming Ozone depletion	Photo oxidant formation Acidification Eutrophication Ecotoxicity Human Toxicity Waste production

5.1 Life cycle agency costs

As presented in Table 5.1, life cycle agency costs (LCAC) can be roughly divided in three main groups: (i) Construction Cost; (ii) Operation Cost and (iii) End-of-life cost. The construction cost refers to the process related to the design of the bridge that covers the cost of material acquisition and transportation, cost of construction equipment and cost of man-power. The operation cost includes the inspections, maintenance, and repair actions. The end-of-life cost concerns issues about demolition cost and residual value. The residual value here is intended to the real value of demolished materials. In the LCAC calculation, one of the most popular approaches to define a financial/economic framework is to apply the Net Present Value (NPV) procedure. Basically, NPV can convert cost at different times to current costs by using a discount cash flow method. Generally, the calculation is given by:

$$LCAC = \sum_{t=0}^{n} \frac{C_t}{(1+r)^t} \tag{5-1}$$

where C_t is the sum of all relevant costs and r is the discount rate. The adoption of a reasonable value for the discount rate is difficult as there is no unique way to define it. According to the different problems those values can change. For example, for the case of infrastructures, those values can range from 0.9% to 2.5% while for financial market, those values can range between 2 and 8% [243]. Those different values of discount rates are also concerned with the period of analysis of a given problem. Given that infrastructures tend to by designed for larger periods of time, high discount rates would devalue the initial value at long term.

For this thesis, the cost calculation is focused on the operations costs, i.e. the costs related with inspections and maintenance activities. Inspection activities are not directly involved in the maintenance of the infrastructures in terms of reducing the rate of degradation. However, to prevent a complete lack of monitorization of the bridge, inspection practices are employed. Those inspection actions are dependent on several factors such as the condition of the bridge, the type of inspection, skill of the inspector, the type of material, among others. Nielsen et al. [255] proposes in their work a reasonable estimation of the costs of inspection, given by the following equation:

$$C_{insp}(t) = \sum_{t=1}^{t=n} \left(\left[\frac{2d}{80} + \left(\frac{(20+0.5L)H \times S \times I \times M}{60} \right) \right] \times (C_l + C_v) \times \frac{1}{(1+r)^t} \right) \tag{5-2}$$

where n is the horizon time, d is the distance from depot in km, L is the length of the bridge, H the condition of the bridge, S the skill of the inspector, I the inspection type, M the bridge material, C_l the labor costs (€/h), C_v the vehicle costs (€/h) and r the discount rate.

Concerning maintenance actions, they are carried out to slow the degradation rate. The quantification on the maintenance costs for one specific year are given by the following equation:

$$MC_{t,nom} = \sum_{i=n}^{m} AUC_i \times Aq_i \times \psi \times \frac{1}{(1+r)^t} \tag{5-3}$$

where $MC_{t,nom}$ is the nominal maintenance costs for year t (€), i is the activity n until m, AUC_i is the unit cost of activity (€/unit), Aq_i the quantity of units for activity i in year t (unit) and ψ is reduction factor of the costs according with the condition state of the structure as proposed in [149].

The maintenance activities can be roughly classified into two groups: (i) preventive maintenance, i.e. activities to slow the degradation rate, and (ii) essential maintenance, i.e. an instant improvement on the condition state of the infrastructure [256]. Figure 5-1 shows an example of application of preventive and essential maintenance activities.

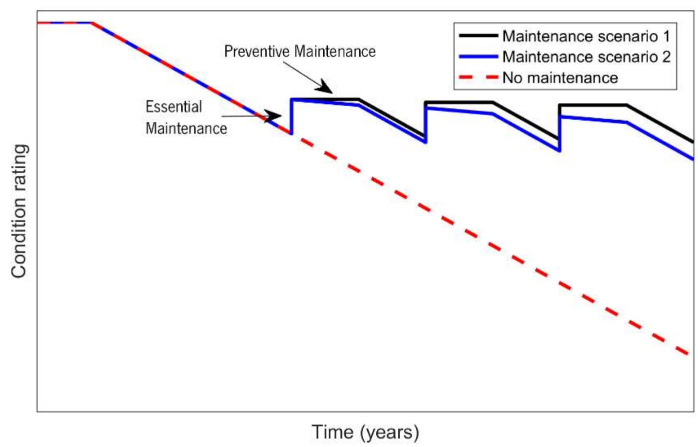


Figure 5-1 – Example of application of preventive and essential maintenance activities

5.2 Life cycle user costs

Social assessment represents the area that is less taken into account while comparing to the economic and environmental [257, 258]. As stated by International Committee on Guidelines and Principles [253], social impacts are defined as "*the consequences on human populations of any public or private actions that alter the ways in which people live, work, play, relate to one another, organise themselves so as to meet their needs and generally cope as members of society".* Di Cesare [259] conducted a literature review on the positive impacts of life cycle social assessment (LCSA) stating that there was an effort to express the social indicators as quantitative variables. Rodolfo Vasquez [260] conducted a work for the definition of a practical guide for the implementation of social sustainability in the construction projects during the design and planning phase. Regarding bridges, there are specific social indicators that are of great importance to be analysed. For example. Gervásio [261] defined mandatory indicators namely vehicle operating costs, traffic delay costs and traffic accident costs, and optional indicators such as noise and aesthetics. Thof Christensen [262] presented in his work several reports of consideration of user costs and argued that in most of the cases, life cycle agency costs were ten times lower than the user costs.

There are proposed in the literature measures to obtain these costs. Sundquist and Karoumi [263] proposed equations to obtain measures for vehicle operating, traffic delay and traffic accident costs. These costs were the result of work zones that were associated with the construction and maintenance of the infrastructure during its life-cycle. In addition, for railway infrastructures there were also some works presenting the estimation of the user costs. For example, in the work of Almeida [149], the maintenance activity costs, for the application of railway bridges, is given by the following expression:

$$Ind_{Cost}(t) = \sum_{t=1}^{n}[VA_v \times DUR \times TMD \times PER \times (ATR + ATF)] \times \frac{1}{(1+r)^t} \quad (5\text{-}4)$$

where VA_v are the delay costs (€/min) that the infrastructure company must pay to the train operator in case of maintenance activities and obtained according with an asset owner, DUR is the duration of the maintenance activity given in days, TMD is the average daily traffic of trains, PER is the traffic conditioning duration of the bridge related to the preventive intervention, ATR and ATF and delays related to the speed reductions and braking, respectively. Those parameters are herein estimated according with [264] and given by:

$$ATR = 60 \times (BL + 0.15) \times \left(\frac{1}{S_r} - \frac{1}{S_n}\right) \quad (5\text{-}5)$$

$$ATF = \frac{1000}{60 \times 60 \times 60} \times (S_r - S_n) \times (2.2 - 0.0105 \times S_r) \quad (5\text{-}6)$$

where S_r and S_n are the reduced and normal speed in km/h, respectively, BL is the bridge length, in km, added of 150 meters wherein there is the reduced speed. Note that the numerical member of the equations the serves the purpose of converting the units from kilometers to meters and the time units of hours to minutes.

Furthermore, in the work of Nielson [255], it is proposed an estimation loss during maintenance activities on the railway track wherein the time loss due to the activities is given by:

$$\mathrm{T_l} = (\mathrm{l}_r + (3 \times \mathrm{l}_t)) \times \left(\frac{60}{S_r} - \frac{60}{S_n}\right) \quad (5\text{-}7)$$

where $\mathrm{T_l}$ is the time lost due to the speed restriction (minutes), $\mathrm{l_r}$ is the length of the restricted track (Km), $\mathrm{l_t}$ is the train length (Km), $\mathrm{S_r}$ is the restricted speed (Km/h) and $\mathrm{S_n}$ is the normal speed (Km/h).

5.3 Discussion of LCAC, LCSA, LCA works applied in the civil engineering field

In the civil engineering field, mainly on bridges, there are numerous works conducted by on the LCAC, LCSA and LCA topics. In what concerns the LCAC, all the works focused mainly the direct inspection, maintenance and/or replacement the components at the infrastructure and network level. Those works can be seen in [265], [266], [267], [255, 268], [269], [270], [271], [14].

Regarding LCSA, the literature has shown that there were still few studies in the field of bridges regarding social analysis and the costs for the society along the life cycle. In the work of Orcesi and Cremona [205], it was proposed a network bridge management system considering the position of each bridge. They used the visual inspections to assess the performance of the bridge. While defining the optimization problem to search for the optimal strategies, the user and owner interests were taken as two conflictive objectives; Gervásio and Silva [272] quantified the user costs along the life cycle of bridges by probabilistic analysis. Social indicators adopted in the study were: (i) Drivers delay cost; (ii) Vehicle operation costs; (iii) Accident costs; Navarro et al. [273] conducted a Reliability-based durability evaluation on the maintenance procedure (corrosion assessment) and the analysis of the social impact on the life cycle of the bridges. The social indicators under analysis were the workers, society, consumer, and the local community.

Concerning LCA, the literature denoted different topics under study. The first regarded the comparison among different alternatives to bridges, their costs along the life cycle and their environmental impact, see the works of [274], [275], [276] and [277]. The second was similar to the first but the comparison was made in terms of components of the bridges. Studies regarding these comparisons can be seen in [278], [279], [280], [281], [282] , [283] and [284]. The third compared the adoption of new materials such as high-performance concrete and wood, see [285] and [286]. The fourth study concerned the environmental impact of CO2 emissions under natural hazards such as earthquakes, see [287] and [288].

After a review of these studies, it was concluded that all these issues, regarding quantification of costs along the life cycle, were focused more on roadway bridges than railway bridges. The same occurred with the type of materials adopted. Majority of the concerned the reinforced concrete structures, steel structures and composite concrete-steel structures. Masonry and timber structures were the less investigated.

5.4 Optimization and decision-making process

Optimization denotes a very important part of a management system since it is responsible to provide several different optimal plans along the life cycle for the decision-making process. The complexity of the optimization problem depends on the level of analysis (e.g., member level, infrastructure level, among others). There are several techniques to solve optimization problems. In real problems, the optimization problem involves more than one objective. Therefore, the problem turns into a multi objective optimization (MOP). Regarding its mathematical formulation, MOP can be stated as:

$$minimize\ f(x) = \left(f_1(x), \ldots, f_s(x)\right)\ where\ x \in\ \Omega \tag{5-8}$$

subjected to:

$$g_j(x) \geq 0, with\ j = 1, \ldots, m$$
$$h_i(x) \geq 0, with\ i = m+1, \ldots, m+p$$

where x is the decision variable vector, $f(x)$ it the objective function vector, $g(x)$ is the inequality type restriction vector and $h(x)$ is the equality type restriction vector. Commonly in MOP, conflicting objectives is very likely to occur, and the problem can be formulated into three different types: (i) minimize all objectives; (ii) maximize all objectives; (iii) minimize/maximize some objectives [289]. In these types of problems, there are no single optimal solution but a set of optimal solutions that define the so-called Pareto optimal front, a set of tradeoffs that represent different compromises between the objectives. The Pareto front contains all the non-dominated solutions of a MOP, meaning that none of them is better than another. An illustration of a MOP and the optimal Pareto front can be seen in Figure 5-2.

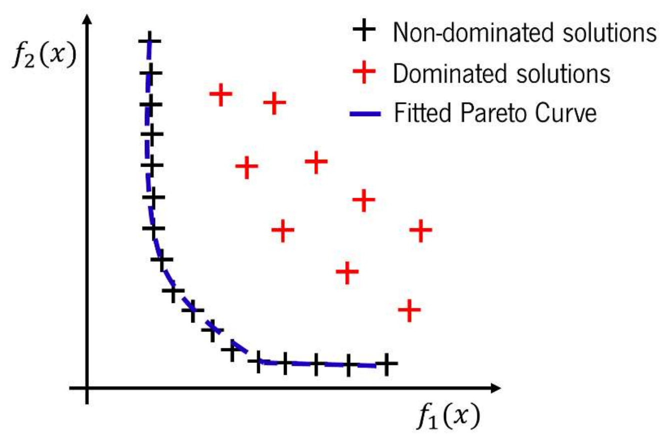


Figure 5-2 – Pareto front example

There are several ways of solving an optimization problem: (i) the traditional algorithms and (ii) evolutionary algorithms (EA). Thorough concern will be given to EA since they will be applied in this thesis. Nevertheless, for further details on the mathematical formulations and practical applications of the traditional algorithms, the reader is referred to [290].

EA have been gaining high interest in the field of engineering during these recent years. EA are used to solve problems that adopt computational models based on evolution mechanisms. These evolution mechanisms are related to biological evolution processes [289]. EA presents the following features: (i) set of individuals that represent potential solutions maintained in a population; (ii) selection mechanism enhancing the survival of the individuals which represent best quality solutions for the problem; (iii) exploring mechanisms of the space of search (and diversity control). As traditional algorithms, EA are solved by iterative processes. The process, normally, initiates by an approximation to the optimal solution. The iterative process is stopped once the criteria pre-defined is achieved. Almost all the algorithms use the objective function as well as the restrictions to the problem. Table 5.2 presents some differences between the two types of algorithms.

Table 5.2 – EA vs Traditional algorithms

EA	**Traditional**
Search, in parallel, with a basis of a population of potential solutions	Sequential search with a basis solution on search fields
Does not requires more information than objective function and restrictions	Can be required more information than objective function and restrictions
Initial population	Initial approximation
No conditions due convexity/differentiability	Can demand conditions due convexity/differentiability
Probabilistic search rules	Deterministic search rules

5.4.1 Genetic algorithms

Genetic Algorithms (GAs) are one of the examples of EA that can be applied to solve the optimization problem in field of bridge management. GAs are a heuristic optimization technique based on Darwin's natural selection and evolution being a robust method capable of solving optimization problems of great complexity and dimension, presenting a great advantage when the number of combinations is extremely large, since it allows to limit the search of a value closest to the optimum. The main strengths of GA's that can be mentioned are: (i) the efficiency in producing good solutions for difficult combinatorial optimization problems and the potential capability to converge to the optimum; (ii) the reduced probability of the solution to converge to a local minima; (iii) the exemption to express the objective function, constraints, or any other problem parameters in mathematical form and its high adaptability to different types of problems [291]. However, it is worth to mention that GA leads to solution closest to the optimum and not properly to the optimal solution. Another drawback is the high time-computing. Regarding its differences between other optimization processes, Goldberg [292] pointed out four main differences: (i)

GAs work with a coding of the parameter set, not the parameters themselves; (ii) GAs search from a population of points, not a single point; (iii) GAs use payoff (objective function) information, not derivate or other auxiliary knowledge and, finally, (iv) GAs use probabilistic transition rules, not deterministic rules. The GA algorithm initializes with an initial population that, according with the fitness function, will be evaluated through steps such as selection, crossover, and mutation. Figure 5-3 illustrates the GA flowchart.

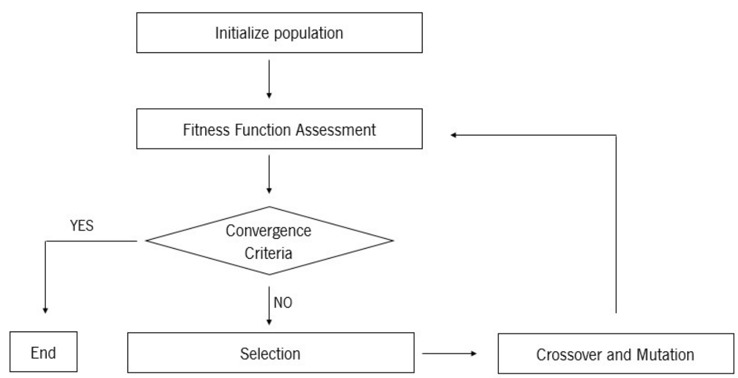


Figure 5-3 – Flowchart GA algorithm

Regarding the population reproduction, each group of individuals represents a candidate solution that is evaluated according with the objective function. The main objective of the reproduction it to highlight the good candidates and eliminate bad solutions. This operation is covered by the following steps: (i) identification of the good solutions; (ii) perform multiple copies of good solutions; (iii) eliminate the bad solutions from the population in way the good solutions can be placed in the population [293]. To solve the above-mentioned steps there are applied several methods such as tournament selection, proportionate selection, ranking selection. In the crossover operator, the genetic code of parents is combined to obtain the child. The genetic code of the parents can be split by different ways to generate the children. Figure 5-4 shows the crossover mutation obtained by a single point. Mutation operator is responsible to impose variations on the population, by changing on the string 0 to 1, to avoid the algorithm to be trapped and to avoid reaching local minimum, see Figure 5-4.

The stopping criteria process for the described steps is achieved in the following situations: (i) number of generations, (ii) computation time, (iii) fitness value, (iv) difference on the average fitting between generations is less than a specific value and (v) the average change in the spread of Pareto solutions is less than a specific value [294].

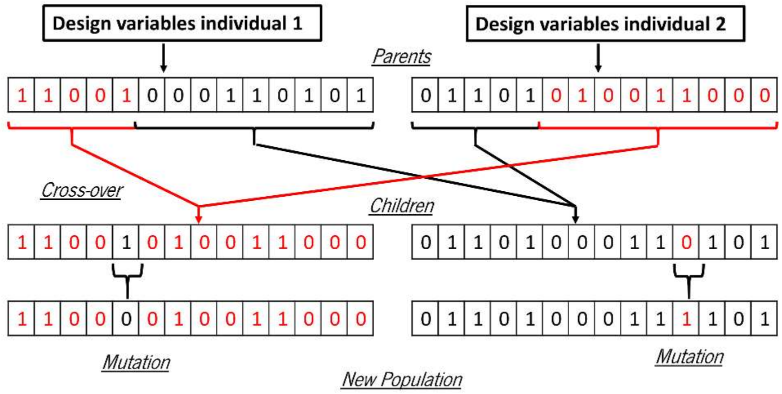


Figure 5-4 – GA Operations (adapted from [295])

The literature review presents several works on the GAs field wherein different types of EA are used to solve the engineering problems. For a thorough a comprehensive understating of the several EA applications, the reader is referred to [296] , wherein a state of the art on multi objective evolutionary algorithms (MOEA) is discussed. Further applications of MOEA can be consulted in [297]. For this thesis, the non-dominated-sorting genetic algorithm (NSGA) II is herein applied. NSGA II is one of the most well-known and used on implementations for MOP. It is fast sorting and elite multi objective GA and presents three special characteristics, fast non-denominated sorting approach, fast crowded distance estimation procedure and simple crowded comparison operator. Basically, this algorithm covers the following steps: (i) population initiation; (ii) non-domination sort; (iii) crowding distance; (iv) selection; (v) genetic operators and (vi) recommendation and selection [298]. Figure 5-5 illustrates a schematic functionality of the algorithm. For a deeper understanding of the algorithm, the readers are referred to [299] and [300].

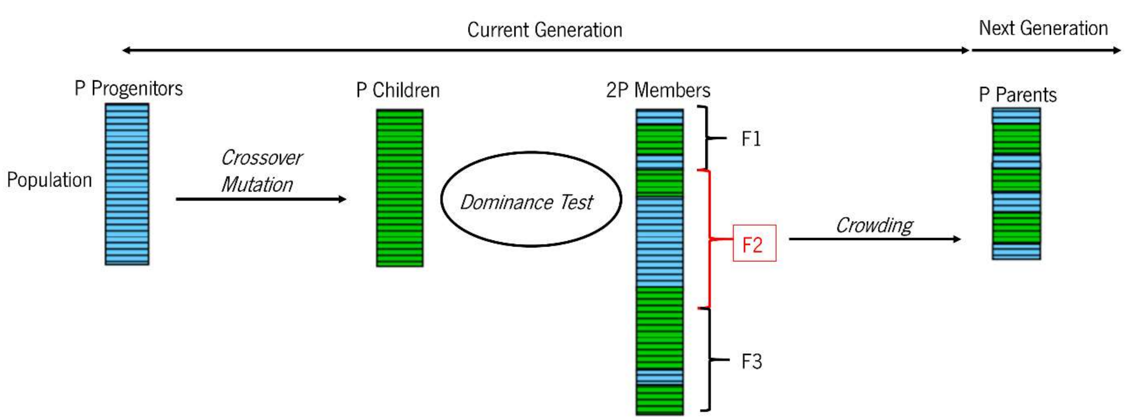


Figure 5-5 – Schematic representation of NSGA II steps (adapted from [301])

5.4.2 MOP works in the field of civil engineering based on GA

There are two different phases in the process of optimization, the phase which generates optimal solutions for the problem under analysis and the phase of decision-making that corresponds to the selection of the solution considering the objectives of the problem. A great number of studies considering life-cycle optimization of infrastructures were developed with the aim of presenting the best scenarios of maintenance and rehabilitation (M&R) [266, 269, 302-315]. After reviewing these studies, all of them aimed to solve the problem of having conflict objectives: the performance of the infrastructure and the investment to keep the infrastructure on a desirable level, i.e. life cycle costs. The most typical studies only considered two objectives. However, some exceptions could be found on the studies of Sunyong Kim and Frangopol [314] in which six objectives were considered: maximizing the probability of fatigue crack damage detection, minimizing the expected fatigue crack damage detection delay, minimizing the expected repair delay, minimizing the damage detection time-based probability of failure, maximizing the expected extended service life, and minimizing the expected life-cycle cost; Frangopol [315] wherein it was reviewed the accomplishments in the life-cycle performance and it was shown an example of a Pareto solution with three-objective optimization: the maximum condition index, minimum mean safety and present cumulative cost at the end of lifetime; Okasha and Frangopol [266] wherein it was presented a Pareto optimal set for tri-objective optimization considering minimum redundancy index, maximum probability of failure and life cycle costs; García-Segura and Yepes [311], in which it was considered in their study an optimization problem with three objectives: Cost, CO2 emissions and Safety. All these studies pointed out the importance of quantifying all the costs that were measured at economic, social, and environmental level.

5.5 Framework validation considering life-cycle analysis and optimization

Considering all the concepts previously presented, Figure 5-6 depicts a framework by combining the life cycle cost analysis and the optimization problem. The framework is divided into three modules: (i) structural assessment and degradation modelling previously discussed in chapters 3 and 4; (ii) maintenance activities and (iii) optimization problem. The framework is hereafter validated through two different practical applications, a simply supported RC Bridge, previously presented in chapter 3 and a network analysis.

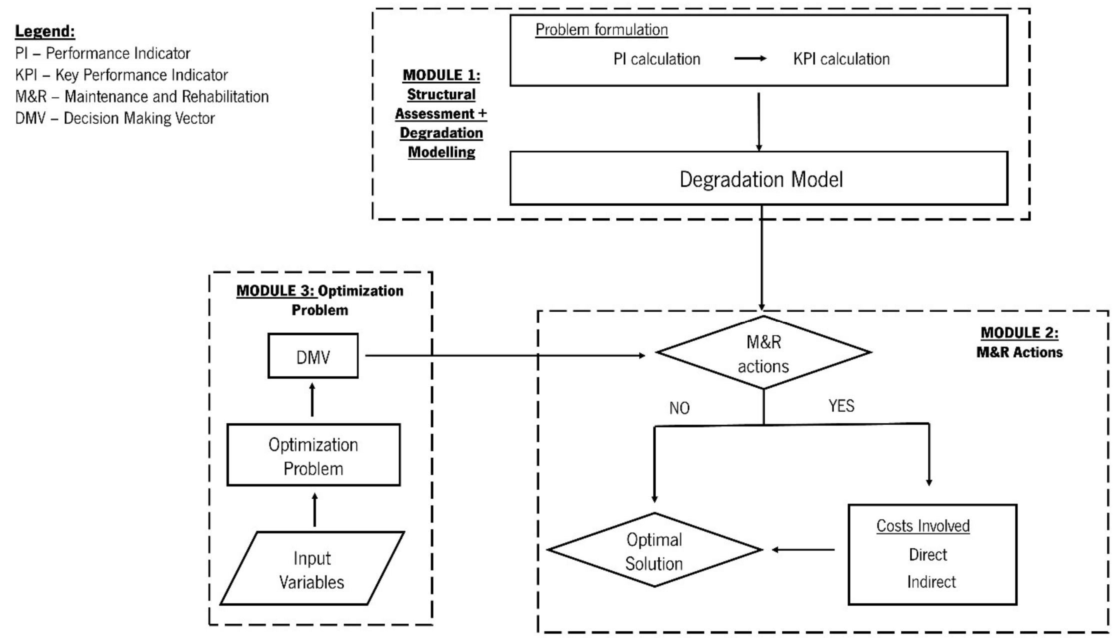


Figure 5-6 – Proposed framework

5.6 Practical case A: simply supported RC bridge

Considering again the practical application of the simply supported RC bridge described in chapter 3, the following section aims to apply the concepts previously discussed regarding life cycle analysis and optimization and validate the framework presented in Figure 5-6. Considering the LCAC, i.e. the direct costs associated to the maintenance and rehabilitation (M&R) activities, Table 5.3 resumes the adopted values for the current application. For the sake of simplicity, the M&R activities were categorically labeled as preventive and essential activities. Furthermore, the considered effects, as well as the costs were also assumed to highlight the practical application. To address uncertainty, a triangular distribution was assumed on the quantification of the effects.

Table 5.3 – Effects of the M&R actions

Type of maintenance	**Effect of the maintenance**	**Quantification**		**Cost** ($€/m^2$)
Preventive maintenance	ID 1: Slow the degradation rate	Time of reduction (t_r)	Rate of reduction (δ)	25 $€/m^2$
		TD* (years) (2 , 3 , 4)	TD* (40% , 60% , 75%)	
	ID 2: Keep the same condition during a period	Time of delay (t_d)		50 $€/m^2$
		TD* (years) (2 , 3 , 4)		
Essential maintenance	ID 3: Improvement on the condition state	TD* (5%,10%,15%) of the KPI considered for the initial age		150 $€/m^2$

** TD stands for triangular distribution with (min value, most likely value, max value)*

As for the LCSA, i.e. the user costs, its calculation were based on equations (5-4) , (5-5) and (5-6). Accordingly, Table 5.4 shows the adopted parameters

Table 5.4 – Parameters considered in the indirect costs

Parameters	**Quantification**
DUR	Expert opinion
l_r (m)	13
Type of train	Medium to long trip trains
VA_v (€/min)*	2.5
S_r (km/h)	30
S_n (km/h)	120
TMD^*	15
PER	TD** (10%-40%-70%) - preventive
	TD** (80%-90%-100%) - corrective

** Values considered according with* [149]

*** TD stands for triangular distribution with (min value, most likely value, max value)*

5.6.1 Optimization problem

Having defined the parameters to estimate the life cycle costs, the next step concerned the optimization problem definition. The project manager seeks to obtain the best maintenance plan by defining an appropriate schedule. Although maintenance plans with a great number of maintenance activities result in a less deteriorated structure, their costs are considerably higher. This leads to a multi-objective optimization with two conflicting objectives that must be simultaneously optimized. The optimization problem was solving using the NSGA–II, through the *gamultiobj* built in function of MATLAB [316].The population size was of 500 and considered alongside with 50 generations. A crossover scattered function, to generate the crossover for children, was adopted and a tolerance criterion of 1e-6. Accordingly, the optimization problem was stated as follows:

Find:

- The optimal maintenance schedule (decision-making vector)

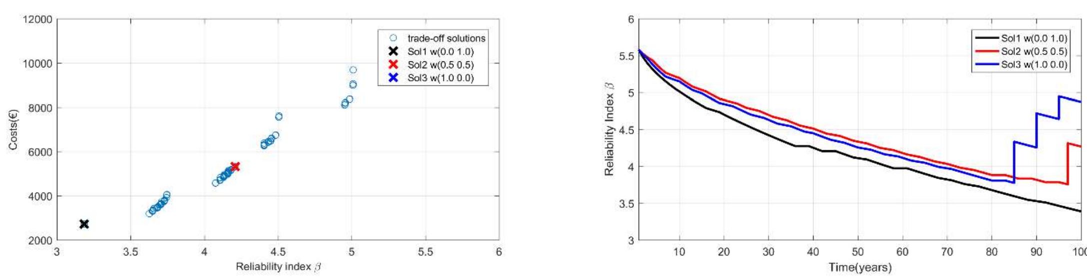
To achieve the following conflicting objectives:

- Maximize the final reliability index
- Minimize the costs

Submitted to the following constraint:

- $t_i - t_j \geq 5\ years$ (the interval between two applications shall not be lower than 5 years)

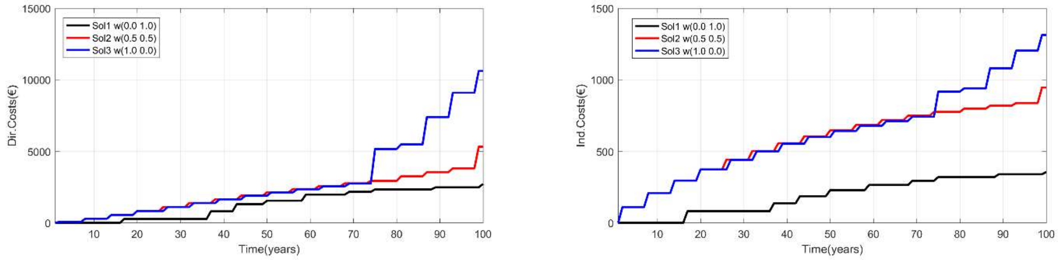
The obtained results can be shown in the Pareto front, which illustrates possible trade-off solutions, see Figure 5-7a), for the lifetime of 100 years. It is worthwhile to mention that each solution of the Pareto front represents an optimal solution with a different possible scenario. For sake of simplicity, three representative solutions were chosen.



a) b)

Figure 5-7 – Optimal solutions: a) Pareto front; b) Reliability-based

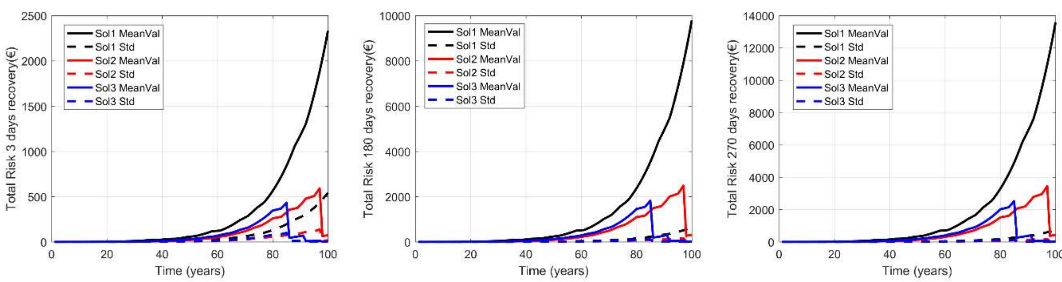
Each solution related its performance as well as the life cycle costs, see Figure 5-7b), Figure 5-8a) and Figure 5-8b). Solution 1 was the least expensive solution, presenting thus the worst performance. Solution 3 represented the most expensive solution, because more maintenance actions were applied, yet the best in terms of performance. Solution 2 represented the solution wherein the same weight was given for costs and performance.



a) b)

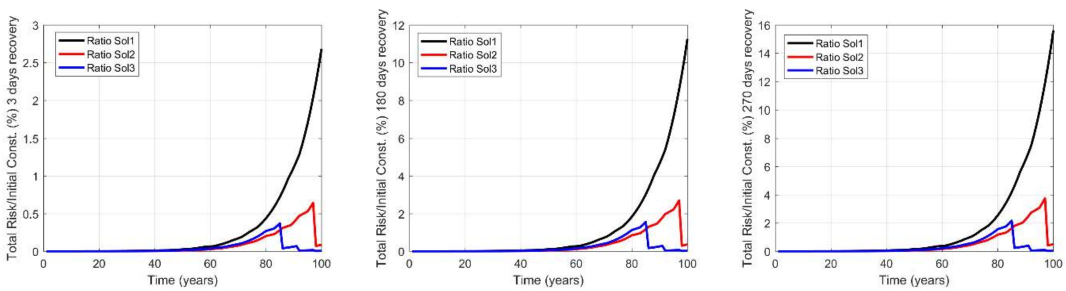
Figure 5-8 – Costs estimation: a) Direct Costs; b) Indirect Costs

Associated to the obtained reliability index, Figure 5-9 and Figure 5-10 presents the corresponding risk as well as the ratio, respectively, for 3, 180 and 270 days of recovery. It was noticed that the implementation of the maintenance activities decreased considerable the initial risk of the structure as well as the ratio that relates the risk of the structure and the initial value of the construction.



a) b) c)

Figure 5-9 – Total Risk: a) 3 days of recovery; b) 180 days of recovery; c) 270 days of recovery

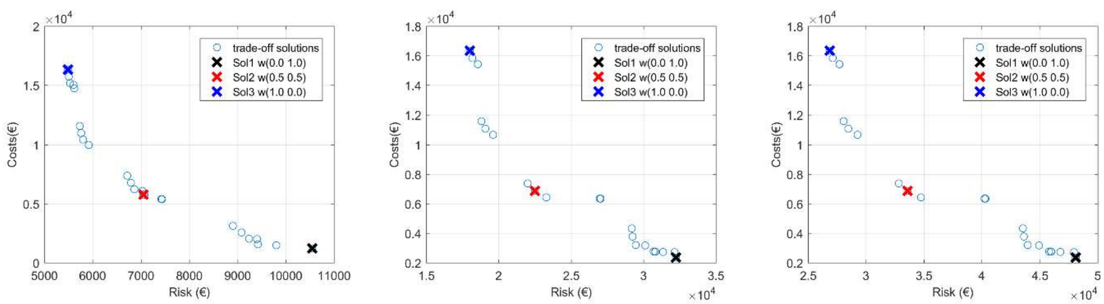


a) b) c)

Figure 5-10 – Ratio estimation: a) 3 days of recovery; b) 180 days of recovery; c) 270 days of recovery

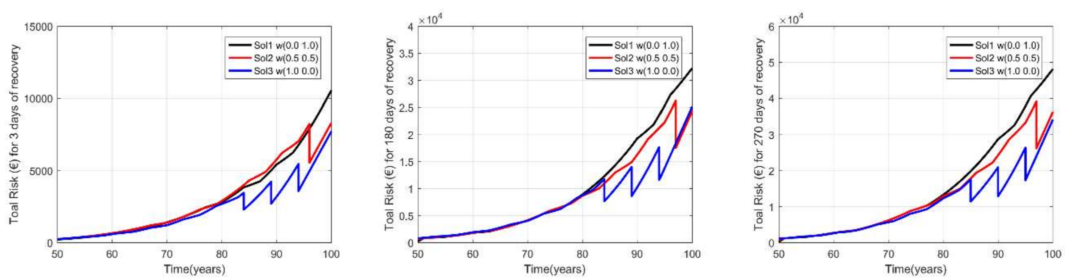
Another important aspect to discuss relies on the risk measure. Note that the previous calculations relied on the reliability index in the objective function. The risk was obtained indirectly from the reliability index.

Considering the risk directly on the objective function, the purpose of this analysis was to investigate how different the costs of maintenance were when compared to the reliability index. Note that from the time-dependent risk curve obtained in the chapter 4, the risk was considered important around the middle age of the bridge. In this way, the problem of optimization was formulated after the age of 50. Furthermore, a constraint of a time interval of application of maintenance actions between 5 years was assumed in this approach. For sake of simplicity, the risk was considered and evaluated by its mean value. Considering all these assumptions, Figure 5-11, Figure 5-12, Figure 5-13, Figure 5-14 and Figure 5-15 depict the obtained results.



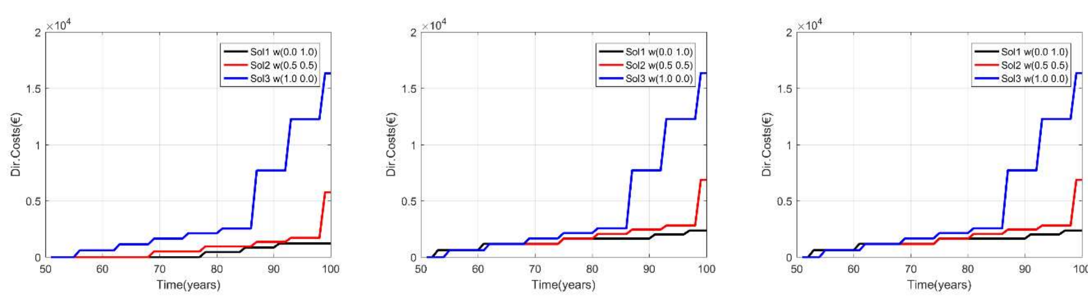
a) b) c)

Figure 5-11 – Pareto Front: a) 3 days of recovery; b) 180 days of recovery; 270 days of recovery



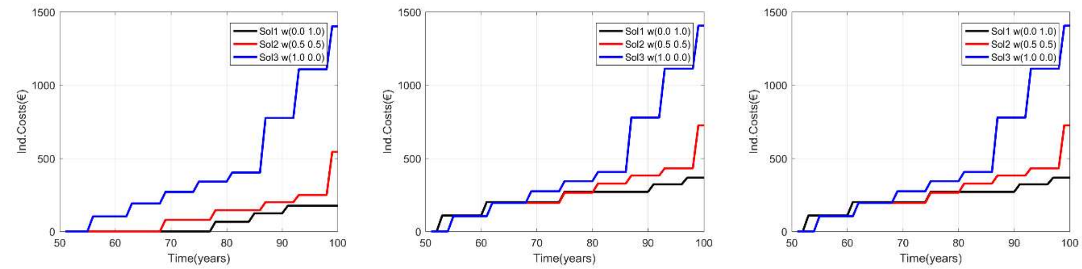
a) b) c)

Figure 5-12 – Total optimal risk estimation: a) 3 days; b) 180 days; c) 270 days



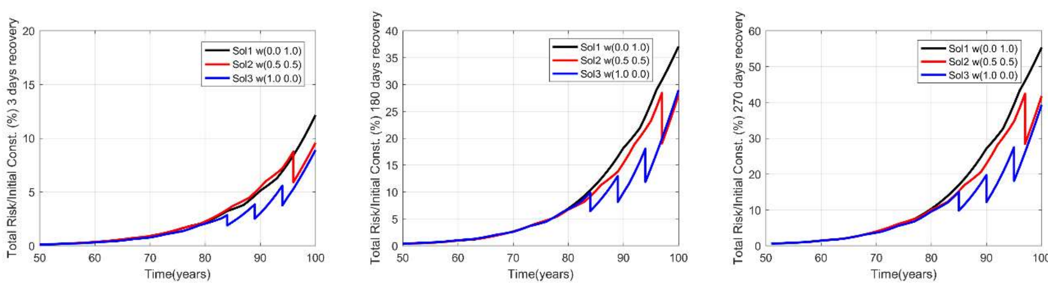
a) b) c)

Figure 5-13 – Direct costs: a) 3 days of recovery; b) 180 days of recovery; c) 270 days of recovery



a) b) c)

Figure 5-14 - Indirect costs: a) 3 days of recovery; b) 180 days of recovery; c) 270 days of recovery



a) b) c)

Figure 5-15 – Total Risk ratio: a) 3 days of recovery; 180 days of recovery; 270 days of recovery

Observing the optimization results by considering the risk measure, it was concluded that the involved costs, in comparison with the previous optimization, were higher. In fact, this result was expected as the optimization was carried out only after the age of 50. Since the risk exponentially grows every year, to reduce those values, more direct costs are involved. Hence, the consideration of the reliability curve instead in the objective is though more conservative, less costly, and therefore more recommended for this type of analysis.

5.7 Practical case B: Network Analysis

The following practical application considers an example of a theoretical network aiming to emphasize an optimization process in a higher level rather than an isolated bridge. For the following example, a simple network was considered, see Figure 5-16.

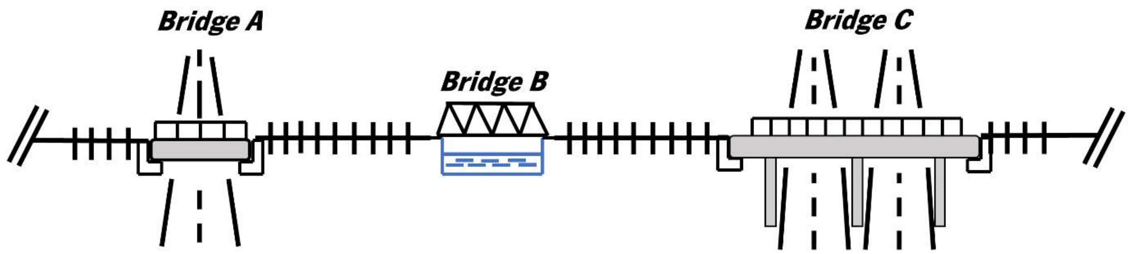


Figure 5-16 – Network scheme

The railway bridges were assumed to present the same materials as the examples covered in the previous chapters 3 and 4, i.e. concrete and steel. Because two bridges might be insufficient to conduct this practical application, a third bridge was added. It was assumed that the third bridge was made of concrete material and composed of large span deck. For the network level, the level of complexity regarding the structural assessment decreases as the calculations do not focus on a simple element. In this way, the inspection based key performance indicators (KPIs) present a better option. Bearing this in mind, the following analysis will consider the condition index based KPI. The following bridges were considered to present the same degradation pattern for the intensity matrixes calculated in the chapter 4. For the sake of simplicity, the time horizon assumed here will be reduced and considered as 30 years. Table 5.5 resumes all the assumptions for the present case application.

Table 5.5 – Bridge features considered for the application

	Span length(m)	**Width (m)**	**Material**	**Initial condition state**
Bridge A	10	5.0	Concrete	2
Bridge B	30		Steel	2
Bridge C	50		Concrete	3

The considered effects as well as the costs were the same as presented in Table 5.3 except for the improvement effect, in which a triangular distribution, *TD(0.5,1,1.5)*, was considered. For the indirect costs, the parameters are presented in the Table 5.6.

Table 5.6 – Paramters of the indirect costs

Parameters	Quantification
DUR	Expert opinion
l_r (m)	Length accordingly with Table 5.5
Type of train	Medium to long trip trains
VA_v (€/min)*	2.5
S_r (km/h)	30
S_n (km/h)	160
TMD	20
PER	TD** (10%-40%-70%) - preventive
	TD** (80%-90%-100%) - corrective

**TD stands for triangular distribution

5.7.1 Optimization problem

The optimization problem was considered by looking each bridge individually at the first stage. In this stage the objective was to compute the optimal schedule of each bridge considering the cost and the performance of the bridge, same principle as the previous chapter. The difference relied on the MOP initial formulation wherein the objective was to minimize the worst condition state obtained. In this way, the MOP was stated as follows:

Find:

- The optimal maintenance schedule (decision-making vector)

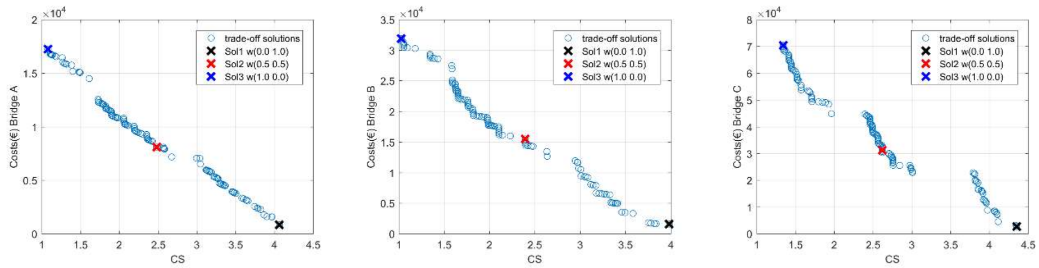
To achieve the following conflicting objectives:

- Minimize the worst condition index obtained
- Minimize the costs

Submitted to the following constraint:

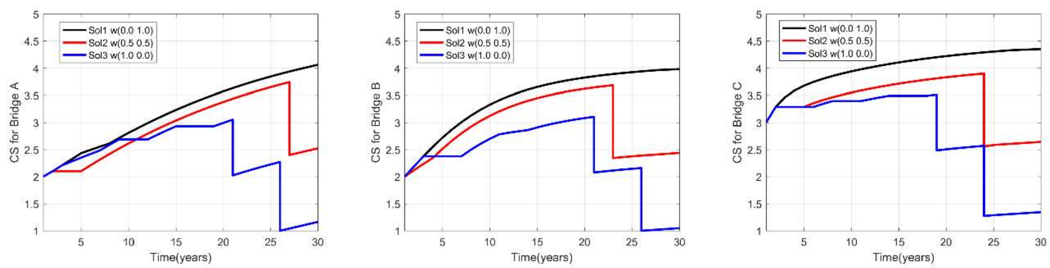
- $t_i - t_j \geq 5\ years$ (the interval between two applications shall not be lower than 5 years)

The results for each bridge are shown in Figure 5-17, Figure 5-18, Figure 5-19 and Figure 5-20. Again, for the sake of simplicity, three different solutions were selected to show the performance of the curves as well as the involved costs. It was observed from those figures that the maintenance activities efficiently kept the condition state on low values, mainly for the schedules that are related with the application of the essential maintenance, i.e. ID3. Note that, although the effect of the ID3 was more expensive, the optimization opted for choosing this solution since there was a considerable improvement on the performance.



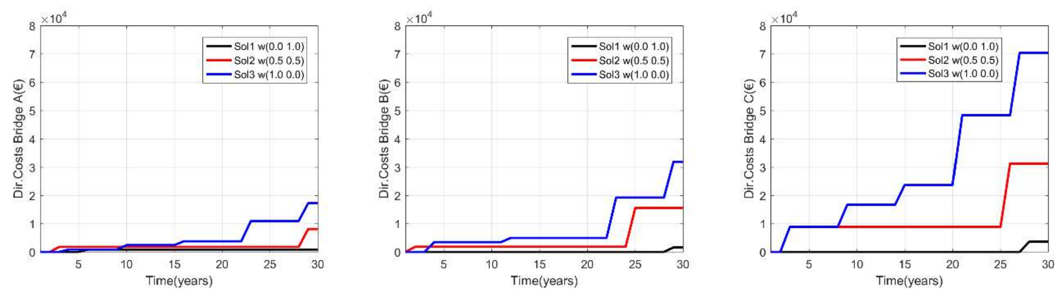
a) b) c)

Figure 5-17 – Pareto front: a)Bridge A; b) Bridge B; c)Bridge C



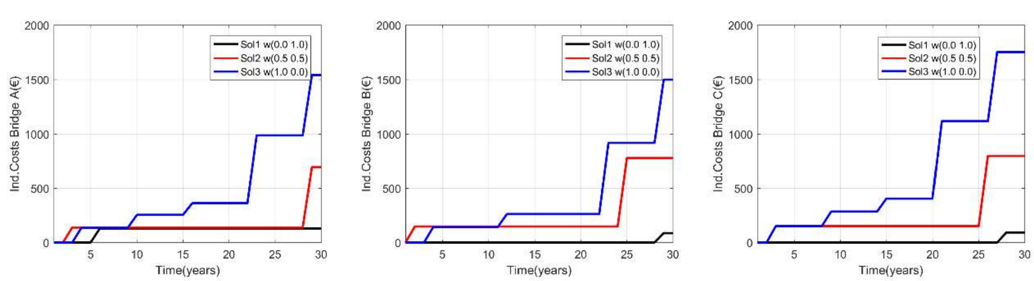
a) b) c)

Figure 5-18 – Condition state index: a) Bridge A; b) Bridge B; c) Bridge C



a) b) c)

Figure 5-19 – Direct Costs: a) Bridge A; b) Bridge B; c) Bridge C



a) b) c)

Figure 5-20 – Indirect Costs : a) Bridge A; b) Bridge B; c) Bridge C

5.7.2 Combination of the optimal solutions

The next step on the definition of an optimal network combined the previous results of a single bridge on global optimal maintenance of the network. Table 5.7 shows a resume of the maintenance actions applied on each bridge as well as the accumulated costs. Table 5.8 shows the corresponding combining results for the network.

Table 5.7 – Proposal of optimal M&R actions and costs for each bridge

		S1			S2			S3		
		ID1	ID2	ID3	ID1	ID2	ID3	ID1	ID2	ID3
Bridge A	Year(s) of application	6	-	-	-	3	29	4	10,16	23,29
	Final Cum Costs		962.0			8795.0			18781.0	
Bridge B	Year(s) of application	29	-	-	2	-	25	12	4	23,29
	Final Cum Costs		1670.0			16294.0			33381.0	
Bridge C	Year(s) of application	28	-	-	-	3	26	-	3,9,15	21,27
	Final Costs		3682.0			32040.0			72106.0	

Table 5.8 – Proposal of the optimal M&R actions and costs for the network

		S1			S2			S3		
		ID1	ID2	ID3	ID1	ID2	ID3	ID1	ID2	ID3
Network	years(s) of application	6(B1) 28(B2) 29(B3)	-	-	2(B1)	3(B1), 3(B2)	29(B1), 26(B2), 25(B3)	4(B1), 12(B3)	10,16(B1) 3,9,15(B2) 4(B3)	23,29(B1) 21,27(B2) 23,29(B3)
	Final Costs		6314.0			57129.0			124268.0	

According with the results obtained by Table 5.8 for the network analysis, it was concluded a considerable increase on the obtained costs for the three optimal solutions. Figure 5-21 shows a pie-chart with the weight of each bridge in the overall costs. It was observed that the bridge C represented more than 50% of the costs. This fact was associated to the length of the span. If the span increases, the more dependent the total cost is of that bridge. Besides the length of the span, the fact of the bridge initial condition state being higher than the remaining, led to more interventions.

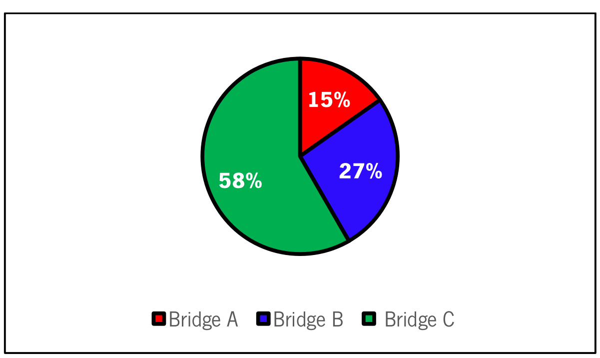


Figure 5-21 – Weight of the bridges on the overall costs

This small application of extending the local assessment of a simple bridge to a network reflected on the considerable increase on the overall costs. Besides that, it is noteworthy to mention that it could be added to this analysis more infrastructures such as the railway track or tunnels to a more complete analysis of the network rail. However, as there were not enough data regarding inspections of these infrastructures, they were not included in this application.

5.8 Final remarks

This chapter aimed to point out and discuss the issues encountered on the life-cycle cost analysis. The literature stated several approaches to the quantification of those costs by dividing them into three categories, agency costs, i.e. the direct costs, social costs, i.e. indirect costs from the users and environmental costs. Another important aspect discussed in the literature related to life cycle analysis relied on the optimization problem, due to the limited budget faced by the stakeholders. Given that performance and costs are conflicting objectives, a MOP presented a reasonable option for the analysis. Moreover, a review on the formulation of the GAs were made, especially on the NSGA-II. Regarding the practical applications, the first case relied on the application of the life cycle analysis and optimization of one bridge. The KPIs adopted for this bridge relied on the reliability and the risk. The MOP shown a set of optimal solutions, i.e. the Pareto front, in which three different optimal solutions were chosen based on their weight on the objectives. The most expensive solution proved to be very effective by keeping a high performance, though with very high costs when compared for example with the least expensive solution. The MOP was formulated considering either the main objective as the reliability, and thus calculating the risk, or the risk as the main objective. The obtained results were different since the metrics, although related to each other, present different scales. For example, looking at the MOP by the risk point of view would lead to the conclusion that the bridge would need maintenance after the year 50, while looking for the reliability would need maintenance on the initial years thus leading to a more conservative solution in terms of performance. The second practical application focused on an application of the problem in a higher level, i.e. a network. In this way, three bridges with different span length and different degradation rate were hypothetically considered on the problem. The network was considered non-redundant given the pattern of the railway infrastructures in Portugal. The same MOP formulation was applied in here and thus combined at a global level. The results shown that the bridge with a large span presented more than half of the total costs of the network. Besides that, the initial condition state of the bridge was higher than the remaining. Because of these two issues, a warning should be raised on the stakeholders while looking at the network, regarding bridge infrastructures.

Another important concerning issue relates to the continuous natural hazards that those bridge infrastructures are subjected during their entire life cycle. Thereby, an establishment of mitigation actions are important to reduce the effects of those hazards. Moreover, when the mitigation activities cannot respond to the incoming threats, recovery plans should be established to minimize the post damage. These above-mentioned issues are further discussed in the next chapter.

Chapter 6

6 Hazard Analysis

The present chapter reviews and discuss potential sudden events that infrastructures might be subjected in their entire life cycle with special focus on bridges. The resilience of infrastructures, a indicator to evaluate the recovery of a system, is hereafter discussed as well as the analytical formulations proposed on the literature concerning the recovery functions. To finalize the chapter, the two practical applications considered in the previous chapter, are presented to discuss the influence of the hazard events on the degradation curve as well as the post recovery by estimating the resilience.

6.1 Failure causes of infrastructures – Bridge Case

A bridge is designed to provide an adequate load-carrying capacity, which is based on the assumed loads and the strength of materials to be employed, and comfort to users during its life-cycle. However, some errors can result on the collapse of a bridge. Failure of bridges has been shocking over the years both in engineering community and public in general. Such event can be defined as a loss of a certain structural element of the bridge or due to a catastrophic collapse. As such, lessons need to be learned from each failure. In the literature, there are already several publications relating case studies of bridges that failed and their origin causes such as in [317, 318].

For a more detailed analysis about bridge failures, several authors have been proposing a creation of a bridge failure database. This database aims on gathering all the documentation of failures or damages of bridges to carry out useful post event studies. For example, Imhof [115] studied several cases of bridge failures and developed database which listed around 350 events of bridges collapses. A report of United States Bridge Failure [319] was conducted from 1980 to 2012 at the University of Buffalo and developed by the Multidisciplinary Center for Earthquake Engineering Research. The above mentioned references were reviewed in order to answer to three main questions [115]:

1. Which types of failures can occur?

2. When they occur?

3. What are the main causes?

Regarding the first question, there are two types of failure, the ultimate limit state (ULS) and the serviceability limit state (SLS). Imhof [115] presented in his PhD thesis 384 structural failures analysed by Matousek and Schneider either for ULS or SLS. Regarding the first group, i.e. ULS, the failure type accounted with loss of equilibrium, complete collapse, partial collapse, and failures of other nature. As for the SLS, the failure types accounted with excessive cracks, settlement or deformation and wrong dimensions. Concerning their occurrence, the failures can occur during their construction or service life. According with Imhof [115], the majority causes of failure are related with the construction phase. Knowing that in this phase the bridge is more vulnerable, since it is supported by temporary elements such as falseworks or scaffoldings, there are several processes that need to be checked to avoid those fails. Detailed information about this type of failures can be consulted in [320]. Nevertheless, for this thesis, bridge failure will be thoroughly discussed during their service life.

Concerning the third question, the causes of bridge failures can be either internal, like design errors, lack of maintenance, deficiency in construction and material defects or external like earthquakes, scour, floods, collisions, environmental degradation, overload, fire, and winds. Despite all this causes, it is known so far that the main causes of collapse are by hydraulic nature (floods and scour). Figure 6-1, depicts the main causes of collapse of 1062 bridge in the USA since 1980 until 2012.

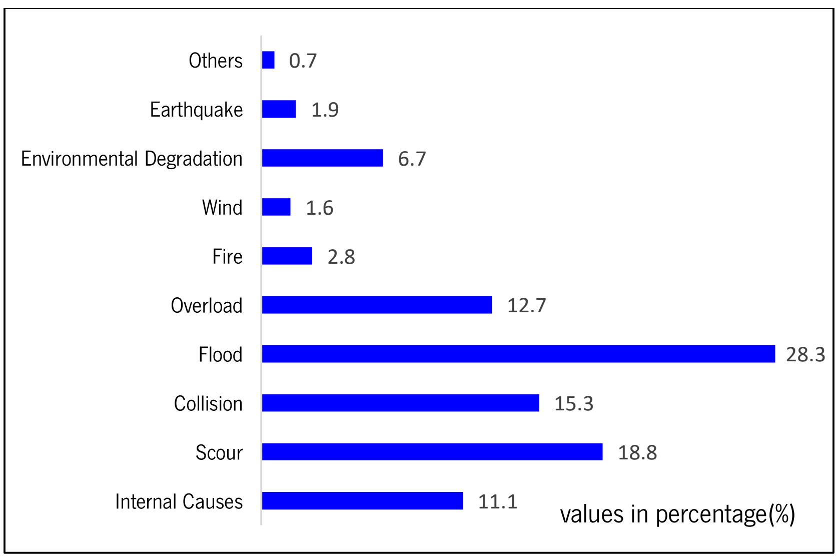


Figure 6-1 - Causes of failure according with [319]

Analyzing Figure 6-1, besides the hydraulic hazard events, it is noted that the collisions represent a considerable importance on the failure of bridges. Those collisions are associated with ships and vessels. Despite their low velocity, their huge quantity of mass can cause large amounts of damages in the bridge that cannot be repaired. These case studies can be consulted in [317, 318, 321].

Besides the USA, in Europe the topic of bridge failure comprises a serious issue. For example, Network Rail, the UK company responsible for the railway infrastructures, states that the railroad bridges are very sensitive to strikes that damages both the road traffic and the railway track. Estimations on 2019 shown that there was an occurrence of 14 strikes in one day and a total of 1787 strikes on the overall year resulting on estimating cost on the UK economy of around 23£ million [322].

In Portugal, the study case of Hintze Ribeiro bridge collapse by scouring, which killed around 59 people, brought off the authorities for a regular inspections of the bridges, in order to avoid/anticipate these kind of disasters, by periodic underwater bridge inspections [323].

6.2 Resilience

Resilience is a well-established concept, with its first definition being introduced by Holling, in the field of ecology, [324] where it is defined as *"the persistence of relationships within a system and is a measure of the ability of these systems to absorb changes of state variables, driving variables, and parameters, and still persist"* . Nevertheless, this concept has evolved to comply with current demands. Resilience has been developed and explored in several fields of knowledge such as psychology, material science, economics, environmental studies, and infrastructures. After the 2000s, there has been a surge of scientific research related to resilient infrastructures in which some of them are worth to mention. Bruneau et al. [325] provided a general framework to define and quantify the seismic resilience of communities. Their framework also included an indication on the four properties of resilience: robustness, redundancy, resourcefulness, and rapidity (four Rs). In Dalziell et al. [326], the concept of resilience has been introduced as the ability of organisations to continue working when facing unexpected events. Moreover, the relationship between resilience and risk management was also discussed. Chang et al. [327] proposed a series of quantitative measures of resilience and demonstrated them in a case study of an actual community. Their case study focused on seismic mitigation decision making for the Memphis, Tennessee, water system. Another important goal of their study was to explore the extent to which earthquake loss estimation models can be used to measure resilience. McDaniels et al. [328] developed a conceptual framework for understanding the factors that influence the resilience of infrastructure systems in terms of two dimensions: robustness and rapidity. Cimellaro et al.[329] attempted to formulate a framework to quantify resilience that may combine the information coming from different fields into a unique analytical function. Losses were described as functions of fragility of systems and distinguished between structural and non-structural losses. Moreover, three types of recovery functions were proposed: linear, exponential, and trigonometric. The proposed methodology was applied to an hospital in San Fernando Valley for four different hazards levels. Later, the same authors [330] improved their conceptual framework. Specifically, a mathematical description of the two properties, proposed by Bruneau et al.

[325] (rapidity and robustness), was presented. The framework was applied to two case studies: (i) economic loss estimation study of a specific hospital, (ii) regional loss estimation study aimed to evaluate the economic losses of a hospital network within a geographical region, such as a city. Ouyang et al. [331] introduced a three-stage resilience analysis framework that considered both joint hazards and system resilience capacities. After introducing the typical response curve of an infrastructure system subjected to a disruptive event, the curve considered three stages: reflecting system resistant, absorptive, and restorative capabilities. For each stage, a series of resilience-based improvement strategies were highlighted and some sample strategies for enhanced infrastructure resilience in practise was provided. Taking the power transmission grid in Harris County, Texas, USA, as a case study, the study compared an original power grid model with several hypothetical resilience-improved models to quantify their effectiveness at different stages of their response evolution to random hazards and hurricane hazards. Barker et al. [332] defined two approaches to measure the importance of network components from the perspective of component contribution to network resilience as a function of stochastic vulnerability and recoverability terms. Resilience was described as a function of four interacting paradigms: reliability, vulnerability, survivability, and recoverability. Francis et al. [333] proposed a resilience analysis framework and a metric for measuring resilience. The framework and metric were applied to the example of an electric power infrastructure of a fictional city. Their analysis framework consisted of system identification, resilience objective setting, vulnerability analysis, and stakeholder engagement. The implementation of this framework was focused on the achievement of three resilience capacities: adaptive capacity, absorptive capacity, and recoverability. Adjetey-Bahun et al. [334] proposed a simulation-based model for quantifying resilience in mass railway transportation systems by quantifying passenger delay and passenger load as the system's performance indicators. Their approach integrated all subsystems that make up mass railway transportation systems (transportation, power, telecommunication and organisation subsystems) and their interdependencies. Gonzàlez et al. [335] studied an interdependent network recovery model for partially destroyed system identifying the optimal restoration strategy of the damage network by minimizing the costs. This analysis was carried out considering different interdependencies of the network such as geographical or functional. Lin et al. [336, 337] proposed a simulation-based building portfolio recovery model to predict the functionality recovery time and recovery trajectory of a community resilience. This building portfolio referred merely to an inventory of buildings for which it was defined a functionality state after natural scenario hazard events. Each functionality states were defined by a number that ranged from 1 to 5. The simulation was carried out into two steps. The first step was modelling individual building-level restoration as a Markov-Chain process aggregating the

physical process of building level. The second step was modelling building portfolio-level recovery through aggregating the restoration processes of individual buildings across the domain of the community and over the entire recovery time horizon. Barabadi et al. [338], introduced a model for predicting the recovery rate of infrastructures, by considering the effect of influencing factors. The proposed methodology was based on the availability of historical data.

6.2.1 Resilience applications on bridges

Concerning bridges, the emphasis of many researchers in this field has been carried out especially on structural performance since bridges are a fundamental component of the transportation system. As a matter of fact, the interruption and the reduced capacity of this vital infrastructure system could have large implications in terms of health safety economics and social aspects of affected communities. The quantification procedures and the methodologies for the valuation of resilience of bridges presented in literature are varied and involve many fields of engineering such as seismic, transportation and hydraulics. Table 6.1 shows the several works applied on this field by chronologic order as well as the encountered limitations.

Table 6.1 – Contributions in field of resilience applied to bridges

Author, date	Main Contributions	Limitations
Padgett and DesRoches, 2007, [339]	• Assessed the relationship between bridge damage and the resulting loss of functionality of the bridge based on expert opinion;	• Although loss of functionality is thoroughly assessed, the authors do not respond to the resulting consequences before such losses
Bocchini amd Frangopol,2012, [340]	• Analysed several bridges within a transportation network which was affected by a significant seismic event aiming at finding the optimal restoration planning and resource allocation based on network resilience, functionality and cost.	• Although cumulative costs resulting from interventions were calculated for each optimal scenario, quantification of direct and indirect losses to the network require further detailing.
Lokuge and Setunge, 2013, [341]	• Proposed a study to evaluate bridges damaged due to floods and its resilience.	• The overall framework is based solely on qualitative approaches.
Deco et al.,2013, [342]	• An evaluation tool was proposed to be used for decisions concerning proactive maintenance, retrofit or life-cycle management.	• The previous state condition of the bridge in terms of degradation along its life-cycle is not considered.
Chandrashekaran and Banerjee, 2014, [343]	• Multi-objective algorithm analysis applied to a RC bridge affected by the multi-hazard effects of earthquake and flood induced scour;	• Losses associated to the events are not covered
Venkittaraman and Banerjee, 2014, [344]	• Demonstration that bridge retrofit strategies enhances bridge both seismic performance as well as system resilience.	• Indirect consequences are briefly described.

Dong and Frangopol, 2015, [345]	• Framework developed to investigate the effects of aftershocks on seismic consequences and functionality associated with damaged bridges.	• Low information on the pre-event description related to loss of performance in case of an earthquake occurrence.
Karamlou and Bocchini, 2015, [346]	• Simulation-based methodology to improve resilience quantification and expected life-cycle loss assessment of highway bridges in seismic engineering.	• Indirect consequences are not thoroughly analysed.
Alipour and Shafei, 2016, [22]	• Seismic resilience of a transportation network considering the component degradation.	• Assessment at the bridge level is more uncertain and inaccurate.
Dong and Frangopol, 2016, [124]	• Multi-hazard assessment life-cycle assessment considering earthquakes and floods. • The main novelty introduced in the study is the time-dependent model of earthquake occurrence.	• The estimation of the total loss could be divided into direct and indirect losses on its final presentation
Minaie and Moon, 2017, [240]	• Practical and simplified multistage framework to quantify bridge resilience, based on a series of restoration times for a simplified single-path recovery period.	• Estimation of consequences in terms of associated costs was not made.
Vishwanath and Banerjee, 2019, [23]	• Seismic resilience assessment over the life cycle of a reinforced concrete bridge • Estimation of the direct and indirect losses of the system	• Although there is an estimation of the losses, a quantification in terms of money could be provided to understand the order of magnitude

6.2.2 Resilience formulation

Bruneau et al. [325] proposed a framework to define seismic resilience, defined as *"the ability of social units to mitigate hazards, contain the effects of disasters when they occur, and carry out recovery activities in ways that minimize social disruption and mitigate the effects of future earthquakes".* In analytical terms, functionality can be expressed as:

$$\boldsymbol{RS = \int_{t_0}^{t_0+t_R} Q(t)dt} \tag{6-1}$$

where Q is the functionality, t_0 is the occurrence time event and t_R is the time to complete recovery of the component under analysis.

Several models have been proposed to describe the recovery function which can be either empirical or analytical, depending on the source of data and the type of analysis [347]. At any given time, the performances of a system can be represented by a function that describes its time-dependent functionality. Functionality can be represented as a non-stationary stochastic process that ranges from 0% to 100%, where 100% means no reduction in performances, while 0% means total loss.

The occurrence of both natural and manmade events can cause an abrupt loss in its performance, followed therefore by a gradual restoration to normal performance levels, depending on the resources employed. This conceptual definition is illustrated in the Figure 6-2.

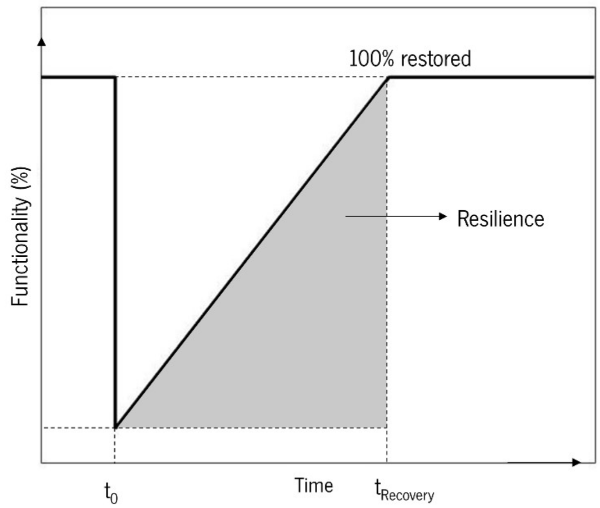


Figure 6-2 – Resilience definition

The recovery curve of a system is typically a function that describes the process of restauration to the initial properties of the system. Recovery means to reach newly a state of acceptable performances after a sudden event. However, system recovery is complex process once it is affected by several factors and parameters, many of these difficult to estimate due to several uncertainties. The essential requirement of the analytical recovery models is simplicity. Therefore, the model should be selected so that it is easy to fit to real or numerical observation data. The recovery models can be divided into long-term recovery models, i.e. recovery models adopted for reconstruction phase, and short-term recovery models, i.e. for emergency phase after the sudden event. For this thesis, only long-term models are covered. As stated by [330], long-term models can be divided according their number of parameters. Simple models account with less parameters while compared to the complex. Table 6.2 shows the different types of long-term recovery models proposed by different authors as well some observations to the models.

Table 6.2 – Long term recovery models formulation

Type	Formulation	Observations
Uniform cumulative distribution recovery function	$Q(t) = Q_0 + F(t/t_0\,, t_0 + T_R)[Q_R - (Q_0 - L_0)]$ $F(t/t_0\,, t_0 + T_{RE}) = \frac{(t - t_0)}{T_R} I(t_0, t_0 + T_R)$	• Model adopted when there is no information regarding the network domains, social response, available resources or preparedness [330]; • It is considered the simplest model being dependent only on the recovery time T_R; • Q_0 is the initial functionality after the drop; L_0 is the initial total loss of functionality after the drop; t_0 is the time of occurrence of the disruption event. $F(t/t_0\,, t_0 + T_{RE})$ is the uniform cumulative distribution.
Lognormal cumulative distribution recovery function	$Q(t) = Q_0 + F(t/t_0\,, t_0 + T_R)[Q_R - (Q_0 - L_0)]$ $F(t/\theta\,, \beta) = \frac{1}{\beta\sqrt{2\pi}} \int_{-\infty}^{t} \frac{e^{\frac{[\log(x) - \theta]^2}{2\beta^2}}}{x} dx$	• Exponential [348] and trigonometric [327] models are combined; • Three parameters describe the recovery function, the parameter L_0, which can be used to define the initial loss of functionality after the drop, the parameter θ , that can be used to define the time frame when the societal response and recovery are driven by lack or limited organization/resources and the parameter β which defines the rapidity of the recovery process [347].
Harmonically over-damped recovery function	$Q(t) = Q_0 + \left\{1 - e^{-\alpha t}\left[\left(\frac{\alpha + \beta}{2\beta}\right) e^{\beta t} + \left(\frac{\beta - \alpha}{2\beta}\right) e^{-\beta t}\right]\right\}[Q_R - (Q_0 - L_0)]$ $Q(t) = 1 - L_0 e^{-\omega t}(1 + \omega t)$, for critical damped systems ($\xi = 1$) $f(t\vert\theta, \beta) = \frac{1}{\beta x\sqrt{2\pi}} e^{\frac{[\log(x) - \theta]^2}{2\beta^2}}$	• This model is based on [330]. • The harmonically over-damped recovery function may be employed to consider other structural factors, such as damping, which may be used for the seismic hazard; • L_0 defines the initial total loss of functionality after the drop; $\alpha = \omega\xi$ and $\beta = \sqrt{\xi^2 - 1}$. • ω and ξ are related to the rapidity dimension. In particular, $\xi \geq 1$ and $\omega \leq 1$. Furthermore, rapidity of recovery increases when either ω increases or ξ reduces.

6.3 Framework validation considering the hazard event

Considering the hazard event analysis, Figure 6-3 depicts the framework, previously presented in chapter 5, extended with the inclusion of the hazard event scenario. Likewise the previous chapter, the framework is hereafter presented and validated into the same practical applications as the chapter 5.

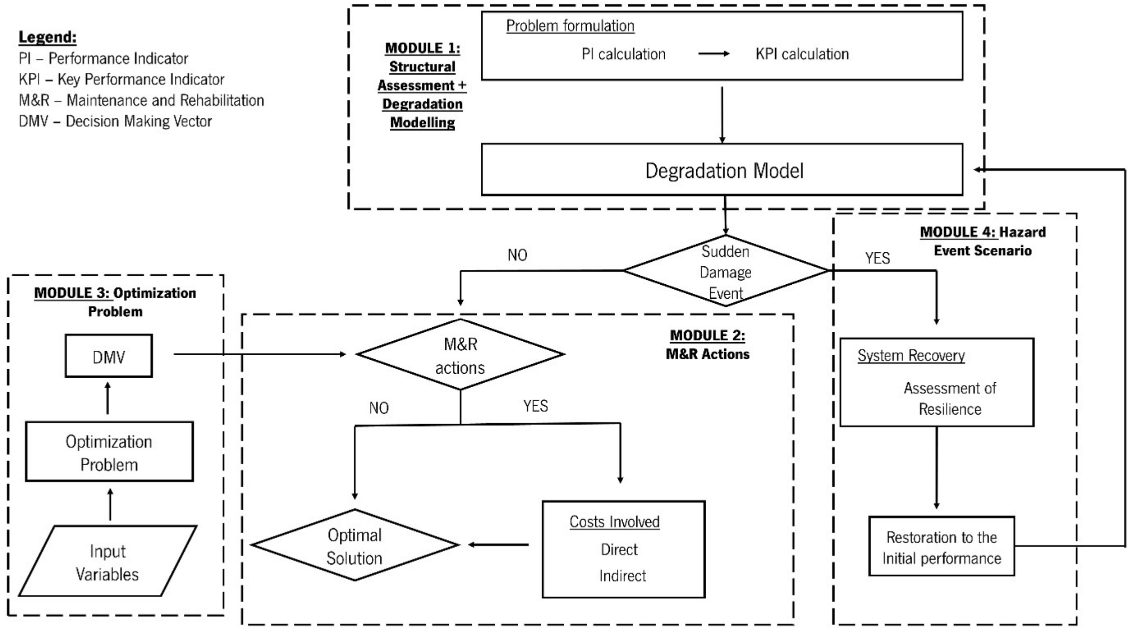
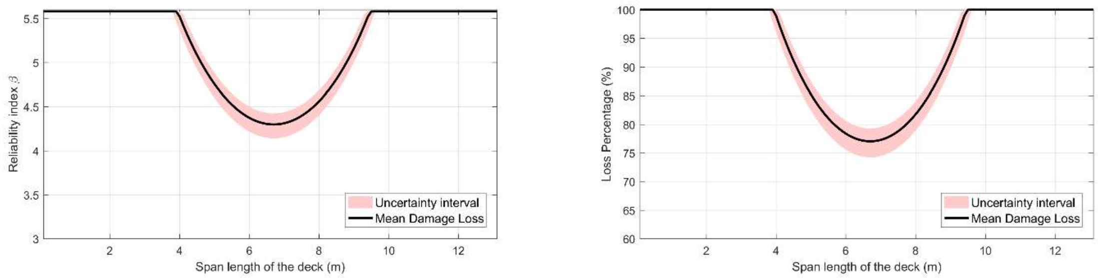


Figure 6-3 – Proposed framework with the inclusion of the hazard scenario event

6.4 Practical case A: Simply supported RC bridge

This practical case relied again on the simply supported RC bridge described in chapter 3. The aim of this application was to detect potential hazard the bridge might be subjected and apply the concepts previously discussed. Here, the hazard was considered as a sudden event that compromises the bridge functionality in a short period of time resulting in its immediate intervention. Observing the bridge as well as the surrounding infrastructures, the most incoming hazard was related to the bridge strikes due to the vehicles that cross under the bridge. Thereby, the simulation of this hazard occurrence was given by loss of resistance in the deck. According with the expert judgment of the Portuguese company "Infraestruturas de Portugal", this event can result in a loss of around 20% on the reinforcing bars. Given that this loss is uncertain, an uncertainty value of 5% was assumed. Hence, the loss was considered to follow a triangular distribution, *TD (15,20,25)*. Furthermore, the company stated that these events can occur up to 5 times per year in the whole network. However, for the following application, the event was idealized to happen one time since the occurrence of more than one event without intervention is extremely unlikely to occur. It is known that the shock of the vehicles can occur in different locations of the deck, hence leading to different performance losses. Figure 6-4 shows the loss estimation accordingly with the location on the span. The results are given by its mean value with an interval of confidence of 95%.



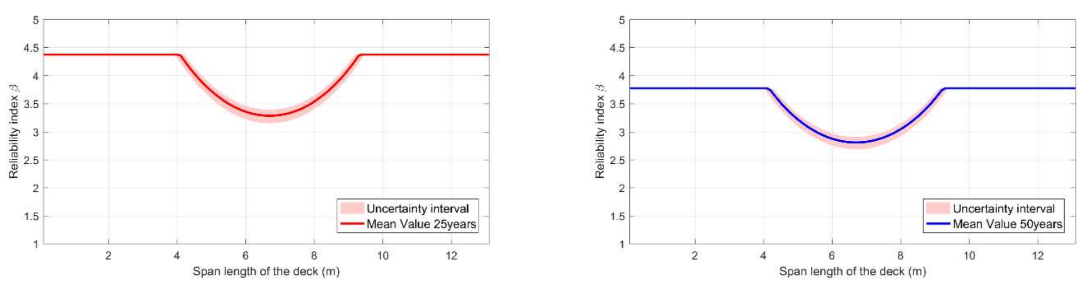
a) b)

Figure 6-4 – Loss estimation along the span: a) Reliability loss initial age; b) Percentage loss

Observing the Figure 6-4, the possible shock of a given vehicle led to a loss on the performance of the bridge within the span range from around 4m to 9m. As expected, the highest losses were verified in the mid-span of the deck with a mean value for the reliability of 4.30 and an upper and lower bound of 4.42 and 4.16, respectively. Those values corresponded to a loss of around 23% of its initial value for the mean and 21% and 25% for the upper and lower bound values, respectively. Note that the values were considered for the bridge in its initial age, i.e. with no effect of the corrosion.

6.4.1 The effect of the hazard event at long term

The consideration of the corrosion in the life-cycle of the bridge along with the hazard event occurrence can result on a considerable loss and therefore an immediate intervention. Figure 6-5 shows the loss on the reliability along the span over the time. For sake of simplicity on the visualization of the result, four different period of analysis were chosen, 25,50,75 and 100 years.



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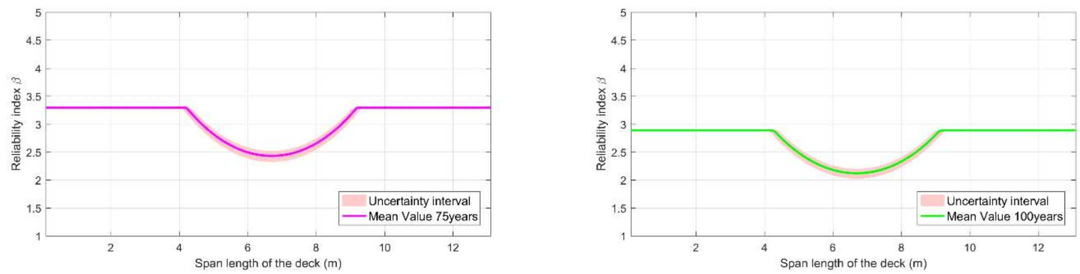
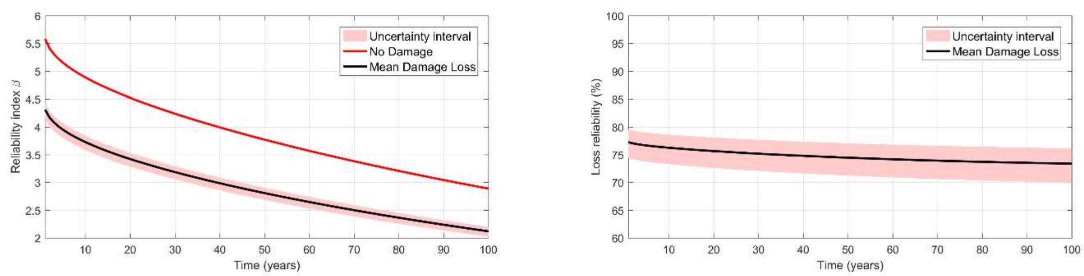


c) d)

Figure 6-5 – Time dependent damage loss along the span: a) 25 years; b) 50 years; c) 75 years: d) 100 years

Considering the worst location of the damage, i.e. the mid-span, the envelope of the degradation curve over the time is depicted in Figure 6-6. It was also observed that the losses on the reliability with the damage were practically constant ranging from 21% to 25%.



a) b)

Figure 6-6 – Envelope of degradation given an hazard event over the time at the mid-span: a) reliability loss; b) percentage loss

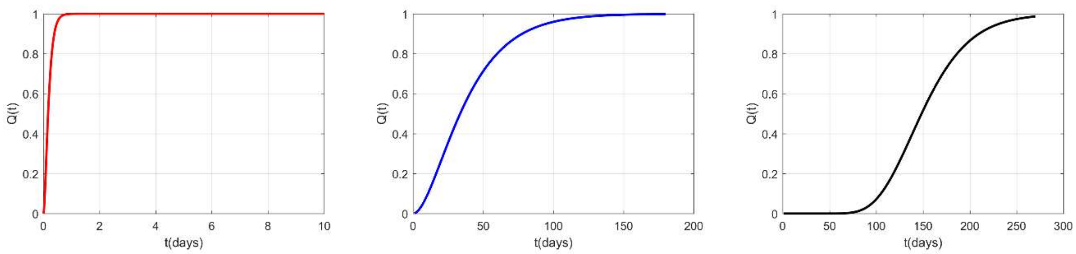
6.4.2 Resilience estimation

The resilience was obtained accordingly with the formulations described in section 6.2.2. The recovery functions were applied to simulate an event recovery considering the degree of preparedness of the society. Note that the assumptions on this formulation served the purpose of illustrating the methodology to apply. Hence, such methodology is broadly applied of other values are assumed. For this application, the same recovery days adopted for the risk calculation were herein assumed [240]. Hence, a very well-prepared society was modelled considering 3 days of recovery while very bad prepared societies were modelled considering 180 days and 270 days. Note that these recovery times were affected by three factors, a factor based on the agency, a factor based on historical information on the hazard and the factor based on the bridge characteristics. For this thesis, those factors were all considered 1 according with the case studies herein presented. Considering these recovery times, Table 6.3 shows the adopted recovery functions for each case. The recovery functions were based on the Table 6.2. Figure 6-7 illustrates the respective recovery functions. Note that the recoveries are normalized from 0 to 1.

Table 6.3 - Considered parameters for the recovery functions per recovery time

Recovery function	**Recovery time**	**Input parameters***
Critically over damped recovery	3 days	$L_0 = 1$
		$\omega = 10$
	180 days	$L_0 = 1$
		$\omega = 0.05$
Lognormal cumulative recovery	270 days	$L_0 = 1$
		$\theta = 5.0$
		$\beta = 0.27$

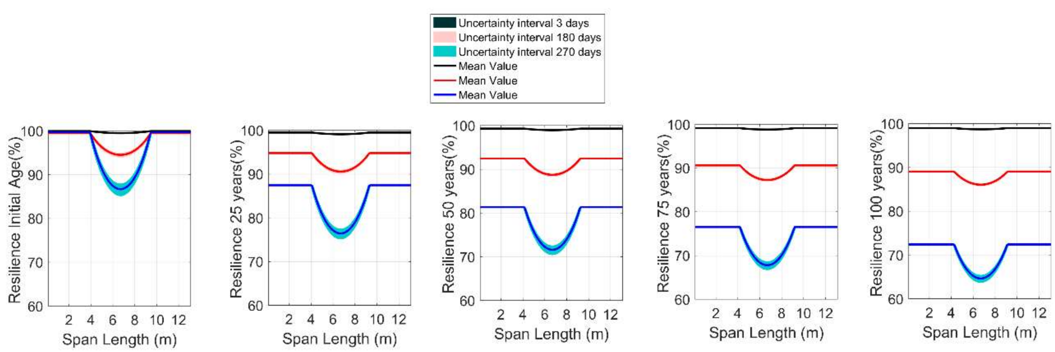
*estimated parameters according with the recovery time established.



a) b) c)

Figure 6-7 – Recovery functions for the different recovery times: a) 3days; b) 180 days; c) 270 days

Considering the different recovery functions, the time-dependent resilience of the bridge given the hazard event is depicted in Figure 6-8. As expected, the resilience decreased over the time wherein the higher losses were observed for the worst time recovery of 270 days.



a) b) c) d) e)

Figure 6-8 – Time dependent resilience estimation over the span length: a) initial age; b) 25 years; c) 50 years; d) 75 years; e) 100 years

Figure 6-9 shows the resilience considering the worst location of the damage, the mid-span. Note that the resilience presented a final value of 65% since the reliability did not reach the value of zero.

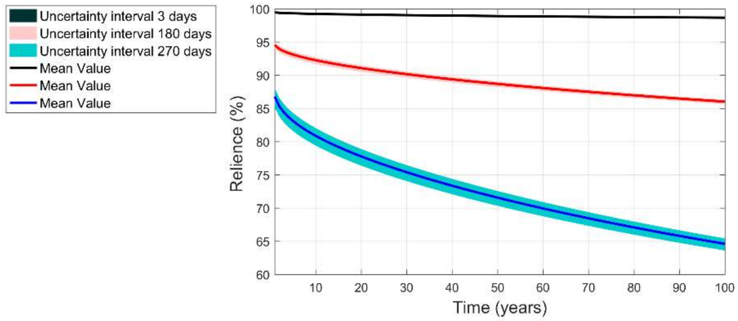
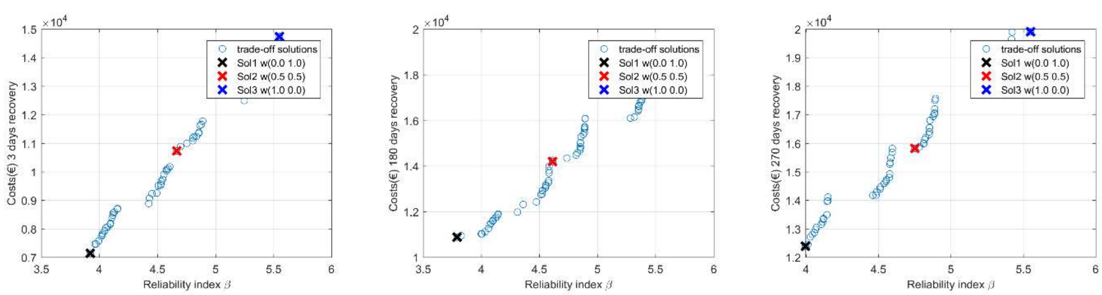


Figure 6-9 – Time dependent resilience considering the mid-span location of the damage

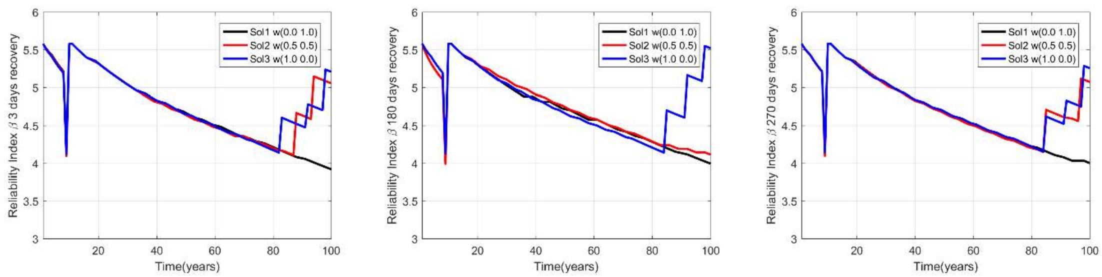
6.4.3 Optimization considering the hazard event

Considering the optimization defined in chapter 5, the current section aimed to introduce the effect of the hazard to investigate how the optimization should differ from the previous results on chapter 5. Note that it was assumed for the present application that each time the event occurs, the system was meant to be recovered to its original functionality, in this case to the initial reliability. For the sake of validation of the application, a recovery cost of $500€/m^2$ was herein assumed and applied on the possible affected areas, i.e. around the mid-span. As for the indirect costs, the calculation followed the equations (5-4), (5-5) and (5-6) wherein the duration activity considered the three different recovery times. The remaining parameters were considered the same from the previous optimization problem. Hence, the following Figure 6-10, Figure 6-11, Figure 6-12 and Figure 6-13 shows the optimization results for an occurrence of the hazard on the year 10.



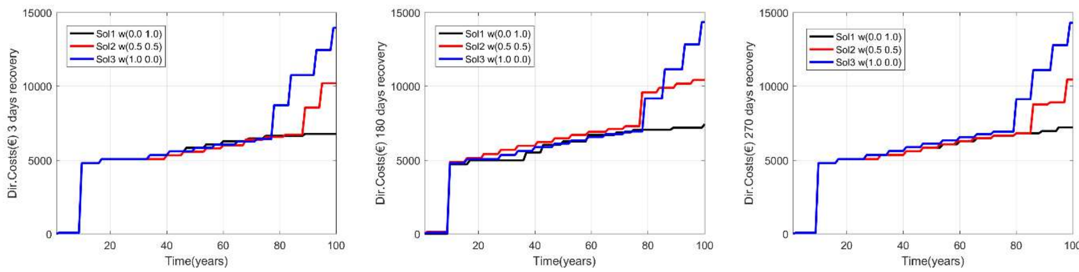
a) b) c)

Figure 6-10 – Pareto front: a) 3 days; b) 180 days; c) 270 days



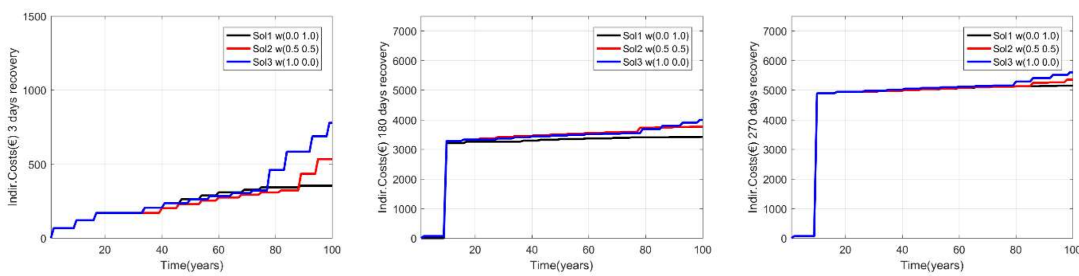
a) b) c)

Figure 6-11 – Reliability estimation: a)3 days; b) 180 days; c) 270 days



a) b) c)

Figure 6-12 – Direct Cost estimation: a) 3 days; b) 180 days; c) 270 days

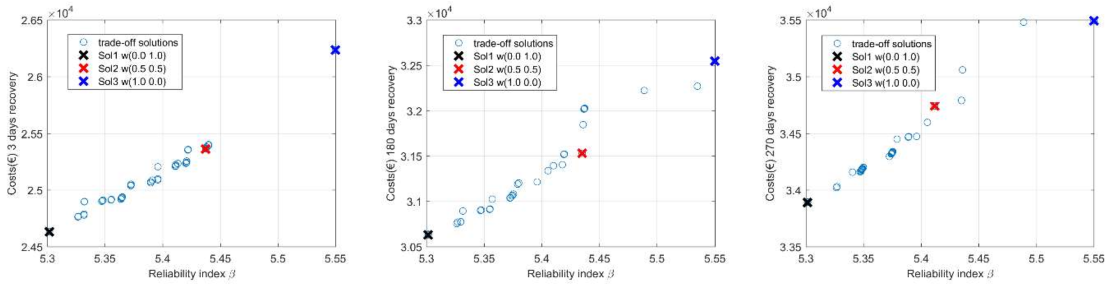


a) b) c)

Figure 6-13 – Indirect Cost estimation: a) 3 days; b) 180 days: c) 270 days

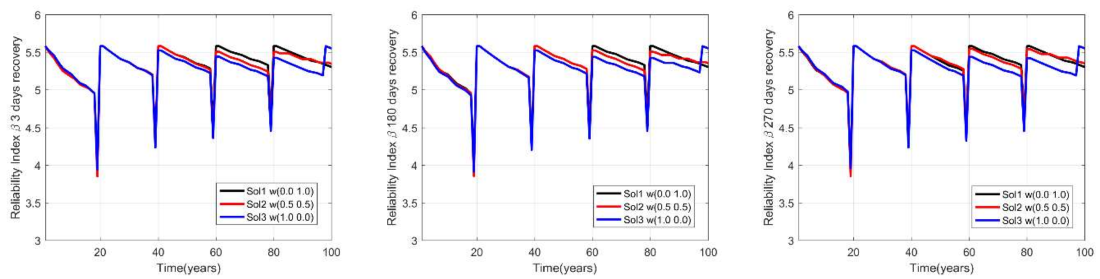
Note that for low recovery times, the indirect costs were very low. However, as the recovery increase, the indirect costs started to present a considerable amount on the overall costs. For example, for the worst recovery time, the indirect costs represented in some solutions almost 50% of the total costs.

Since those scenarios are difficult to predict, they can occur more than one time in the whole bridge life-cycle. The following Figure 6-14, Figure 6-15, Figure 6-16 and Figure 6-17 shows the hazard occurrence for a time interval of 20 years.



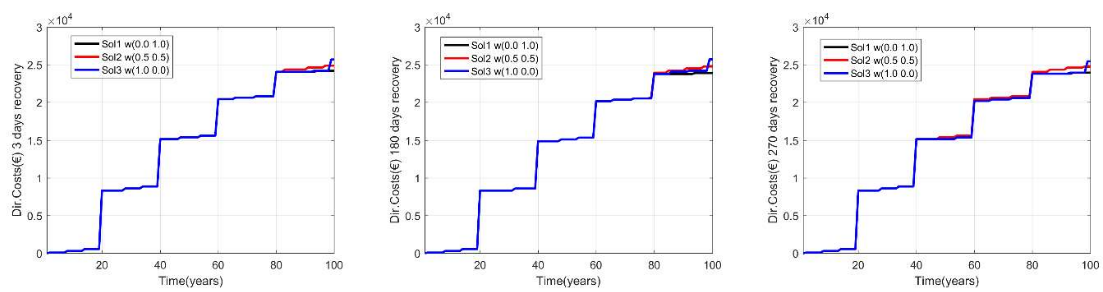
a) b) c)

Figure 6-14 – Pareto front: a) 3 days; b) 180 days; c) 270 days



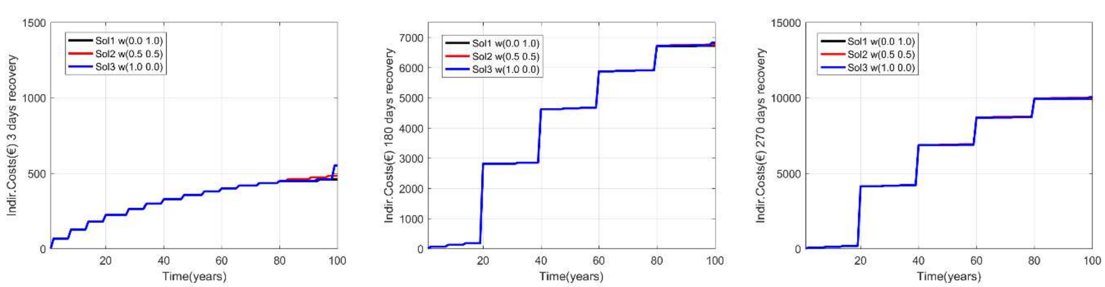
a) b) c)

Figure 6-15 – Reliability index: a) 3 days; b) 180 days; c) 270 days



a) b) c)

Figure 6-16 – Direct cost estimation: a) 3 days; b) 180 days; c) 270 days



a) b) c)

Figure 6-17 – Indirect Cost estimation: a) 3 days; b) 180 days; c) 270 days

Note that as more frequent the event became to occur, the less optimal different solutions were provided by the algorithm. Such aspect was influenced by the assumption of the fully recovery of the event. Since one of the objectives to be maximized by the optimization, i.e. the performance, was already maximized by an *a priori* imposition, the algorithm tended to show less optimal solutions.

Despite all the practices of maintenance to keep the structure with a good performance, another option could be the application of mitigation actions. Normally on bridge strikes, the mitigation activities are more concerned on traffic signs on the road to alert the drivers, particularly the trucks, for the danger of the hazard. Another way to tackle these problems could be to provide some guidelines as well as professional training to the drivers to reduce the risk of happening those events and preserve the infrastructure. The next section aims to apply the concept of resilience to the same network defined in the chapter 5.

6.5 Practical case B: Network system

This practical application aimed to study the system resilience of the network defined on the chapter 5. Note that the previous application dealt with system resilience at the bridge level, wherein more detailed calculation was carried out based on the damage location and on the KPIs previously obtained on the other chapters. However, for the network analysis, qualitative based KPI revealed to be more practical since more than one bridge is analyzed at the same time. Hence, this application dealt with the condition-based index KPI for performance estimation. Along the network, each bridge is subjected to different hazards according with their location and the surrounding environment. For example, a bridge located over a river is subjected to different hazards when compared to a railroad bridge. In this application, it was assumed that both bridge A and C are likely subjected to bridge strikes and bridge B to a flood occurrence.

The flood hazard carries out multiple failure modes in a bridge. Considering the system of bridge is divided into the foundation, pier and deck, the failure modes can occur at the foundation level, given by the scouring effects, at the pier and deck level given by the hydraulic pressure that the effect of dragging and lifting. For a deeper understanding of each failure mode, the readers are referred to [128]. Here, the bridge was considered simply supported with the deck highly susceptible to the dragging and lifting effects given the occurrence that the flood reaches the height of the deck. Since its quantification is out of the scope of this application given that there is not enough information of the required variables for its calculation, its loss is estimated by an immediate decrease on the condition state index. Hence, Table 6.4 shows the considered losses in case of the hazard occurrence.

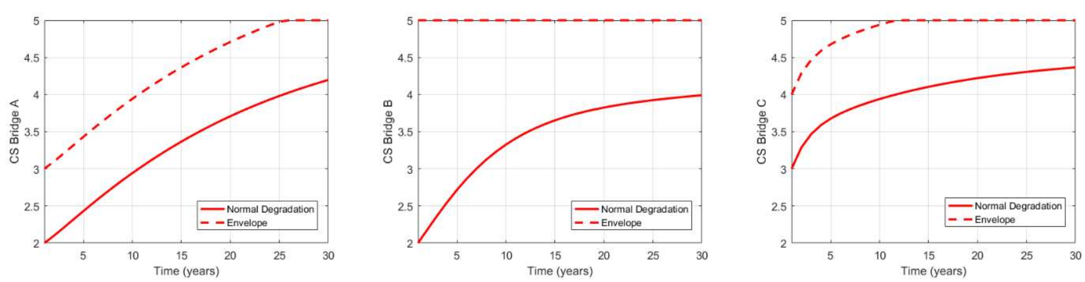
Table 6.4 – Considered loss estimation for each bridge

	Considered loss estimation
Bridge A	$\Delta CS = -1$
Bridge B	Worst condition state assumed, i.e. $CS = 5$
Bridge C	$\Delta CS = -1$

Either for bridge A and C, the loss was estimated for a possible bridge strike and given the intensity of damage discussed in the previous chapter, a loss of 1 for the condition state was assumed. As for the bridge B, the effect of the flood was given by whether the water level reaches the deck. In case the level of the water reaches the deck, the drag and lifting forces were considered to create high instabilities leading thus to assume that the worst condition state was reached, i.e. condition state 5.

6.5.1 Consideration of the hazards on the long-term analysis

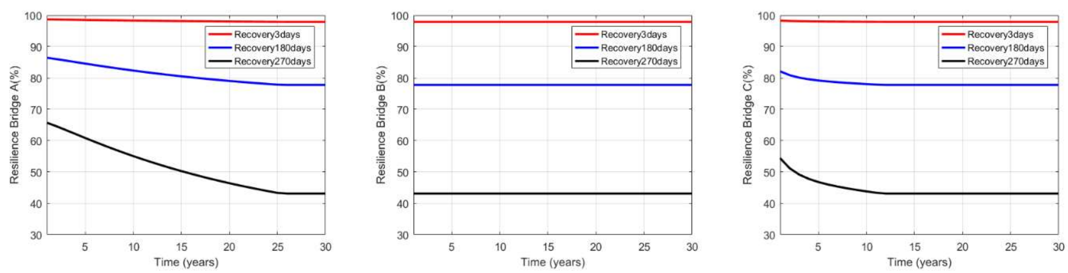
Considering the assumptions previous defined, Figure 6-18 shows the envelope of the degradation curves in case of a hazard occurrence in each bridge. Note that in the case of the bridge B, since it was assumed that the bridge was considered to go for the worst condition state, the envelope was the same value for all the years.



a) b) c)

Figure 6-18 – Time dependent envelope damage due to an hazard event

Considering the recovery curves previously defined in Figure 6-7, the time dependent resilience is depicted in Figure 6-19 for the three different bridges. Again, for the case of the bridge B, the time-dependent resilience was considered constant since the condition state after the hazard event was the same.

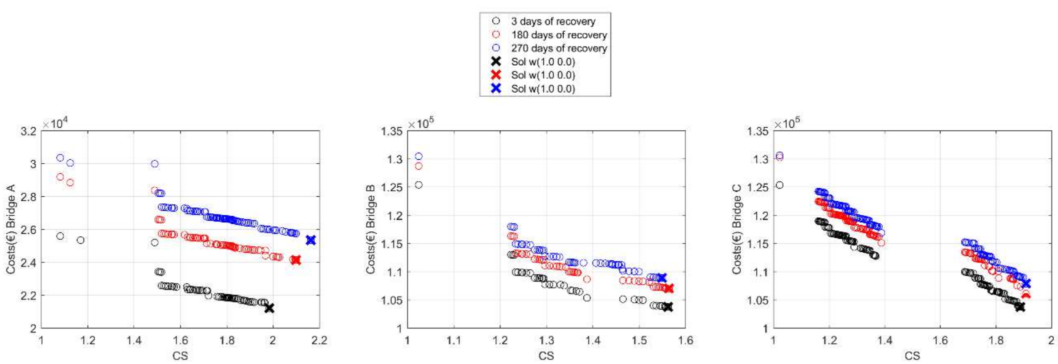


a) b) c)

Figure 6-19 – Time dependent resilience estimation

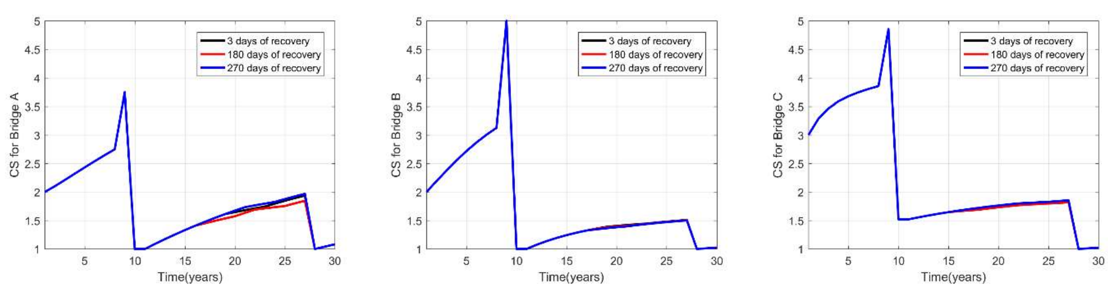
6.5.2 Optimization considering the hazard effect

The optimization problem considered the same conditions defined in the chapter 5 with the inclusion of the hazard event. For the sake of simplicity, only one optimal solution was analyzed for the three different recovery times. The results for each bridge are thus shown in Figure 6-20, Figure 6-21, Figure 6-22 and Figure 6-23.



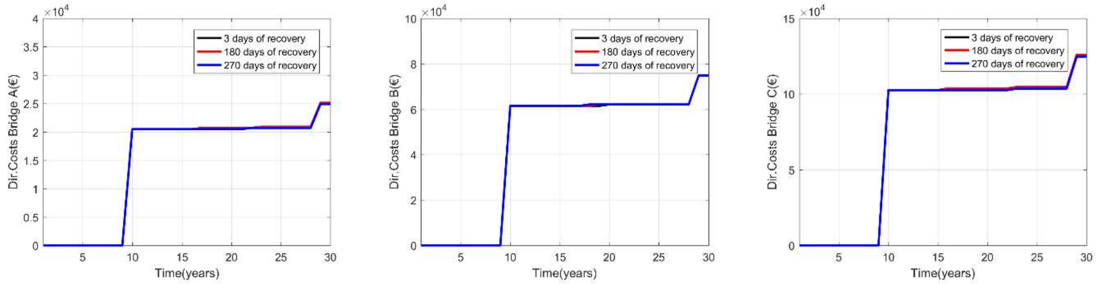
a) b) c)

Figure 6-20 – Pareto front: a) Bridge A; b) Bridge B; c) Bridge C



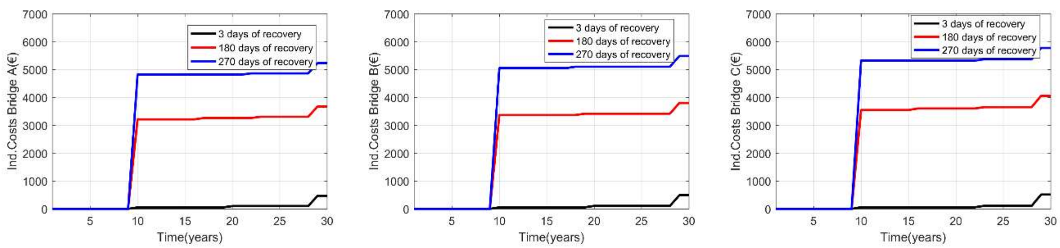
a) b) c)

Figure 6-21 – Condition state index: a) Bridge A; b) Bridge B; c) Bridge C



a) b) c)

Figure 6-22 – Direct Cost Estimation: a) Bridge A; b) Bridge B; c) Bridge C



a) b) c)

Figure 6-23 – Indirect Cost Estimation: a) Bridge A; b) Bridge B; c) Bridge C

From Figure 6-20, Figure 6-21, Figure 6-22 and Figure 6-23 it was observed that the influence of the hazard had a considerable effect on the condition state of the bridge and therefore in the indirect costs since they were proportional to the increase on the days of recovery. Note that the direct costs were practically the same for the three different types of recovery. Also, it was simulated an occurrence of a single event in the whole life cycle of the bridges. However, in the reality, more than one occurrence can happen in the life cycle and thereby affect more the indirect costs. Considering the combination effects of each bridge by the same procedure as in chapter 5, the proposal of the optimal maintenance activities considering one optimal solution is depicted in Table 5.7. Because the bridges were meant to recovery to their initial state after a hazard event, the optimization did not give as many maintenance activities since one of the objectives was already maximized.

Table 6.5 – Proposal of optimal M&R actions and costs for each bridge

	Time hazard occurrence	**3 days**			**180 days**			**270 days**		
	Year 9	ID1	ID2	ID3	ID1	ID2	ID3	ID1	ID2	ID3
Bridge A	Year(s) of application	20	-	29	17,23	-	29	22	-	29
	Final Cum Dir. Costs		24942.0			25153.0			24934.0	
	Final Cum Indir. Costs		468.0			3675.0			5232.0	
	Resilience		98%			82%			55%	
Bridge B	Year(s) of application	20	-	29	19	-	20	18	-	29
	Final Cum Dir. Costs		74827.0			74840.0			74853.0	
	Final Cum Indir. Costs		490.0			3802.0			5487.0	
	Resilience		98%			77%			43%	
Bridge C	Year(s) of application	23	-	29	23	-	29	23	-	29
	Final Cum Dir. Costs		124651.0			125789.0			124651.0	
	Final Cum Indir. Costs		514.0			4058.0			5776.0	
	Resilience		98%			77%			44%	
Network	Final Dir Costs		224420.0			225782			224438.0	
	Final Ind. Costs		1472			11535			16495	
	Resilience		98%			77%			43%	

6.6 Final remarks

The present chapter aimed to highlight the importance on considering the hazard events on the management of a bridge. The chapter firstly focused on describing the most common types of hazard events on the infrastructure railway, more specifically on the bridge case. Moreover, an introduction to the recovery of the system in case of a hazard event was covered by thoroughly carrying out a review on the literature regarding the resilience concept on infrastructures in general and thus on the case of bridges. Furthermore, several recovery functions associated to the resilience concepts were herein introduced as alternatives to model the recovery of the bridge. This concept was after applied in two case studies. The first case study was related to one single bridge subjected to bridge strike events. The first step aimed to estimate the damage bridge location and the severity of the damage. Considering the assumptions for the case study, the severity of the damage was considered of about 21% to 25%. After the damage estimation, a resilience calculation was carried out by considering three different scenarios according with the preparedness of the society. To finalize, an optimization problem considering the hazard event was investigated. The results showed that since the recovery of the bridge was meant to be renewed as new, the effects of the maintenance activities were less evident since one of the objectives were already being a priori maximized. This fact was more evident if more hazard events were added with a specific return period. The second case study considered the network presented in chapter 5. In this case, three hazard scenarios were considered according with the bridge environment exposure, i.e. bridge strike for bridge A and C and flood event for bridge B. The results have shown that the network presented lower resilience for events such as floods wherein the bridge was considered in the worst condition state since the water level reach the deck resulting thus on high instabilities due to the hydraulic forces of drag and lifting. Again, the optimization results showed less maintenance activities, when compared to the previous chapter 5, since the bridge was meant to a fully recover after the hazard event. In terms of overall costs, the indirect costs revealed to be more influent as longer as the bridge took to be recovered. The next chapter aims to combine the concepts discussed along this thesis into a case study of a RC bridge subjected to a seismic event and corrosion.

Chapter 7

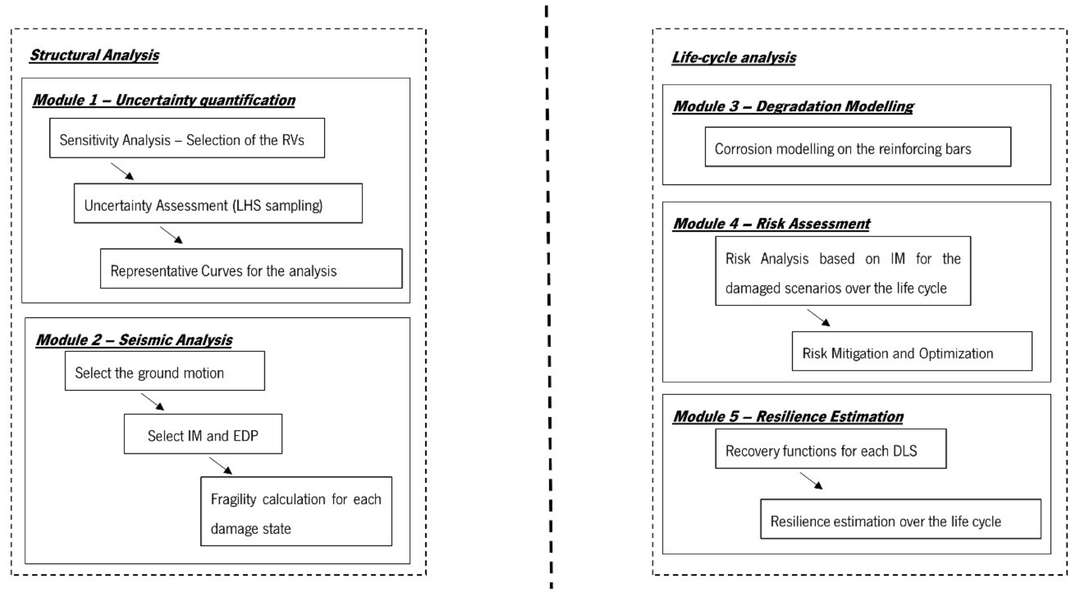
7 Risk and resilience-based assessment of a railway RC bridge subjected to earthquake and corrosion scenarios

7.1 Introduction

This chapter aims to apply a methodology considering a risk and resilience-based assessment. This methodology is validated in a railway bridge located in Albergaria dos Doze, Alfarelos, center of Portugal, which is part of the railway network from the Region of Santarém/Leiria, wherein the seismic actions are considered important. Note that in the methodology presented in this case study, some assumptions were considered. Moreover, it is noteworthy to mention that due to the lack of some information on the case study, some of the assumptions were validated through literature. Figure 7-1 shows the approach adopted for this case study. The approach was divided into structural analysis and life cycle analysis aiming to cover all the concepts discussed in the previous chapters.

Regarding the structural analysis, a 2D finite element (FE) model was initially developed to study the effect of the variables' uncertainties on the structural response by conducting a sensitivity analysis and sampling analysis through Latin Hypercube Sampling (LHS). Thus, data from three main percentiles of the LHS was considered with the purpose of considering three main deterministic models that represent the whole model to reduce the computation time. After uncertainty analysis, fragility assessment for seismic analysis was thereby investigated.

Concerning the life cycle analysis, a degradation model was defined based on corrosion to investigate the performance of the structure for different periods. Therefore, time-dependent risk assessment was computed to investigate the behaviour of the bridge without any maintenance and mitigation actions. To minimize the obtained risk, a multi-objective optimization (MOP) based on genetic algorithms (GA) was proposed to investigate the optimal solutions of maintenance with and without mitigation actions given by a Pareto front, according with the formulation presented on chapter 5. Those optimal solutions were hereafter discussed and compared in terms of their minimization on the post-event, by estimating the resilience.



Legend: RV – Random Variable; LHS – Latin Hypercube Sampling; IM – Intensity measure; EDP – Engineering Demanding Parameter; DLS – Damage Limit State

Figure 7-1 – Step approach adopted for this case study

7.2 Structural analysis

7.2.1 Bridge geometry and modelling considerations

The bridge case study is located between Albergaria dos Doze-Alfarelos in the center of Portugal. The bridge was built in 2005 and it is part of the north railway line network, located in the Region of Santarém/Leiría The total length of the bridge is around 65m and it is divided into four simply supported spans, see Figure 7-2.

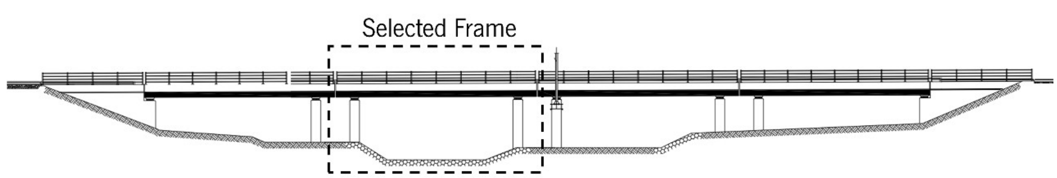


Figure 7-2 – Bridge longitudinal view

Because the bridge is divided into four simply supported spans, it was noticed that the frames worked independently. Therefore, for the modelling consideration, the bridge was represented by the frame highlighted in Figure 7-2. Figure 7-3 shows the cross section, the top view, and the longitudinal view, respectively.

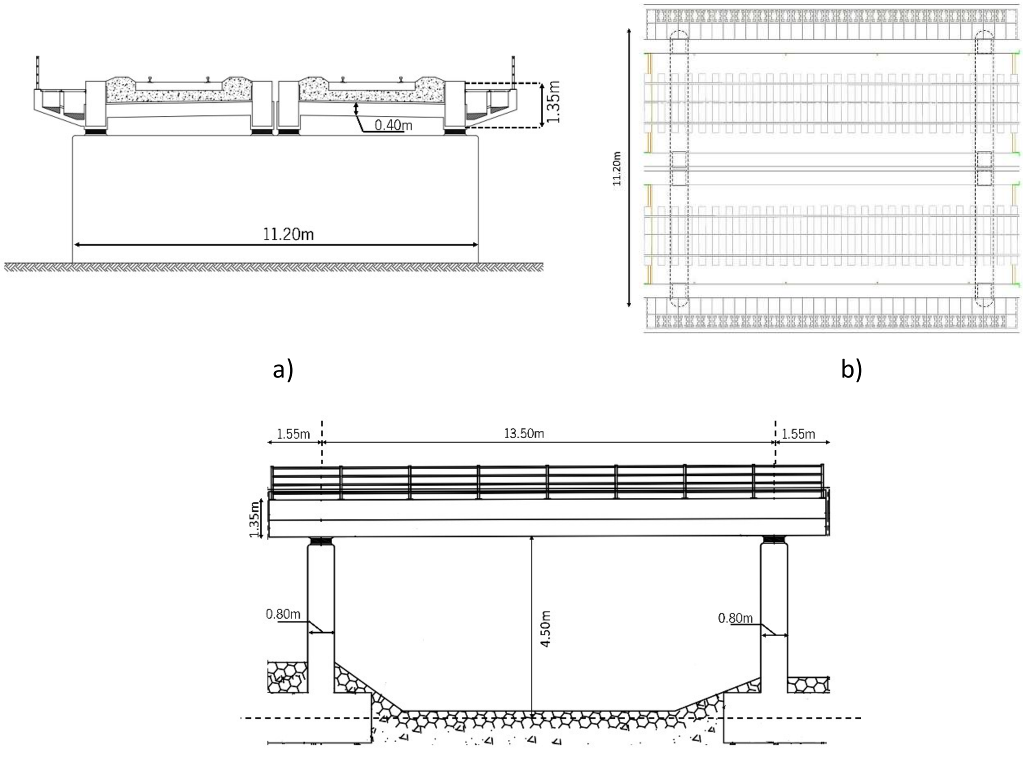


Figure 7-3 – Frame view: a) Cross section; b) Top view; c) Longitudinal view

The superstructure is composed of two beams of 1.35m high and a 0.4m slab connecting both beams, to conform a "H" cross section shape in each direction, see Figure 7-3 a). Furthermore, the frame is composed of two systems disposed in series (pier-bearing), resulting in a global parallel system. The connection between piers and deck is done through elastomeric bearing devices, see Figure 7-3 c).

Cover measurement tests performed after the bridge construction revealed that there are zones with lack of concrete cover, which renders the bridge susceptible to corrosion deterioration [349]. Furthermore, the bridge was in a region wherein the seismic actions are important and thus object of analysis in this study.

Concerning the bridge modelling, a 2D FE model for the bridge was developed using the software TNO DIANA. Note that a 2D model was selected due to the interest of analyzing the response of the bridge along its longitudinal direction given its higher vulnerability to seismic loads in comparison with the transversal direction. An overview of the FE model is herein presented to understand all the assumptions in the model for structural analysis. For the deck, all the elements were accounted and modelled with linear elastic beam elements since the damage is not expected to occur in these elements. As for the piers, non-linear beam elements were modeled assuming a total strain fixed crack model with a brittle tensile behavior, a compressive behavior based on EC2 EN 1992-1-1 [350] and constant shear retention factor. As for the steel material, a Von Mises plasticity model was adopted. Concerning the reinforcing steel quantification, this study adopted a total reinforcing steel mobilized in the longitudinal direction of $0.28m^2$ for each pier. Regarding the bearing devices, they were modelled based on the experimental results and the model proposed by [351] to characterize the frictional behavior of steel-PTFE contact interface and define spring elements with elastic-plastic hysteretic backbone curves. Table 7.1 shows the parameters considered for the constitutive laws as well as Figure 7-4 and Figure 7-5 illustrate the constitutive laws and the considerations on the modelling, respectively.

Table 7.1 – Considered parameter for the modelling of the constitutive laws

Material	Constitutive Law	Parameter	Adopted value
Concrete	EC2 EN 1992-1-1 (Compressive)	Compressive strength (f_{cm})	33 MPa
		ε_{c1}	0.002
		ε_{cu}	0.0035
	Brittle (Tensile)	Concrete Tensile strength (f_{ctm})	2.6MPa
Steel	Von Mises	Steel yielding strength (f_{ym})	560MPa
		Yielding strain (ε_{sy})	0.0028
		Ultimate steel strain (ε_{su})	0.15
Bearing	Elasto-Plastic	Yielding Force (F_y)	1.20MN
		Elastic Stiffness (K_e)	1.0GN/m

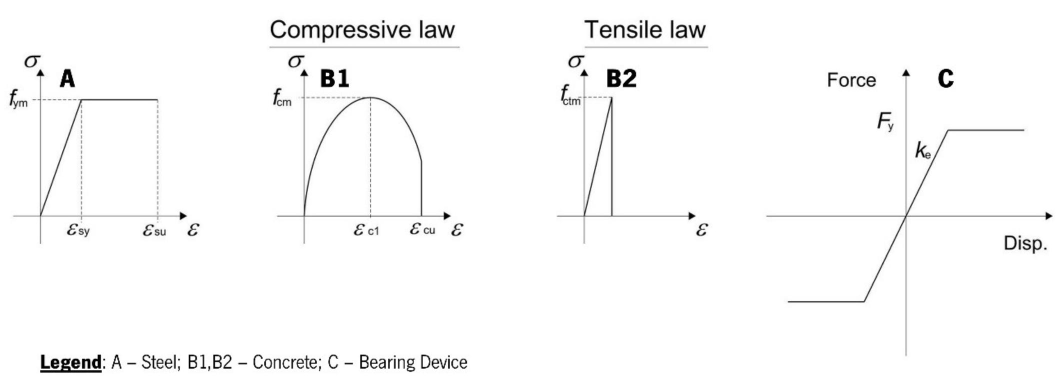


Figure 7-4 – Constitutive laws

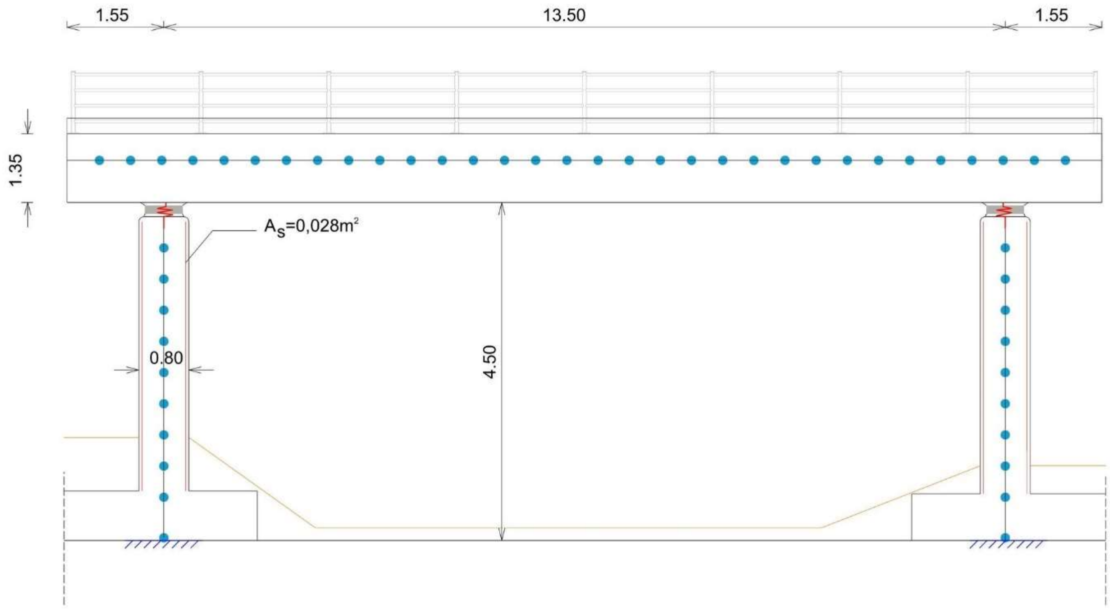


Figure 7-5 – Bridge modeling considerations

7.2.2 Uncertainty quantification

Uncertainty analysis is a widely adopted approach to address the uncertainty of the variables. However, these approaches present a considerable drawback in terms of computational time. Therefore, sensitivity analysis as well as sampling techniques should be embraced to solve this problem. Sensitivity analysis is commonly adopted to reduce the number of random variables (RV), given in here by X_i. Its computation is hereafter conducted, as recommended in [101, 102], given by the following equation:

$$b_k = \sum_{i=1}^{n} (\Delta y_k / y_m)/(\Delta x_k / x_m) \,.\, CV \tag{7-1}$$

where b_k is the importance measure of parameter *k*, Δy_k is the variation of the structural response due to the deviation of Δx_k relative to the mean value of the parameter x_m, y_m is the average response, *n* is the number of generated parameters, and CV is the coefficient of variation given by the ratio between standard deviation and the mean value. For each obtained value of b_k, it was performed a lower bound analysis, corresponding to the percentile 5%, and a upper bound analysis, corresponding to the 95% percentile. Thereby, a total number of the $(X_i \times 2 + 1)$ analyses were thus performed.

Regarding the sampling techniques, as Monte Carlo simulation is highly costly in terms of computational time, an optimal solution commonly adopted refers to the Latin Hypercube Sampling (LHS). The final output is a matrix with dimensions corresponding to the number of simulations and the number of RVs.

The literature presents several algorithms to obtain the LHS matrix. For this study, an algorithm provided by the design of experiments toolbox of MatLab [352] was adopted.

The input variables accounted with uncertainties mainly on the resistance properties of the concrete and steel as depicted in Table 7.2. The remaining variables concerning the geometric properties of concrete were considered deterministic, see Figure 7-5.

Table 7.2 – RV considered for this study [109, 143, 353]

Random Variable	**Distr. Type**	**Mean Value**	**COV**
Concrete compressive strength (f_{cm})	Normal	33 MPa	15.2%
Concrete Tensile strength (f_{ctm})	Normal	2.6MPa	18 %
Concrete Pier Young Modulus ($E_{c,pier}$)	Normal	30.5GPa	7.2%
Concrete Deck Young Modulus ($E_{c,deck}$)	Normal	33.5GPa	7.2%
Concrete density (γ_c)	Normal	25 kN/m3	8.2%
Steel yielding strength (f_y)	Normal	560 MPa	5.6%
Ultimate steel strain (ε_{su})	Normal	0.15	24%
Steel Area (A_s)	Normal	Nominal	2% *
Steel young modulus (E_s)	Normal	200GPa	4.4%

*The value is considered for the initial age

Considering the formulation herein explained, the importance parameter was measured by the peak displacement on the top of the pier. The ultimate capacity of the piers were obtained in a primary stage by a non-linear static pushover analysis. Non-linear static pushover analysis is considerably less time consuming when compared to time history analysis. Furthermore, the peak displacement estimation for the pushover in a sensitivity analysis gives a reasonable estimation of the most influencing RV. Thereby, considering this analysis, the results are shown in Figure 7-6. Note that the obtained results from equation (7-1) were standardized to the higher importance measure for the sake of simplicity on visualizing the results. The critical parameters were those that exceed the cut off dashed line, normally defined by the user. For this study, a 10% cut off was considered. From Figure 7-6, it was observed that f_{cm} and f_{ym} were the two most influencing parameters on the top peak displacement. Following the cut off previously defined, the RVs of the system are from now one represented by the following vector:

$$X = (f_{cm}, E_{cm,pier}, \gamma, f_{ym}, A_s, E_s)$$

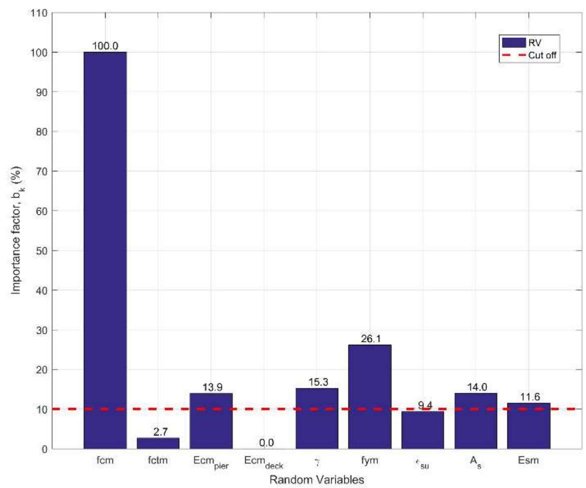


Figure 7-6 – RV influence on the analysis

Considering the RV vector previously defined, 100 samples were initially considered through LHS to investigate the dispersion on the system response. Considering the LHS step approach defined in section 3.1.1, chapter 3, the responses are depicted in Figure 7-7.

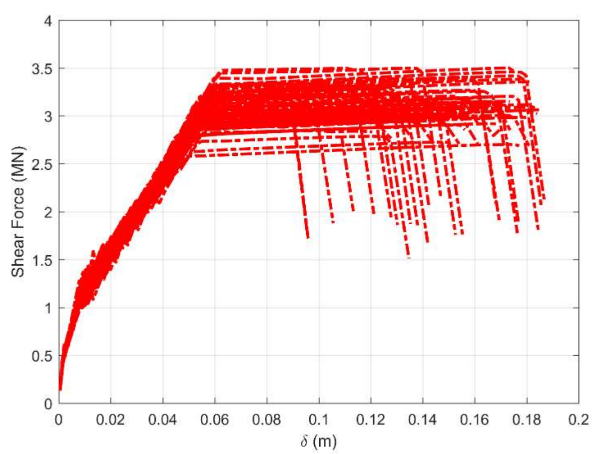


Figure 7-7 – Sampling response for force-displacement analysis

Moreover, a convergency test was performed to confirm the accuracy of the 100 samples. The convergency consisted of investigating the ratio between the cumulative mean on the n^{th} simulation over the mean for the 100 simulations, see Figure 7-8. Observing the figure, it was concluded that as the number of simulations increases, the convergency tended to a smother oscillation and thus to an approximation to the mean value considered for the 100 simulations. Hence, it was concluded that the initial number of simulations were enough to represent the variation of the sampling response.

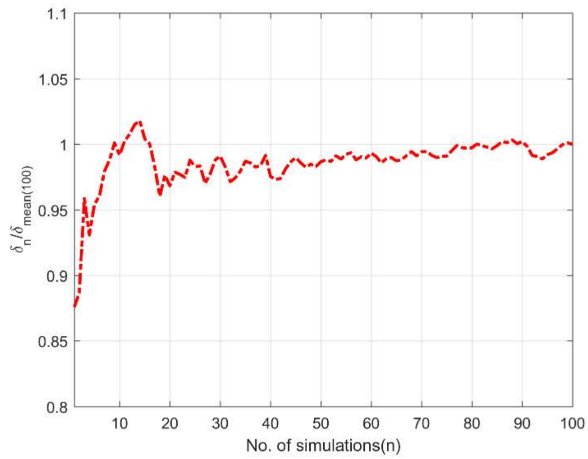


Figure 7-8 – Convergence of number of simulations

Despite the number of simulations serving quite well the presented model, the dimension of the problem is still large and highly costly in terms of computational time, as the further aim is to perform fragility assessment analysis. Bearing this in mind, data from three main quantiles was obtained with the idea of considering three main deterministic models to consider the material properties variation. Such assumption was reasonable to assume since the obtained curves presented the same behavior. Also, this approach was already implemented by other authors [354]. This simplification significantly reduces the computational time of the model. Therefore, from now on, three representative models corresponding to the quantiles of 5%, 95%, and the mean value will be considered, see Figure 7-9.

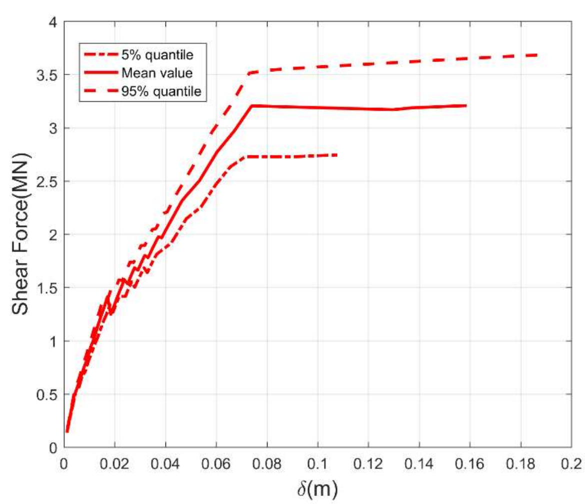


Figure 7-9 – Force-displacement analysis for 5% quantile, mean value and 95% quantile

7.3 Fragility assessment

7.3.1 Ground motion selection

When real ground motion records are unavailable, artificial accelerograms might be a reasonable choice to conduct time history analysis. Eurocode 8 [355] recommends the use of artificial accelerograms compatible with the elastic response spectrum. As such, a significant random number of accelerograms compatible with the elastic acceleration response spectrum were generated resorting to the software SIMQKE_GR [356]. The Eurocode acknowledges a minimum number of seven artificial accelerograms to have a consistent representative response. Some studies regarding seismic assessment of reinforced concrete bridges [357] discuss the adequate number of artificial accelerograms to be used. As for this work, ten artificial independent accelerograms were adopted based on the study of [358]. For each accelerogram, a duration of 14s with a time rise of 2s was considered as well as an envelope function with a trapezoidal shape. The resulting accelerograms were compatible with the elastic response spectrum for the Eurocode 8 type 2 seismic action (near-field), with a return period of 475 years and viscous damping of 5%, see Figure 7-10. The region, based on the parameters of Eurocode 8 Annex, was Soure with an importance factor γ_I of 1.0, a value of reference peak ground acceleration of 1.1 m/s^2, and a soil type C (S=1.60).

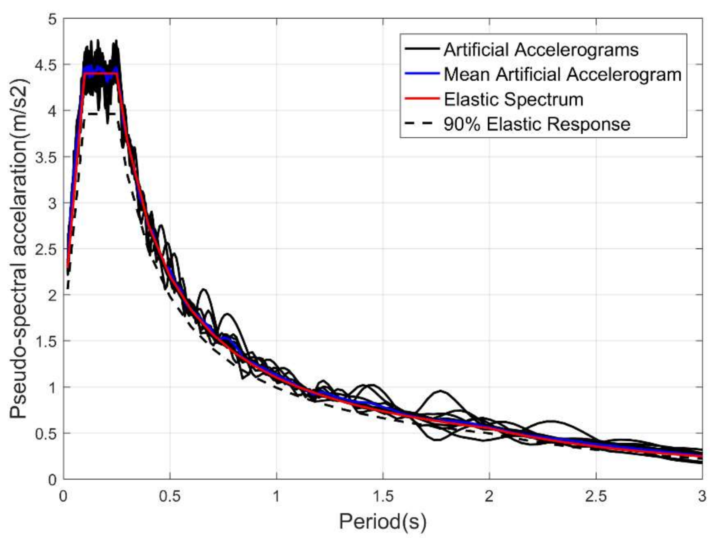


Figure 7-10 – Pseudo-spectral accelerations for the considered accelerograms

7.3.2 Selection of the intensity measures and engineering demanding parameters

The selection of the optimal intensity measure (IM) is of utmost importance to obtain a lower dispersion on the results. Mackie and Stojadinovic [359] studied the efficiency of several IMs in terms of the dispersion of the output. The authors claimed that an efficient IM must be practical, sufficient, effective, efficient, and robust. Additional studies regarding the efficiency can be seen in [360], wherein several IMs were compared and applied to a set of highway bridges, and [361] with another application to highway bridges for a set of IMs in which a correlation between them was investigated.

The literature reports a wide range of IMs such as peak ground acceleration (PGA), peak ground velocity (PGV), peak ground displacement (PGD), Arias intensity I_A, spectral acceleration S_a among others. As regarding bridges applications, the most famous IMs are PGA and S_a. The adoption of the best IM is very debatable. Nevertheless, following the literature of seismic assessment regarding bridges, the majority of studies considers PGA as one of the IMs to adopt for the seismic assessment. For example, [360] states that PGA and S_a are the most optimal parameters for the response. Bearing this in mind, PGA is herein considered in this work.

Concerning the engineering demanding parameters (EDP), before choosing the most adequate for the work in context, it matters to understand the critical components of the structure. Hence, for this work, it was considered the pier and the bearing device as critical components of the bridge, therefore top displacement of the pier as well as the longitudinal displacement of the bearing device were herein chosen for further analysis.

7.3.3 Damage Analysis

The fragility assessment of bridges is developed based on several damage states. A great part of the works on bridges recalls the use of damage states to the HAZUS code [362]. Normally this code refers to five damage limit states (DLS), namely no damage, slight, moderate, extensive, and complete damage. Examples of the application of this scale measure can be seen in [357] on the section of fragility functions for road and railway bridges. Also, other codes, such as Eurocode 8, part 3 [363], refer to three DLS applied to buildings, damage limitation, significant damage, and near collapse.

For the DLS definition, a different damage state criterion must be considered for each component. Regarding the piers, despite the existing codes previously mentioned for bridges, several authors propose different limit states thus being difficult to stick to a specific work. For this study, the work of [364] was considered. In this work, the DLS were based on EDP obtained from the non-linear static pushover analysis wherein relationships between the yielding, δ_y, and ultimate displacement, δ_u.are built to obtain the different DLS, see Table 7.3. As for the bearing device, some assumptions were also made based on its typology. Thus, the DLS was based on the studies of [365], where it was stated that displacements less than the design value imply no damage for the bearings. Conversely, higher values translate into more serious damage states being the ultimate displacement 5 times higher than the design value, meaning thus the complete destruction. Concerning these assumptions for this study, Table 7.3 shows the considered damage states for the bearing devices. Furthermore, since this case study deals with uncertainty, a DLS was herein defined for each deterministic model above-defined (5%, mean, 95%). This assumption served the purpose of introducing the effect of the uncertainty on the materials instead of assuming the mean values for the fragility assessment, as commonly seen in almost all studies about bridge fragility assessment. As for the bearing devices, uncertainty was also assumed. Since the information on these components was very limited, a lower and an upper bound around the most probable value was defined, i.e. 50 ± 25 and so on. Such assumption relied on the information provided by the FIP Industriale catalogues [366], an Italian company that produces bearing devices, which recommends lower and upper bounds for bearing devices. The results are thus summarized in Table 7.4.

Table 7.3 – Damage Limit State definition

Component	**Minor**	**Moderate**	**Major**	**Collapse**
Pier displacement*	$\delta_{pier} \leq 0.7\delta_y$	$\delta_{pier} > \min\,[1.5\delta_y, \left(\frac{1}{3}\right).(\delta_u - \delta_y)]$	$\delta_{pier} > \min\,[3.0\delta_y, \left(\frac{2}{3}\right).(\delta_u - \delta_y)]$	$\delta_{pier} > \delta_u$
Bearing displacement*	50mm	75mm	125mm	250mm

*values based on the longitudinal direction

Table 7.4 – Damage limit state quantification for piers and bearings

Component	**Quantile**	**Minor**	**Moderate**	**Major**	**Collapse**
Pier displacement (m)	5%	0.042	0.067	0.073	0.080
	Mean	0.042	0.082	0.103	0.125
	95%	0.042	0.090	0.147	0.190
Bearing displacement (m)	5%	0.025	0.0375	0.0625	0.125
	Mean	0.050	0.075	0.125	0.250
	95%	0.075	0.1125	0.1875	0.375

7.3.4 Fragility calculation

Having defined the IM, the EDP as well as the damage states, this section aims to compute the fragility curves. The fragility expression is given by a probability of exceedance of the EDP for a specific damage limit state at a given IM as follows:

$$Fragility = P(EDP \geq DLS|IM) \tag{7-2}$$

The cumulative functions that build the fragility curve follow normally a lognormal distribution being the expression herein defined as follows:

$$P(DLS|IM = x) = \Phi\left(\frac{ln\left(\frac{x}{\theta}\right)}{\beta}\right) \tag{7-3}$$

where $P(DLS|IM = x)$ is the probability of exceedance as defined above with an $IM = x$; Φ is the standard normal cumulative distribution function, θ is the mean and β the standard deviation of $\ln\,(IM)$. It is noteworthy to mention here that there are several approaches to obtain the fragility curves. For the present work, Baker [367] approach was adopted. Following this methodology, an estimation of the parameters $\widehat{\theta}$ and $\widehat{\beta}$ were obtained through maximum likelihood estimation (MLE). Before MLE, it was computed the number of events, in which the EDP was exceeded, out of the total number of events given by the number of artificial earthquakes. Thereby, a binomial distribution for the collapse probabilities were estimated by:

$$P(n_c\ out\ of\ N_{events}|IM = x) = \binom{N_{events}}{n_c} p^{n_c}(1 - p_i)^{N_{events} - n_c} \tag{7-4}$$

where p is the probability of exceeding the DLS at $IM = x$. The process is repeated for all the IMs, the likelihood is given by the product of the binomial probabilities and therefore, optimization is applied. To simplify the problem, the formula is re-written in the logarithmic form and given by:

$$(\widehat{\theta}, \widehat{\beta}) = \arg max \sum_{j=1}^{m}\left[n_c \ln\left(\Phi\left(\frac{ln\left(\frac{x}{\theta}\right)}{\beta}\right)\right) + (N_{events} - n_c)\ln\left(1 - \Phi\left(\frac{ln\left(\frac{x}{\theta}\right)}{\beta}\right)\right)\right] \tag{7-5}$$

The parameters were obtained by the MATLAB optimization function *fminshearch* [316]. Considering this approach to obtain the fragility curves, the total number of runs assembled 10 artificial earthquakes per PGA and for each quantile. As the level of intensity was considered to range from 0-1.5g, a total of 450 nonlinear time history analyses were performed. Table 7.5 and Table 7.6 show all the estimated parameters for the three quantiles for the pier and bearing device, respectively. From the obtained results, it was observed that the lower the DLS is, the less disperse were the results for the case of the pier component. Such fact was related to the dispersion of the displacement values for the different quantiles, which was very low for minor damages being the value around 0.60g. As for the moderate damage, this dispersion increased having values ranging from 0.85g to 1.11g. For higher damages, i.e. major and collapse, the dispersion on the results were even more evident as the structure was more likely to exceed the state for higher PGAs. It was also observed for the quantile 95% of the collapse DLS, the estimated parameter was not correctly estimated. Such fact was explained by no exceedance of the threshold established for that DLS in all 10 artificial earthquakes. Thereby, the optimization gave a mathematical error. Such a result could be fixed by considering a higher range of intensity or by lowering the thresholds defined in Table 7.4. The first option seemed to be more reasonable, though a considerable amount of computational time should be added to the calculation turning the problem considerably more time-consuming. The obtained results for the bearing device component resulted to be more dispersed than in the piers due to the range of values defined as threshold DLS for the different quantiles. In fact, with more information regarding experimental analysis, the results could led to more accurate insights. Nevertheless, as it was mentioned at the beginning of the chapter, the aim is to validate the methodology wherein the results could be easily updated. The obtained results can be consulted in Figure 7-11.

Table 7.5 – Estimated parameters for the piers

DLS	5% Quantile		Mean Value		95% Quantile	
	$\hat{\theta}$	$\hat{\beta}$	$\hat{\theta}$	$\hat{\beta}$	$\hat{\theta}$	$\hat{\beta}$
Minor	0.60	0.02	0.60	0.02	0.60	0.02
Moderate	0.85	0.11	1.02	0.10	1.11	0.08
Major	0.91	0.10	1.22	0.08	1.74	0.11
Collapse	1.00	0.08	1.37	0.09	5.71	0.14

Table 7.6 – Estimated parameters for the bearing device

DLS	5% Quantile		Mean Value		95% Quantile	
	$\hat{\theta}$	$\hat{\beta}$	$\hat{\theta}$	$\hat{\beta}$	$\hat{\theta}$	$\hat{\beta}$
Minor	0.40	0.03	0.70	0.08	0.93	0.10
Moderate	0.55	0.10	0.93	0.10	1.28	0.09
Major	0.81	0.08	1.37	0.09	1.74	0.13
Collapse	1.37	0.09	5.71	0.14	5.71	0.14

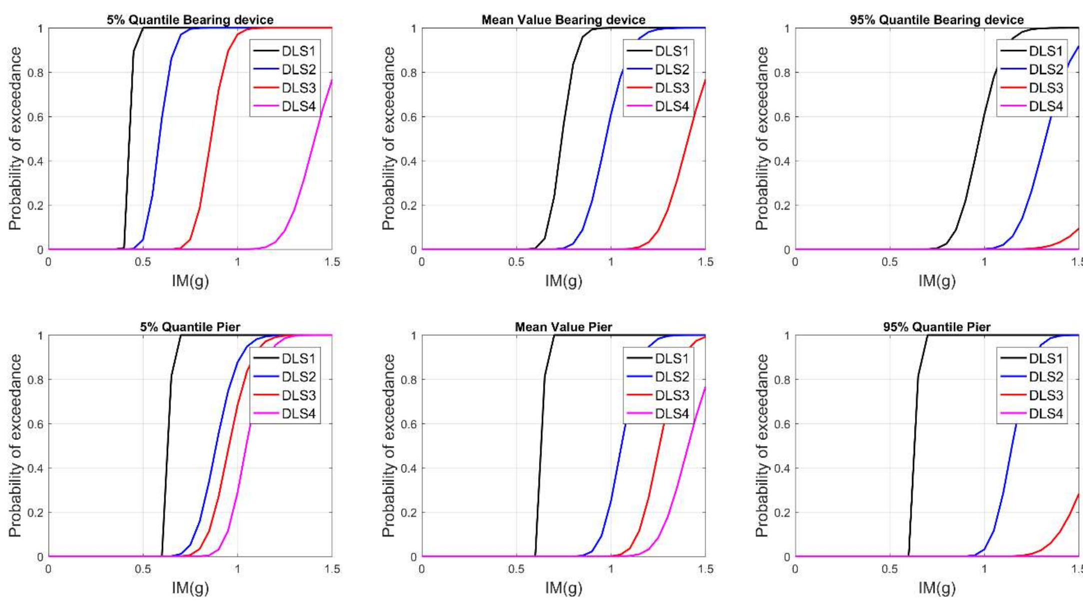


Figure 7-11 – Fragility curves for the piers and the bearings

7.3.5 Fragility assessment combination

After having obtained the estimated parameters through MLE for each component, the next step followed the combination of the fragility assessment. As stated in section 7.2.1, the frame was composed of two systems disposed in series (pier-bearing), resulting in a global parallel system. Thus, considering the symmetry of the global system, a representative global failure was given by the failure of one of the systems.

The combination of the components is proposed in the literature by Choi et al. [368] where a lower and a upper bound of the system fragility for a serial system are given by the following equation:

$$max_{i=1}^{n}[P(F_i)] \leq P(F_{system}) \leq 1 - \prod_{i=1}^{n}[1 - P(F_i)] \tag{7-6}$$

where $P(F_i)$ and $P(F_{system})$ are the probability of component and system failure, respectively. The lower bound corresponds to the completely correlated components while the upper bound assumes no correlation. The real system fragility is located somewhere in the middle of this interval. For example, in the study of Zhang and Huo [369], different importance was given to the components and measured by weights for the final damage state of the system. For this study, the failure probability was assumed by the weakest component in the system. Accordingly, combining the results obtained in Table 7.5 and Table 7.6, the final bridge fragility parameters are presented in Table 7.7 and illustrated in Figure 7-12. Observing the obtained results for the system, it was concluded that, given the DLS thresholds for the different components, the lower quantiles and DLS were controlled by the bearing device, while the higher ones were controlled by the pier. These results gave an important insight into the behavior of the bridge for different DLS and for future maintenance in terms of which components should be more important to do the maintenance activity.

Table 7.7 – Estimated parameters for the system pier-bearing

DLS	5% Quantile		Mean Value		95% Quantile	
	$\hat{\theta}$	$\hat{\beta}$	$\hat{\theta}$	$\hat{\beta}$	$\hat{\theta}$	$\hat{\beta}$
Minor	0.40	0.03	0.60	0.02	0.60	0.02
Moderate	0.55	0.10	0.93	0.10	1.11	0.08
Major	0.81	0.08	1.22	0.08	1.56	0.11
Collapse	1.00	0.08	1.37	0.09	5.71	0.14

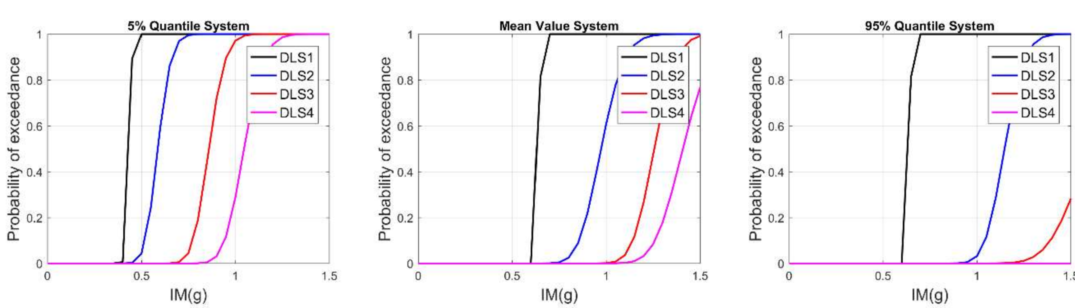


Figure 7-12 – Fragility curves for the system pier-bearing

7.4 Degradation modelling

7.4.1 Corrosion analysis

As discussed in section 4.1, chapter 4, typically the loss of structural strength in aging RC bridges is mainly attributed to the corrosion of reinforcing bars [170]. This study assumed that corrosion lead to a generalized uniform reduction of the reinforcement area along the length of the rebar. However, it is acknowledged that the formation of spatially distributed corrosion pits along the length of the rebar can result in localized weakening of RC piers [170]. This effect was excluded in the present study for simplicity, but it can be incorporated within the proposed methodology together with the modelling of other secondary effects such as loss of bond strength, core and cover concrete strength, and steel yield strength. Also, corrosion on the bearing device was not considered and thus it is out of the scope.

Cover measurement tests performed after the bridge construction revealed that there are zones with a lack of concrete cover, which renders the bridge susceptible to corrosion deterioration [349]. The chloride exposure condition considered in this study corresponded to structures exposed to de-icing salts, whose variables depending on the material and the environment were assumed to be equivalent to the ones valid for the marine splash zone [174].

The probability distributions for the variables to determine T_{corr} and $d_b(t)$ using equations (4.4) and (4.5), respectively, were adopted from Duracrete [174] and Choe et al. [175], see Table 7.8, for the assumed marine splash zone exposure condition, and for assumed Ordinary Portland cement (OPC), water-binder ratio (w/b) of 0.5, and water-cement ratio (w/c) of 0.5. Figure 7-13 shows the time-dependent reinforcement area $A_s(t)$ normalized by the initial reinforcement area A_{s0}, with lower and upper bounds of the uncertainty interval representing 5^{th} and 95^{th} percentile values, respectively. It was evidenced from the figure that chloride-induced corrosion had a significant impact on the reinforcement area, and consequently on the capacity of RC piers to resist seismic demands.

Table 7.8. Probability distributions for the corrosion parameters [174, 175]

Variable	**Condition**	**Distribution**	**Unit**
Model uncertainty coefficient (θ_d)	-	Lognormal (-0.0013;0.05)*	-
Cover depth (x)	-	Lognormal (3.47;0.13)**	[mm]
Environmental factor (k_e)	OPC, Splash zone	Gamma(2.92;11.0)	-
Curing factor (k_c)	At age 7 days	Beta (2.15; 10.7;1.0;4.0)	-
Chloride diffusion coefficient (D_0)	w/c= 0.5	Normal (473.0; 43.2)	[mm^2/year]
Correction factor for test method (k_t)	-	Normal (0.832; 0.024)	-
Reference period (t_0)	28	Deterministic	[days]
Age factor (n)	OPC, Splash zone	Beta (17.2; 29.3; 0; 1.0)	-
Critical chloride concentration (C_{cr})	OPC, w/b=0.5, Splash zone	Normal (0.50; 0.10)	[%] relative to binder
Surface chloride content regression parameter (A_{cs})	Splash zone	Normal (7.76;1.36)	[%] relative to binder
Surface chloride content error term (ε_{cs})	Splash zone	Normal (0;1.11)	[%] relative to binder

* The reported values of the λ and ζ parameters of the Lognormal distribution correspond to a mean model uncertainty coefficient of 1, and a standard deviation of 0.05.

** The reported values of the λ and ζ parameters of the Lognormal distribution correspond to a mean cover depth of 32.6mm, and a standard deviation of 4.2mm.

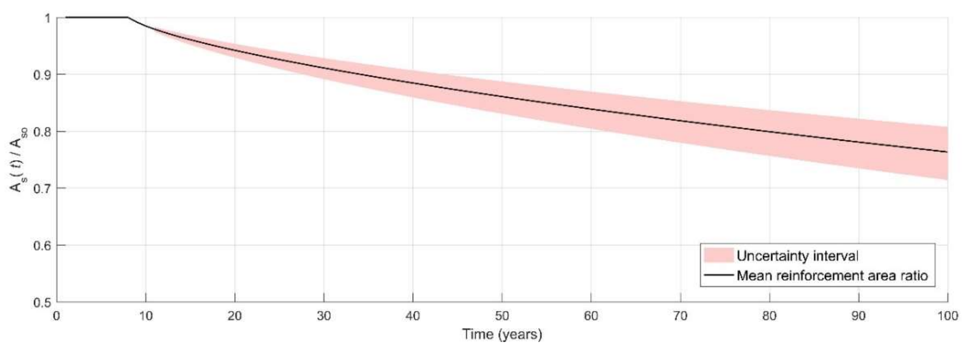


Figure 7-13. Time-dependent mean reinforcement area ratio with lower and upper bounds of 5th and 95th percentile respectively

7.4.2 Corrosion effect on the structure analysis response

The corrosion effect on the structures presents a considerable effect on the EDP and thus on the long-term performance of the structures. In this section, a discussion is made by presenting the effect of the structural response of the structure due to corrosion, by re-computing the static pushover analysis for different timelines and the non-linear time history analysis for a given intensity measure. Note that these results were merely to illustrate the differences observed on the structural responses for a situation with corrosion and non-corrosion and to highlight what is expected to obtain on the time-dependent fragility curves. Normally, the analysis of long-term effects are defined for five different ages, i.e. initial age, 25, 50, 75 and 100 years, see for example [370]. However, considering that the changes for the ages of 25 and 75 years concerning the initial age and 50 years, respectively, were low, it was decided to model the bridge at t=0, 50 and 100 years. Figure 7-14 shows the load-capacity curve for a non-linear static pushover analysis.

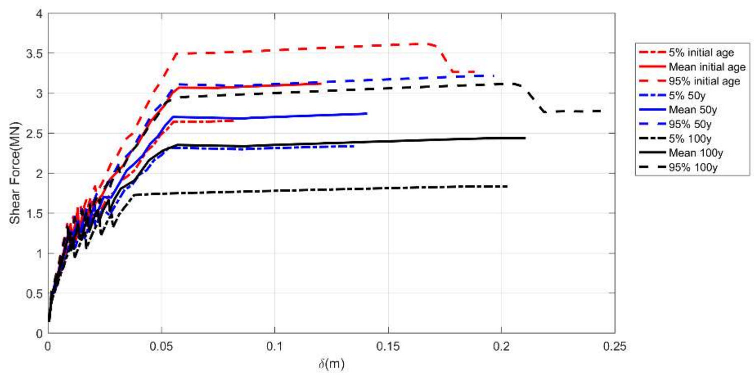


Figure 7-14 – Influence of the corrosion effect on the force-displacement curve

From Figure 7-14 it can be observed that the capacity of the structure was considerable reduced in comparison with the initial age. Moreover, the ultimate displacement slightly increased as expected. Such increase also influenced the fragility curve calculation since the probability of exceedance increased relative to the initial age.

Figure 7-15 shows an example of the force-displacement curves for the three quantiles due to a base excitation corresponding to 1.0g. It was expected that the maximum displacement did not present any relevant change among the different periods since for 1.0g the structure is far from its ultimate limit capacity. This was also due to the high redundancy that concrete structures can provide when compared with other structures as masonry, wherein the re-distribution capacity is reduced. The only relevant change observed was on the maximum shear force in the structure.

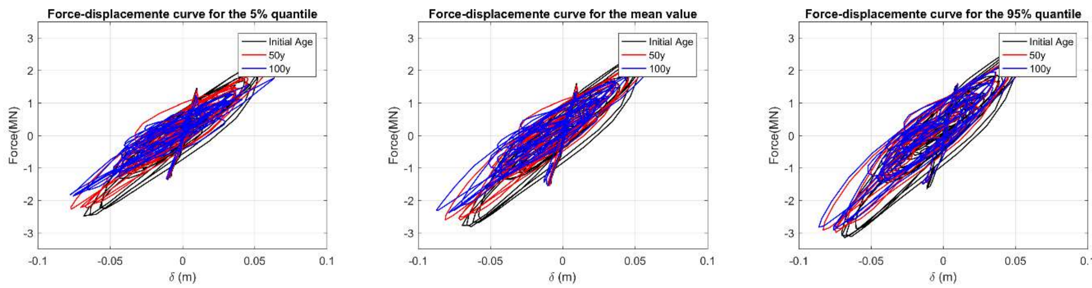


Figure 7-15 – Force displacement response under time history non-linear earthquake analysis

7.4.3 Time-dependent fragility assessment

To understand the effect of corrosion on the fragility curves, the process described in section 4.4 was repeated for different ages. It is important to mention here that the process of obtaining the fragility curves for each period is highly time-consuming thus, the same periods of time were adopted as mentioned in the previous section.

Moreover, to understand the influence of the corrosion on the fragility curves, a comparison of the new obtained displacements was made with the DLS thresholds defined in Table 7.7 for the initial age. The estimated parameters for 50 and 100 years of corrosion for the system are expressed in Table 7.9 as well as the resulting fragility curves in Figure 7-16. It was observed from the fragility curves that the probability of exceedance (PoE) was higher for the same intensity measure as the age increased.

Table 7.9 – Estimated parameters of the system for 50 and 100 years

DLS	5% Quantile		Mean Value		95% Quantile	
	$\hat{\theta}$	$\hat{\beta}$	$\hat{\theta}$	$\hat{\beta}$	$\hat{\theta}$	$\hat{\beta}$
	50 years					
Minor	0.38	0.04	0.60	0.02	0.60	0.02
Moderate	0.51	0.11	0.88	0.06	1.02	0.08
Major	0.75	0.06	1.12	0.08	1.46	0.08
Collapse	0.92	0.09	1.32	0.11	1.52	0.01
DLS	**5% Quantile**		**Mean Value**		**95% Quantile**	
	$\hat{\theta}$	$\hat{\beta}$	$\hat{\theta}$	$\hat{\beta}$	$\hat{\theta}$	$\hat{\beta}$
	100 years					
Minor	0.31	0.03	0.50	0.11	0.50	0.02
Moderate	0.48	0.14	0.85	0.07	1.00	0.09
Major	0.70	0.02	1.03	0.12	1.41	0.10
Collapse	0.90	0.08	1.20	0.18	1.51	0.01

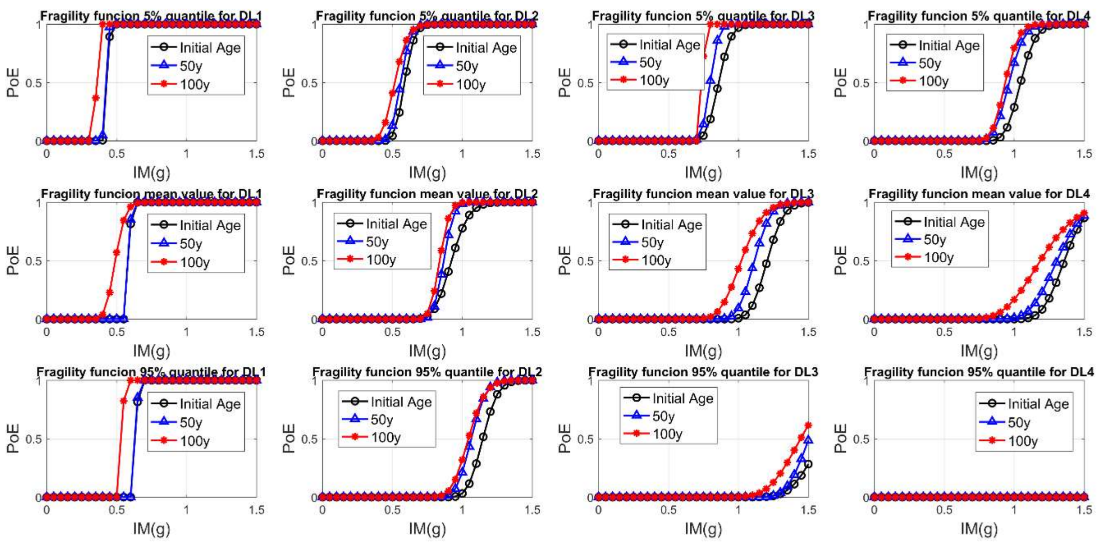


Figure 7-16 – Time-dependent fragility curves for 5% quantile, mean and 95% quantile

7.5 Life cycle risk assessment

This section aims to combine the obtained time-dependent fragility curves with risk analysis and investigate its behaviour over the life cycle. As such, and following the methodology, the first part concerns the time-dependent risk assessment while the second one concerns maintenance and optimization wherein some optimal maintenance scenarios are thus discussed and compared.

7.5.1 Risk assessment

The analysis of risk for bridges follow the equation (3.9), chapter 3. Though, given the case study under analysis, the expression can be accordingly re-written as:

$$Risk = p(DLS_i|IM) \times C \tag{7-7}$$

where $p(DLS_i|IM)$ is the probability of exceedance for a certain DLS given the IM, and C are the consequences of the system. Although the simplicity of the expression, there are several variables included in the calculation of the probabilities of exceedance, see previous sections, and the consequences.

Consequences are a very sensitive issue as they change accordingly with the location of the bridge. As stated in section 3.2.2, chapter 3, consequences can be of direct or indirect nature. The direct consequences were related to direct interventions on the system while indirect consequences retrieved issues related to vehicles delay, detour routes, injuries, and loss of human lives. According to Table 7.10, the definition of the DLS accounts for the description of the state and the required interventions.

Table 7.10 – Description of the DLS [364]

Damage Limit State	**Required Interventions/Consequences**
DLS1: Minor	Inspect, Adjust, Patch
DLS2: Moderate	Repair Components
DLS3: Major	Rebuild Components
DLS4: Collapse	Rebuild Bridge

Bearing this in mind, the assumptions for this case study regarding direct consequences calculation were based on the study of [118] given by the equation (3.16). Moreover, the author claimed that rehabilitation should be estimated at 60% of the rebuilding costs and thus the consequence estimation for the DLS2 [371]. The DLS3 and DLS4, as the interventions indicate, were related to the rebuilding of the damaged elements, i.e. pier and bearing device, and rebuilding of the system, i.e. the entire frame considered for the analysis, respectively. Regarding DLS1, as the damages on the elements are very low as well as the estimated consequences, its analysis was disregarded for the risk analysis.

Concerning the indirect consequences, alternative detour routes were identified due to the closure of the bridge for repair. Being the railway network not as redundant as the roadway network, alternative routes were defined through roads. Its calculation was given by equation (3.17). Due to the unavailability of information regarding possible fatalities estimation as well as injuries, such consequences were out of the scope of this work. Table 7.11 shows the quantification of the considered variables.

Table 7.11 – Quantification of the consequences [149, 240]

Variable	Notation	Quantification		
Rebuilding cost (€/m²)	c_{reb}	TD* (680,1360,2550)		
Rebuilding Area (m²)	A_{pier}	235.0**		
	A_{deck}	108.0**		
Conditioned traffic percentage	PER	TD (80%-90%-100%)		
		Cars	Trucks	
Average daily traffic	TMD	950	50	
Cost per kilometer (€/km)	CK	0.18	0.68	
Cost per hour (€/h)	CH	8.4	10.1	
Restricted Speed (km/h)	S_r	70**	50**	
		Train		
Normal Speed (km/h)	S_n	200**		
Detour route (km)	LD	5.15**		
Normal route (km)	LP	7.50**		
Discount rate	R	2%		
Duration of the activity (days)	DUR	DLS2	DLS3	DLS4
		3	180	270

*TD stands for the triangular distribution

**values directly estimated from the bridge drawings

The estimation of the time-dependent consequences in terms of monetary losses must consider future risk as estimated in section 4.9.1, chapter 4, according with equation (4.22). Considering all the defined variables, the obtained consequences are assembled in Figure 7-17 for the different DLS. Note that the consequences were herein defined as probabilistic. Therefore, the consequences values are assigned to the 5% quantile, mean and 95% quantile covering the best, average and worst scenarios, respectively.

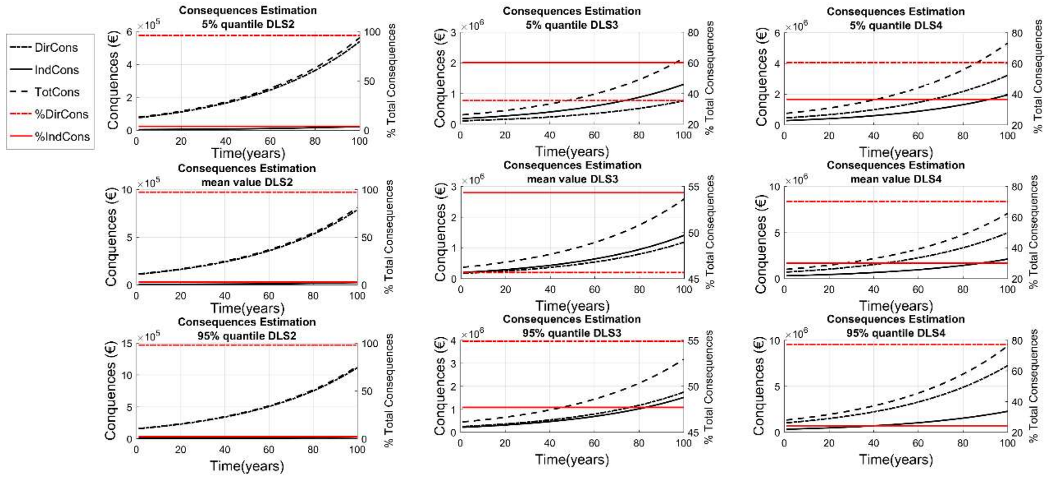


Figure 7-17 – Estimated consequences

Observing the consequences for DLS2, the indirect ones were very low due to the low restoration time of 3 days. On the other way, for the DLS3 indirect consequences, due to the high period of recovery, they represented around 45-60% of the total consequences, which was already significant for further analysis. As for DLS4, although the recovery time was the highest, the indirect consequences only represented around 25-40% of the total consequences as the direct consequences involved the rebuilding of the entire system.

Combining the time-dependent consequences and the fragility assessment, Figure 7-18 illustrates the risk-based fragility curves. Observing the obtained curves, it was noticed that for lower values of PGA, the risk values were very low presenting thus no threat to the performance of the bridge. However, as the IM increased, the risk started to present a serious threat to the bridge indicating that some mitigation and maintenance activities should be embraced to decrease the risk values. The definition of the risk acceptance criterion level, wherein some precautions should be taken, is a very subjective issue once there are no specific guidelines to establish some risk thresholds. Thus, for the present case study, the acceptance criterion level was estimated as 10% of the value of reconstruction of the bridge in its initial year, wherein values above that value will be further considered for maintenance activities. Reports from the present bridge stated a construction value of 3607509 €.

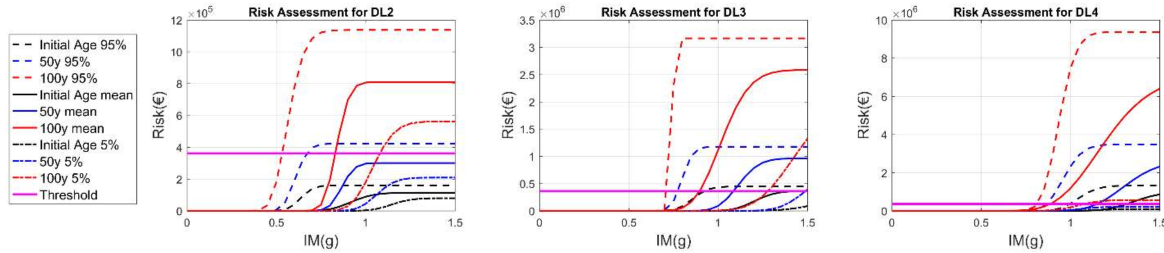


Figure 7-18 – Risk Assessment for the different DLS

Figure 7-19 shows the time-dependent curves given the PGA level of intensity that crosses the acceptance criterion level and thus considered for maintenance activities. Note that for the quantile 95% of the DLS4, no level of PGA crossed the estimated criterion, therefore no plot was needed to evaluate the time-dependent consequences once they are below the threshold value. The next section is dedicated to assessing mitigation actions to investigate how much these risk values decrease. Moreover, a comparison will be made with the obtained values with no mitigation action and a thorough discussion on whether applying the mitigation action is worthwhile.

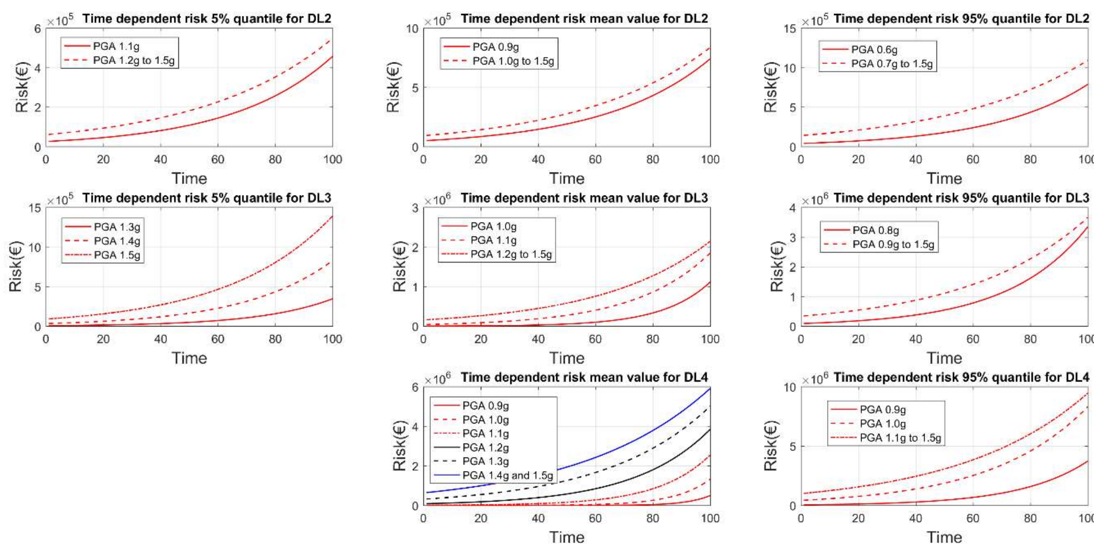


Figure 7-19 – Time-dependent risk quantification

7.5.2 Mitigation actions

The current section is dedicated on decreasing the risk level exposure of the bridge. Such mitigation actions can be implemented through retrofit activities. Retrofit activities have proved by the literature to be an effective response to mitigate the risk to seismic hazard scenarios, see [117, 372]. Moreover, these authors provided a valid list concerning the application of retrofit activities at various levels aiming to reduce the seismic impact such as jacketing, isolation of the bearing devices, among others. For the present case study, steel jacketing retrofit was herein applied aiming to validate the methodology for a further comprehensive discussion on its influence on maintenance and optimal scheduling. The application of steel jacketing retrofit action followed the recommendations of [372] by adding an increase of 15% to the side length of the column and an additional 50% of the longitudinal rebar. Those changes were applied directly to the model presented in section 4.2 and all the previous steps are re-calculated considering the jacketing retrofit measure. Figure 7-20 and Figure 7-21 show the pushover and the force-displacement curves, respectively. Note that the force-displacement curves were applied considering the same conditions as depicted in Figure 7-14. From Figure 7-20, it was observed a higher shear force on the yielding phase and a lower displacement obtained. Such result was expected due to the steel jacketing.

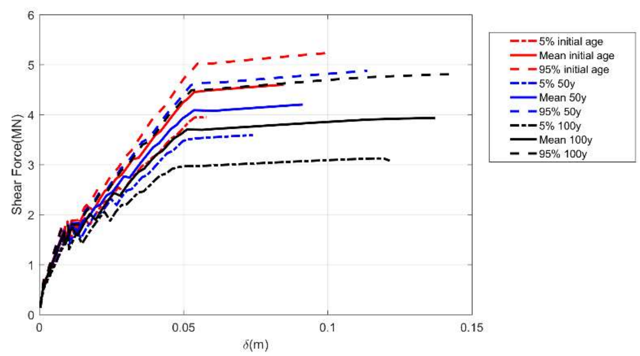


Figure 7-20 – Influence of the corrosion on the force-displacement curve considering the retrofit action

As for the force-displacement curve, it was also observed an increase in the peak force. However, the differences were not as relevant as in the pushover curves since for 1.0g intensity measure, the structure was still far from collapse due to its redundancy.

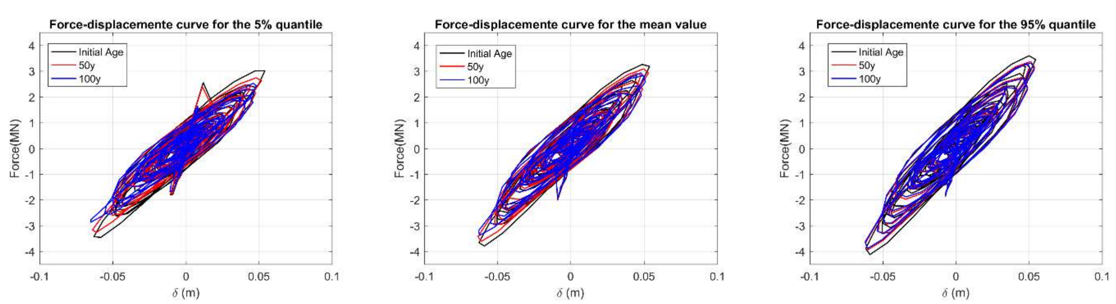


Figure 7-21 - Force displacement response under time history non-linear earthquake analysis considering the retrofit action

Fragility assessment was also conducted following the formulation expressed in section 4.3.4. The results are depicted in Figure 7-22 indicating the decrease in the probability of exceedance due to the steel jacketing. The obtained risk values are assembled in Figure 7-23.

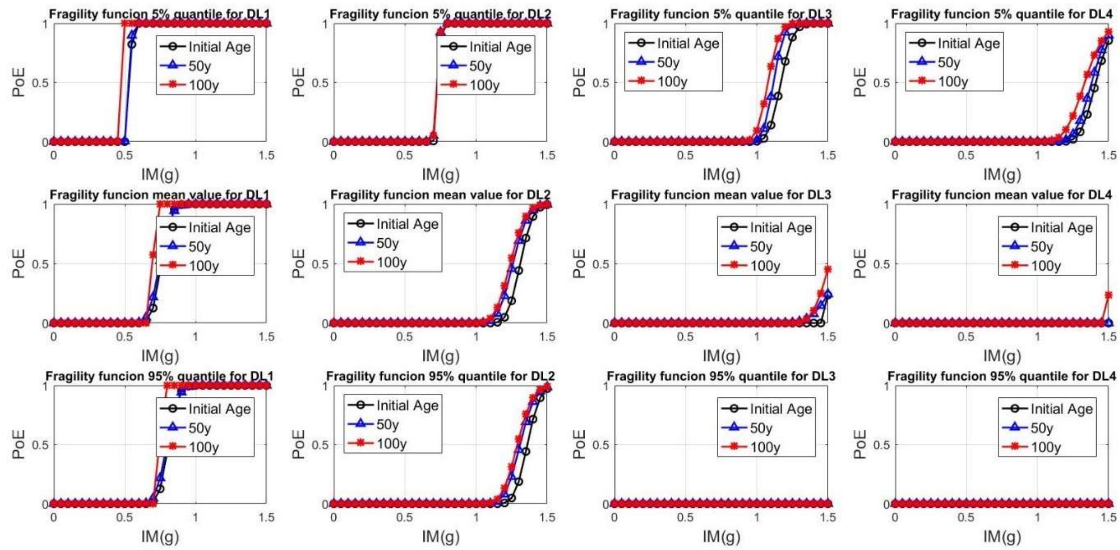


Figure 7-22 – Time-dependent fragility curves considering the retrofit action

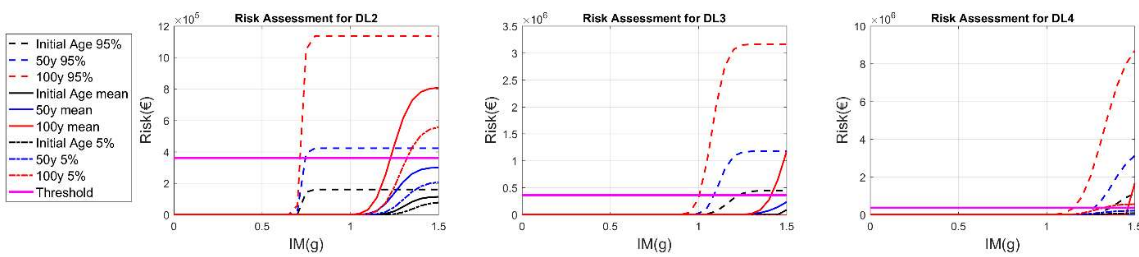


Figure 7-23 – Risk Assessment considering the retrofit action

Figure 7-24 presents the time-dependent intensity curves for the risk values that cross the threshold above defined. Although a retrofit action improved considerably the performance of the bridge by lowering the risk values, yet it was verified high risk values. The following section aimed to introduce the analysis of the maintenance actions on these curves. Those maintenance actions were implemented through an optimization algorithm to minimize costs by keeping the risk values as low as possible. Furthermore, analogously to the present section, a discussion between maintenance actions with and without the retrofit actions were hereafter discussed.

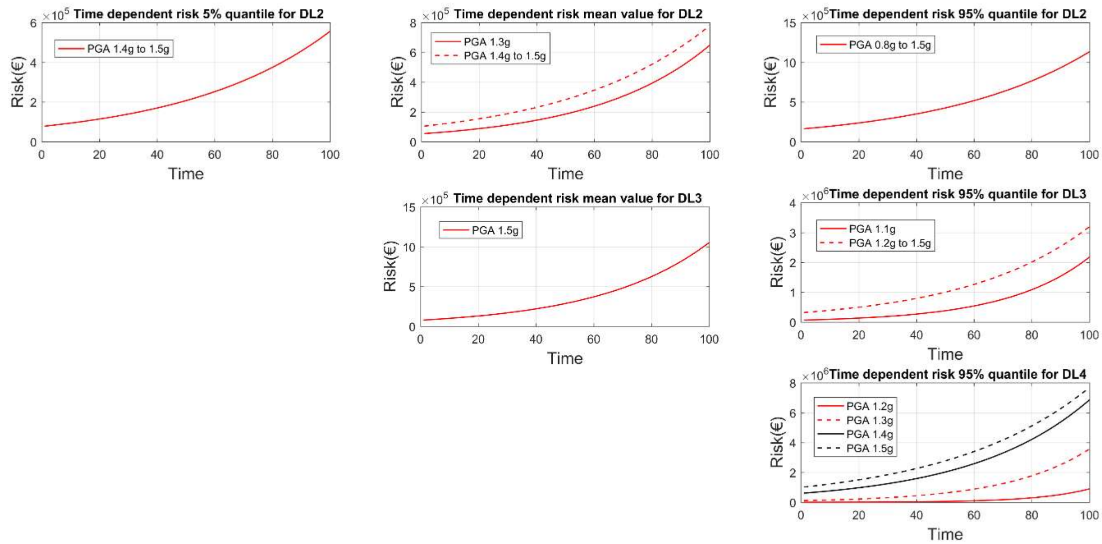


Figure 7-24 – Time-dependent risk quantification considering the retrofit actions

7.5.3 Maintenance and optimization

To preserve the integrity of the bridge over its life cycle, regular maintenance activities should be embraced. Such activities are established based on their effects on the structure and the cost of its maintenance. Regarding their effects, they can be quantified accordingly to their reduction in the rate of degradation as suggested elsewhere [196, 373]. However, their quantification is uncertain because it is strongly dependent on the experience of the project manager. For this study, the maintenance actions and their effects were provided considering the experience of major Portuguese infrastructure management companies and by a comprehensive questionnaire to different stakeholders [12]. Table 7.12 provides a comprehensive list of actions proposed to this case study divided by their impact on the structure as well as the associated direct costs. As for the indirect costs, the case study followed the equations detailed section 5.2, cahpter 5 wherein its parameters quantification are depicted Table 7.13.

Table 7.12 – Actions effects and corresponding costs

Damage State	Type of maintenance	Effect of the maintenance	Quantification		Cost
DLS2	Cleaning of the piers	Keep the deterioration rate in the same level	Time of reduction (t_r): TD* (1,1.5,2) (in years)	Rate of reduction (δ): TD* (30%,40%,50%)	$3€/m^2$
	General repair of pier (e.g. concrete cover)	Keep the deterioration rate in the same level	Time of delay (t_d): TD* (1.5,2,3) (in years)		$50€/m^2$
	Cleaning of bearing devices	Keep the deterioration rate in the same level	Time of delay (t_d): TD* (0.5,1,2) (in years)		$75€/unit$
DLS3 and DLS4	Repair on the bearing devices	Slow the deterioration rate	Time of reduction (t_r): TD* (2,3,4) (in years)	Rate of reduction (δ): TD* (40%,60%,75%)	$120€/unit$
	Crack repair on the piers	Slow the deterioration rate	Time of reduction (t_r): TD* (0.5,1.5,3) (in years)	Rate of reduction (δ): TD* (75%,90%,100%)	$126€/m$
	Anti-corrosive paint on the piers	Keep the deterioration rate in the same level	TD* (4,5,8) (in years) Time of delay (t_d)		$76€/m^2$
	Complete Repair pier	Improve the condition of the structure	Improve on the condition state		$2000€/m^3$

Table 7.13 – Indirect costs quantification [149]

Parameters	Quantification	
DUR	5 days	
l_r (m)	60	
Type of train	Medium to long trip trains	
VA_v (€/min)	2.40	
S_r (km/h)	30	
S_n (km/h)	200	
TMD	150	50
PER^*	TD (10%-40%-70%)	

In Table 7.13, l_r is the length of the rail track (km), l_t, S_r the restrained speed (km/h) and S_n the normal speed (km/h); TMD is the average daily traffic for railways and PER is the availability of the bridge related to each intervention.

As for the optimization procedure, following the algorithm explained in Figure 5-3, the MOP was defined by minimizing the life-cycle risk by considering the lowest possible maintenance costs. For the Genetic Algorithm (GA), NSGA–II, through the *gamultiobj* built-in function of MATLAB [316], was adopted with a population size of 500 alongside with 500 generations, a crossover scattered function used to create crossover children and a tolerance criterion of 1e-6. Accordingly, the MOP was stated as:

Find:

- The optimal maintenance schedule, i.e. the decision-making vector (DMV)

To achieve the following conflicting objectives:

- Minimize the overall risk
- Minimize the direct costs

Subject to the constraints:

- $t_i - t_j \geq 2\ years$, i.e the interval between two applications shall not be lower than 2 years.

Two representative optimal solutions were chosen. Solution 1 represented the best-case scenario in terms of minimization of the risk, but more maintenance costs are involved, while solution 2 corresponded to the worst scenario in terms of risk values. Figure 7-25 shows the effect of the maintenance actions for solutions 1 and 2. Note that the obtained results considered the effects of maintenance actions without any retrofit action. From the obtained results, a considerable decrease in the risk values was observed.

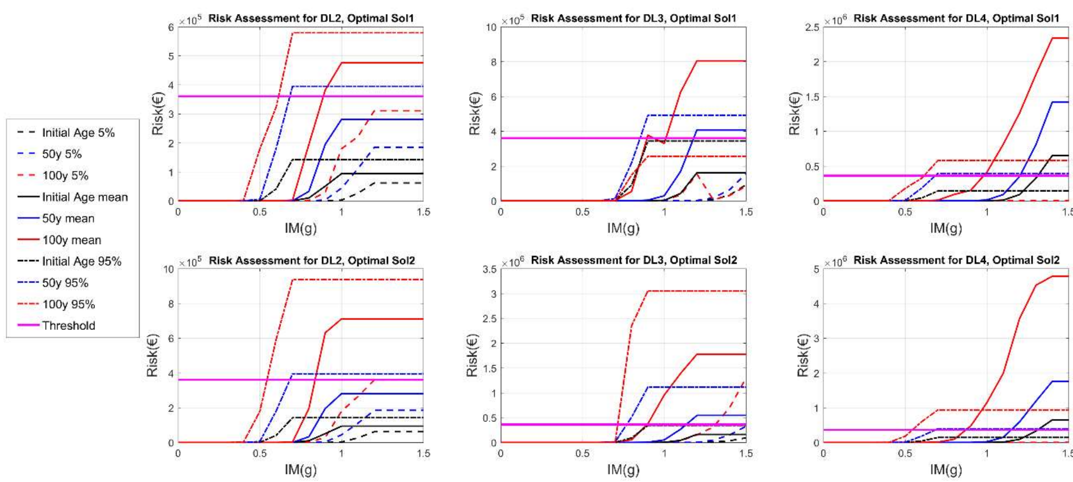


Figure 7-25 – Risk Assessment for optimal Solutions 1 and 2 without retrofit action

It can be noticed that the threshold value was still crossed. Such a scenario might represent a major problem to the owners of the bridge while assessing the optimal solutions. In this way, it was concluded that maintenance solutions might be insufficient. Nevertheless, the combination of the maintenance activities with the mitigation action previously defined could be a wise solution to solve this problem. Figure 7-26 shows the optimal solutions considering the maintenance actions and the retrofit action denoting that the problem of having risk values crossing the threshold value is practically surpassed for solution 1. However, solution 2 still presented values over the threshold value. Such a problem could be solved by adding more restrictions. Yet, this measure could lead to a mathematical problem with the GA by not attaining to obtain optimal solutions at all. Another option could be to establish a maintenance schedule combined with the mitigation action. Note that, by adopting this option, the owners/project managers could be neglecting the optimal solutions and selecting more expensive solutions. More detailed calculations, in terms of optimal solutions per each intensity over the time given by the Pareto front for both maintenance actions, with and without retrofit actions, are described in the annex C1 and C2.

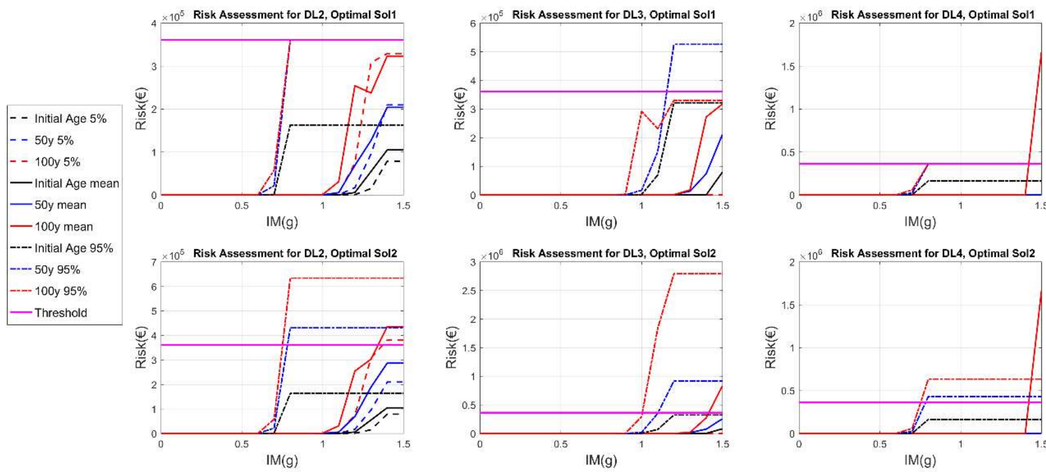


Figure 7-26 – Risk Assessment for optimal solutions 1 and 2 considering retrofit action

7.5.4 Benefit Analysis

The present section aimed to analyse and compare the different proposed solutions to decrease the risk values by comparing the benefit gain with the initial risk, see Figure 7-18. The adopted metric for the benefit calculation is given as follows:

$$Benefit_i(\%) = \left(1 - \frac{\int_{IM=0}^{IM=1.5g} Fragility\ Curve(IM)_{scenario\ i}}{\int_{IM=0}^{IM=1.5g} Fragility\ Curve(IM)_{initial\ risk}}\right) \times 100 \qquad (7\text{-}8)$$

Alongside the benefit calculation, the total cost of the solutions was also expressed for further comparisons. The following figures express the benefit for three scenarios: (1) No maintenance with retrofit action, see Figure 7-27; (2) maintenance without retrofit action, see Figure 7-28; (3) maintenance with retrofit actions, see Figure 7-29. Note that the value assumed for the cost of applying the steel jacketing retrofit action was based on the study of [117], where a cost of 5100€ was estimated to retrofit each column.

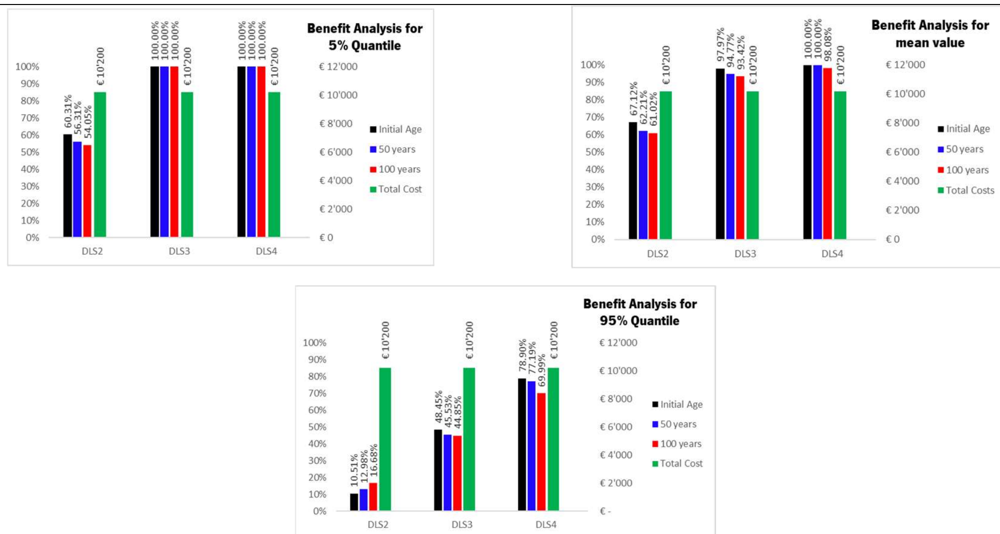


Figure 7-27 – Benefit Analysis considering only retrofit actions

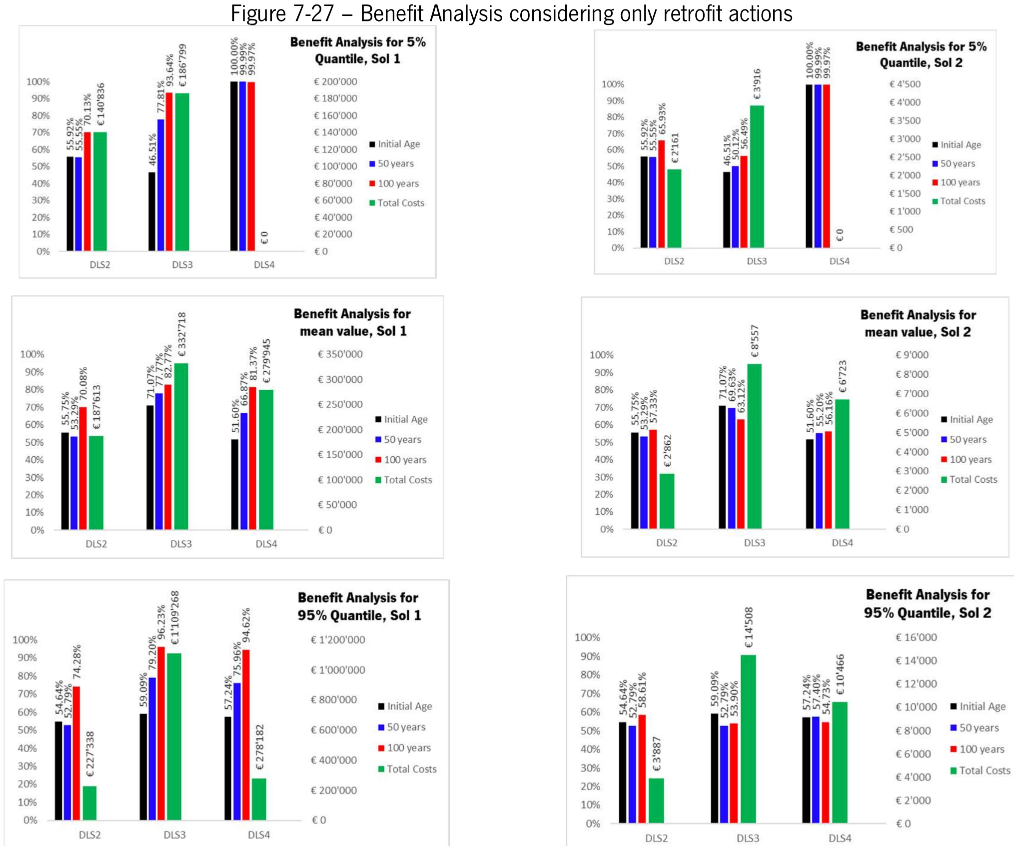


Figure 7-28 – Benefit Analysis considering maintenance actions without retrofit action

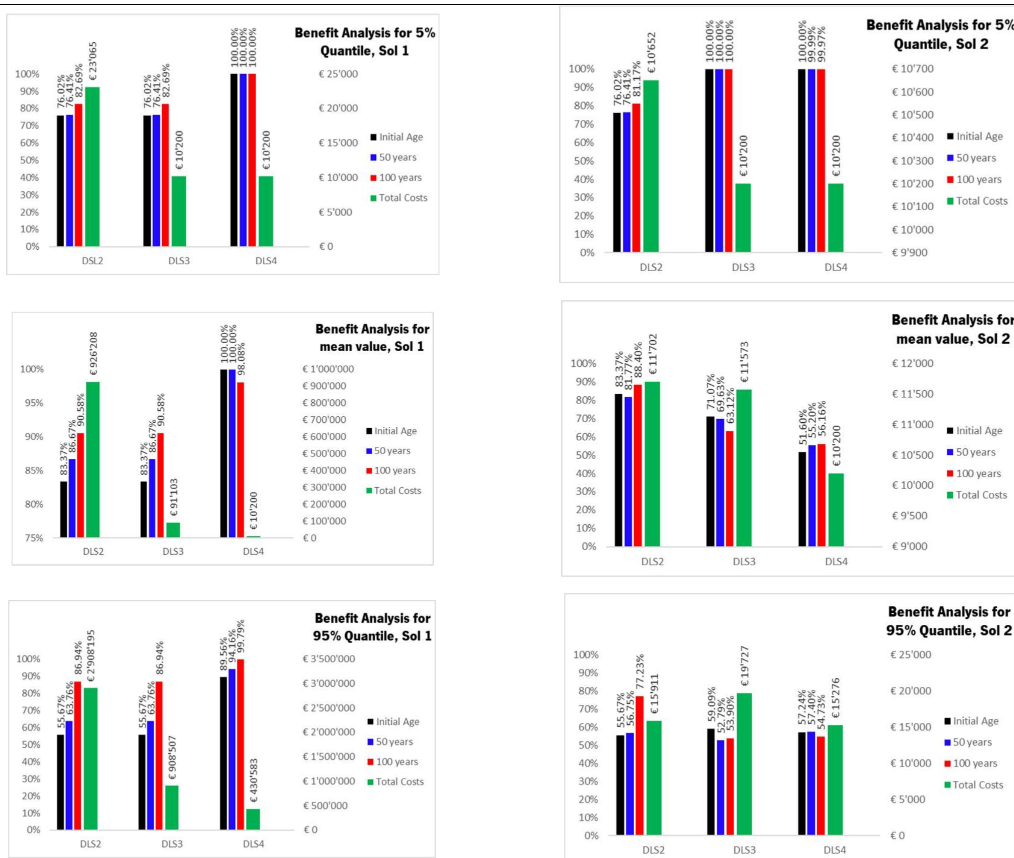


Figure 7-29 – Benefit Analysis considering maintenance and retrofit action

For scenario 1, it was observed that the lowest benefit was attributed to DLS 2. Such fact was given by the threshold limits previously defined in Table 7.3, being easily crossed for both retrofit and no retrofit options. As for the DLS 3 and DLS4, the benefit was higher once the PoE on the fragility curves considerably decreased. Indeed, there was a high benefit for solution 2 for lower costs than solution 1. However, such a metric should not be assessed only by itself once the values were still crossing the threshold value for some intensity measures, see Figure 7-25 and Figure 7-26. Therefore, such analysis should be complemented by careful analyses of the time-dependent risk curve per IM.

7.6 Resilience estimation

Following the risk assessment, resilience estimation was conveniently addressed to estimate the system recovery by establishing recovery plans according to the IM. The following section analysed the recovery of the system for the original solution without maintenance actions, as well as considering the optimal solutions and the retrofit action.

7.6.1 Recovery function

The recovery functions for this study adopted the same recovery functions defined in the chapter 6. The parameters as well as the recovery functions were considered based on the Table 6.3 and Figure 6-7, respectively. Those functions were based on the rapidity of recovery given the DLS, i.e. a lower DLS allowed a faster recovery. Likewise, the critical over-damped recovery function was adapted for the DLS2 and DLS3 and a lognormal cumulative function for DLS4, see equations in Table 6.2.

Considering the recovery functions for each DLS and the equation 6.1 to estimate the resilience, the overall resilience for the solutions in section 4.5 was calculated given the IM. The results are depicted in Figure 7-30 and Figure 7-31 for the situation without and with the retrofit actions, respectively. Note that the results of the resilience per each IM were normalized to the worst-case scenario per DLS, i.e. the maximum risk value obtained. Likewise the obtained risk in section 4.5, the resilience tended to be worse for DLS3 and DLS4 as long as the IM increases. For the DLS2, the resilience was almost 100% for all the IM once the recovery time is very short. Although the resilience values were considerably high, except for the DLS4 wherein the worst resilience level was around 40%, risk values mitigation should be always considered and analysed to avoid a post-event recovery and higher losses than those estimated just for maintenance scenarios. Analogously to the risk calculation, the application of the maintenance solutions along with the retrofit actions improved considerably the resilience, especially in solution 1. The time-dependent resilience for each IM can be consulted in Annex C3.

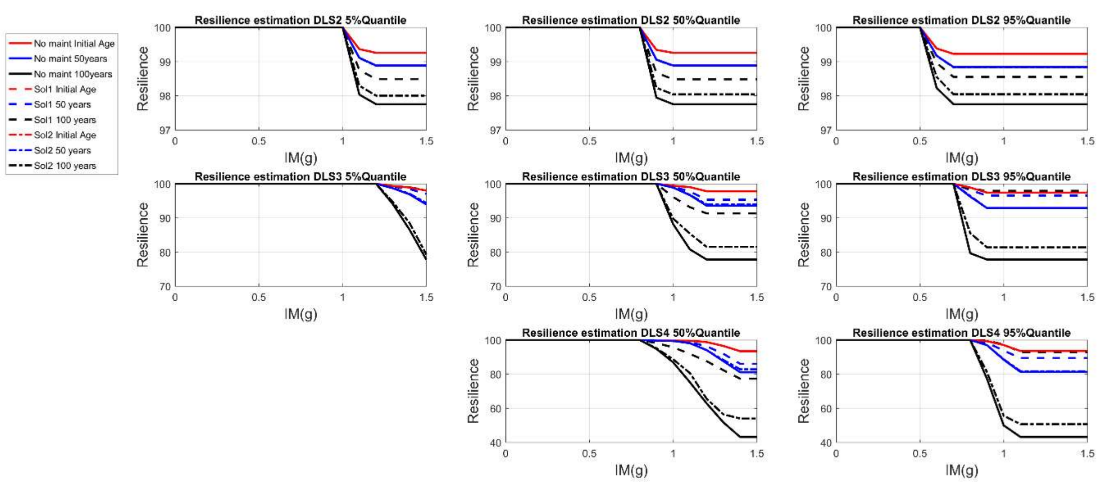


Figure 7-30 – Resilience Estimation without retrofit actions with and without maintenance

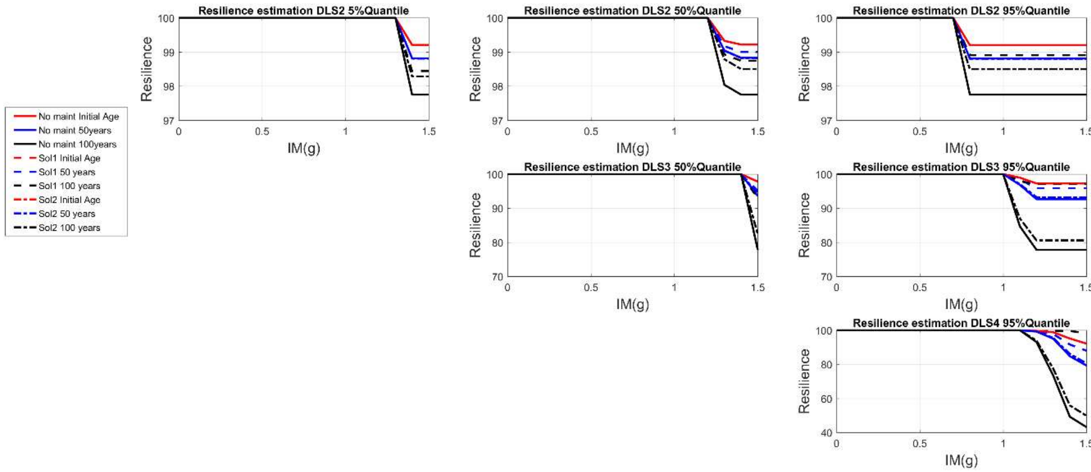


Figure 7-31 – Resilience Estimation with retrofit actions with and without maintenance

7.7 Final remarks

This chapter outlined the methodology of the thesis by combining the risk and resilience for the process of decision-making and by proposing some practices in the field of reinforced concrete railway bridges subjected to an extreme event (earthquake) combined with a progressive damage event (corrosion). The non-linear analyses accounted with a probabilistic assessment by considering a set of RV on the problem. Since the probabilistic analysis enhances a high computational time, three deterministic models were considered representative of the analysis. Furthermore, a non-linear time history analysis was posteriorly embraced to obtain the fragility curves for the pier-bearing system. In order to consider the progressive damage of the bridge over time, a corrosion model was established and applied to obtain time-dependent fragility curves for the ages of 50 and 100 years. Moreover, the risk assessment was calculated by combining the obtained consequences with the fragility curves. By establishing a threshold value, it was concluded that the values were unacceptable for some intensities leading to embrace mitigation and maintenance activities. The mitigation action accounted with the steel jacketing of the columns and it has shown to be a very effective measure to decrease the risk. Although some IM measures still crossed the threshold value, the benefit analysis has shown that it could be a good option alone without considering maintenance activities. However, some precautions should be still considered. Moreover, by establishing maintenance scenarios through MOP, it was possible to obtain even lower risk values, yet with an extra cost leading to the conclusion that a combination of maintenance activities with the retrofit action might be the best option regardless of the costs of maintenance involved.

To finalize, a resilience estimation based on the risk levels was carried out to understand the post-event recovery of the system in case of happening the event. The resilience values showed to be very high for the DLS2 since the recovery time proposed was very fast (3 days). On the other hand, for the DLS4 the worst values have shown to raise some warning given that the estimated resilience was of about 40%.

Chapter 8

8 Conclusions and future works

8.1 Conclusions

This thesis focused on the proposal of an infrastructure management system. Although its validation throughout the thesis was on bridges, the current methodology is broadly applied to other types of infrastructures. Slight changes would be needed on the conceptualization of the problem regarding the type of key performance indicators (KPIs) and on the limit state analysis in case of a quantitative analysis. These problems related to management systems are a very complex project since it is composed of many tasks to guarantee its fully functionality. Hence, it becomes fundamental at the first stage to structure the problem and breakdown into small pieces to fully tackle the whole project. In this thesis, the way to tackle the problem relied on the division of the problem into four modules. Furthermore, after a brief introduction of the problem regarding the management of infrastructures, the chapter 2 aimed to conduct a review on the most relevant projects among infrastructure management systems. From the literature review, it was concluded that the field with most applications of civil engineering were concerned to bridges and pavements. However, by consulting the existing research projects described from 1998-2020, it was verified that those applications were already being extended to other types of facilities such as buildings, tunnels, retaining walls, among others. Concerning the bridge management system (BMS) field, a deeper investigation concluded that there were currently several BMS implemented worldwide, accordingly with IABMAS report. The major drawback observed was that each one of these BMS followed their own standards by conducting their management systems with different KPIs. To overcome this drawback, recently, a research project on the field of bridges, denominated COST TU1406, was developed with the goal of proposing a standardized KPI of assessment of existing bridges.

The chapter 3 aimed to discuss issues related to the assessment of existing structures, with a deeper focus on the bridge case. Currently there are six levels of structural assessment wherein the first level concerns the assessment by visual inspections, while the most complete relates to probabilistic assessment. One of the first steps for the assessment concerns the establishment of the KPIs. Those KPIs can be of qualitative or quantitative nature. The former is normally measured by a scale composed of integers values from 1 to 5 wherein, generally, 5 is the worst and 1 the best condition state. The qualitative KPIs are considered very handy for situations concerned to visual inspections and expert judgment. For a more comprehensive and analytical analysis, the quantitative indicators suit better. In this thesis, three different types of KPIs were analyzed, reliability, risk, and robustness. Those KPIs were discussed in terms of formulation and applied into three different cases. For all these cases, the failure mode investigated concerned the load-carrying capacity. The obtained results of the KPIs revealed that in all cases the structures were considered safe with very low risk levels involved. Moreover, a risk-based robustness index was estimated. From this KPI it could be observed the dependency of the indirect consequences. In this way, it was concluded that for a complete and reliable measure of the bridge, the best KPIs to assess were either the reliability or the risk.

In a management system, the degradation model has the role of simulate the long-term damages expected in order to capture the processes of degradation. Chapter 4 was dedicated on discussing the process related to the degradation of structures. From the literature review, it was concluded that there are several types of models adopted on modelling the degradation wherein the Markov models were considered the most popular despite on their limitations. Nevertheless, other types of models have been emerging among civil engineering field such as the Petri-nets and the artificial intelligence models. Although, those models only work well if a considerable amount of data is provided. Otherwise, the models could give problems related to very high bias values. In case that there is not enough data to model the structures, several models have been proposed in the literature to model the degradation of structures based on mechanistic models such as corrosion of the concrete. For practical applications, two different types of models were adopted, the mechanistic models, considering the effects of the corrosion, and the stochastic models, considering the Markov approach based on inspection reports. Concerning the mechanistic models, although it was observed in the models that the threshold value was crossed at around the age of 30, the risk levels started to be serious after the 50 years of age. Considering the Markov model, the results were highly dependent on a database. The more robust is the database, the better the results are. For these applications, the structures started to present poor levels regarding the condition state after 20/30 years of age. Because these two models adopt different KPIs on their modelling, it was not possible to compare those two models directly. However, it might be useful to understand how these two types of models can be complemented. The Markov theory is very practical to be applied and fits very well on qualitative analysis. But, because they are based on similar records of other bridges, such information by itself might not be totally reliable to assess the actual condition. In that way, understanding the condition of the structure by performing non-destructive tests, assessing the levels of corrosion and developing mechanistic models, it was possible to have a more accurate information regarding its true condition state.

Because maintenance actions are important to control the degradation of structures, chapter 5 aimed to discuss the issues related to the life cycle analysis as well as the optimization problems. It was investigated that the life cycle costs were divided into three fields: agency costs, user costs and environmental costs. Furthermore, because there are limitations on the budget, the optimization problem was discussed. The literature has shown several optimization techniques. However, a multi-objective optimization problem based on genetic algorithms revealed to fit better for this thesis. Those concepts were applied into two different applications, a single bridge and a network composed of three bridges. For the case of the single bridge, the optimization was considered with two conflictive objectives, reliability and cost of maintenance. Moreover, the optimization was also considered as risk and cost of maintenance as objective functions. The results have shown that the costs of maintenance considering reliability and risk were practically the same. Therefore, since the optimization considering the reliability gives more conservative results in terms of performance, the reliability should be considered instead. The purpose of showing the application for the network was to highlight the differences on the involved costs while considering more than one bridge in the optimization process.

The chapter 6 introduced the problems related to the exposure of the bridges to hazards. The literature review on hazard analysis concluded that the most common bridge failures are related to collision of vehicles and flood events. Furthermore, an overview on the post-recovery analysis was conducted by investigating the adopted recovery models in case of a hazard occurrence. Those recovery models were given by an indicator called resilience that measures the capacity of the system to return to its original state. Those concepts were applied in same cases as the chapter 5. For the case of a single bridge, a bridge strike hazard event was investigated wherein it was modelled a possible bridge damage. Thus, a resilience estimation was considered for three different times of recovery. The difference on the time of recovery affected the value of resilience as well as the indirect costs involved on the recovery of the system. Moreover, the hazard event was included in the optimization problem in order to investigate how the maintenance schedule could be affected by introducing a hazard. The inclusion of the hazard significantly affected the optimization results since less maintenance activities were considered, due to the costs spent on the recovery of the system after the hazard occurrence. Considering the network analysis, similar conclusions were drawn regarding the effect of the hazard on the bridges. Differences relied on the amount of the costs.

Those concepts discussed in the chapters 3, 4, 5 and 6, were applied in a case study of a reinforced concrete railway bridge in chapter 7. Since an earthquake hazard scenario was considered in this case study, the framework was slightly adapted to consider that scenario in the analysis. The calculation of the performance indicators, described in the case study as the engineering demanding parameters, were obtained by developing a finite element model using the software TNO DIANA. For the first module, the goal was to obtain the performance of the bridge through a fragility analysis. Furthermore, a time dependent analysis was performed by considering the effect of the corrosion to obtain the time-dependent fragility curves. This step was important to obtain the probability of exceedance of a given intensity measure and combine with the consequence estimations to obtain the KPI of risk. For the risk analysis, three different scenarios were considered based on the expected damage on the structure. The next step combined the module 2 and 3 to apply the concepts of maintenance and optimization. Since maintenance activities were not enough to keep low levels of risk, a mitigation action concerning the jacketing of the piers was applied to investigate how this mitigation would affect the fragility assessment. The results have shown a considerable decrease on the probability of exceedance on the fragility curves and thus a lower value of risk. To finish the case study, a resilience estimation was made based on the risk values obtained. The obtained results showed very high resilience for scenarios wherein it was verified lower damages and a low resilience for scenarios with higher damages involved. In the overall analysis, the results obtained by the optimization as well as by the inclusion of the mitigation activities revealed to be consistent and an important step to validate the proposed methodology.

8.2 Future works

The developed methodology covered aspects concerned to the establishment of the KPIs and the decision-making process stage. Although the main objectives were fulfiled, there are still some aspects that should be considered for future developments: Thus, future works are listed as follows:

1. The first suggestion for the future development goes to the integration of the methodology to be ready to be used by the stakeholders. Hence, a software with a front-end could be carried out. For example, the development of a Graphical User Interface (GUI) covering this methodology could be interesting to therefore provide a friendly interface for the users.
2. These types of methodologies for management systems are highly dependent on the information provided. For example, for the degradation model, the stochastic models are highly dependent on the historical database. If a database with a very few data is provided, the degradation model might present problems in terms of accuracy. Hence, stronger databases should be considered other than just based on the condition state and date of inspection. Database with more features such as type of traffic loads, dimensions, presence of cracks, among others would allow to build more powerful models such as artificial intelligence.
3. On the life-cycle analysis, the consideration of the environmental costs could be interesting to add on the methodology to understand if any significant changes would be added on the

optimization results. Furthermore, more detailed analysis on the traffic models would allow the methodology to provide more accurate results mainly on the estimation of the user costs.

4. The detection of the damage could be improved and with more accuracy. In this thesis, the damage consideration was assumed to validate the methodology. Although, this measure could consider a combination of the inspectors' expertise and artificial intelligence techniques. By considering a database with similar records of other damaged elements, the damage estimation of the damaged element could be done by considering convolutional neural networks. This would allow a more accurate measure of the extension of the damage.
5. The consideration of the historical information about the recovery process after an hazard event. The recovery of the system was considered by the recovery functions provided on the literature. The same was considered for the recovery time. However, it could be interesting to compare those recovery functions with a real recovery.
6. Still concerning the hazard events, it could be interesting to understand exactly the costs regarding the labor work and the number of workers involved on the team. This would allow to state a multiobjective optimization problem that would take as the objectives the minimization of the recovery time and the minimization of the labor costs. In other words, the problem could be stated by the following question: *"How much money are the owners willing to pay to the workers for the fastest recovery possible?"*
7. Integration of other assets in the framework. The inclusion of other assets such as the railway track would be interesting to consider as well as the hazards that the tracks are exposed such as derailment problems and combine them to provide a multi-asset network perspective.

References

[1] V. A. Profillidis, *Railway management and engineering*: Ashgate Publishing, Ltd., 2014.

[2] J. Souza Dias, *Os Caminhos de Ferro Portugueses 1856-2006. pp. 238*, 2006.

[3] J. Clemente, "Inspecção e Manutenção de Pontes Ferroviárias – Seminário Segurança e Reabilitação de Pontes em Portugal.," *Edições FEUP, Colecção Colectâneas 8, Porto,* 2001.

[4] L. C. Neves and D. M. Frangopol, "Condition, safety and cost profiles for deteriorating structures with emphasis on bridges," *Reliability engineering & system safety,* vol. 89, pp. 185-198, 2005.

[5] N. M. Okasha and D. M. Frangopol, "Computational platform for the integrated life-cycle management of highway bridges," *Engineering structures,* vol. 33, pp. 2145-2153, 2011.

[6] G. Du and R. Karoumi, "Life cycle assessment of a railway bridge: comparison of two superstructure designs," *Structure and Infrastructure Engineering,* vol. 9, pp. 1149-1160, 2012.

[7] D. Nielsen, D. Raman, and G. Chattopadhyay, "Life cycle management for railway bridge assets," in *Proceedings of the Institution of Mechanical Engineers, Part F: Journal of Rail and Rapid Transit* vol. 227, ed, 2013, pp. 570-581.

[8] M. Safi, H. Sundquist, R. Karoumi, and G. Racutanu, "Development of the Swedish bridge management system by upgrading and expanding the use of LCC," *Structure and infrastructure engineering,* vol. 9, pp. 1240-1250, 2013.

[9] L. M. Freire, J. De Brito, and J. R. Correia, "Management system for road bridge structural bearings," *Structure and Infrastructure Engineering,* vol. 10, pp. 1068-1086, 2014.

[10] J. O. Almeida, P. F. Teixeira, and R. M. Delgado, "Life cycle cost optimisation in highway concrete bridges management," *Structure and Infrastructure Engineering,* vol. 11, pp. 1263-1276, 2015.

[11] D. Fernando, B. T. Adey, and N. Lethanh, "A model for the evaluation of intervention strategies for bridges affected by manifest and latent deterioration processes," *Structure and Infrastructure Engineering,* vol. 11, pp. 1466-1483, 2015.

[12] R. Denysiuk, J. Fernandes, J. C. Matos, L. C. Neves, and U. Berardinelli, "A computational framework for infrastructure asset maintenance scheduling," *Structural Engineering International,* vol. 26, pp. 94-102, 2016.

[13] P. C. Yianni, D. Rama, L. C. Neves, J. D. Andrews, and D. Castlo, "A Petri-Net-based modelling approach to railway bridge asset management," *Structure and Infrastructure Engineering,* vol. 13, pp. 287-297, 2016.

[14] H.-B. Xie, W.-J. Wu, and Y.-F. Wang, "Life-time reliability based optimization of bridge maintenance strategy considering LCA and LCC," *Journal of cleaner production,* vol. 176, pp. 36-45, 2018.

[15] M. Griffin and S. Patro, "Railway bridge assessment for effective asset management," in *Proceedings of the Institution of Civil Engineers-Bridge Engineering*, 2018, pp. 303-312.

[16] J. Fernandes, J. C. Matos, D. V. Oliveira, and A. Henriques, "Computational framework for a railway bridge maintenance strategies affected by gradual deterioration," *Life-Cycle Analysis and Assessment in Civil Engineering: Towards an Integrated Vision - Proceedings of the 6th International Symposium on Life-Cycle Civil Engineering (IALCCE),* pp. 1443-1449, 2018.

[17] J. Fernandes, M. Santamaria, J. C. Matos, D. V. Oliveira, L. Costa, and A. A. Henriques, "An infrastructure management system for railway bridges: Overview and application to a case study," *IABSE Symposium, Guimaraes 2019: Towards a Resilient Built Environment Risk and Asset Managemen,* pp. 168-175, 2019.

[18] A. Decò and D. M. Frangopol, "Risk assessment of highway bridges under multiple hazards," *Journal of Risk Research,* vol. 14, pp. 1057-1089, 2011.

[19] Y. Dong, D. M. Frangopol, and D. Saydam, "Time-variant sustainability assessment of seismically vulnerable bridges subjected to multiple hazards," *Earthquake Engineering & Structural Dynamics,* vol. 42, pp. 1451-1467, 2013.

[20] B. Zhu and D. M. Frangopol, "Risk-based approach for optimum maintenance of bridges under traffic and earthquake loads," *Journal of Structural Engineering,* vol. 139, pp. 422-434, 2013.

[21] S. Kameshwar and J. E. Padgett, "Multi-hazard risk assessment of highway bridges subjected to earthquake and hurricane hazards," *Engineering Structures,* vol. 78, pp. 154-166, 2014.

[22] A. Alipour and B. Shafei, "Seismic resilience of transportation networks with deteriorating components," *Journal of Structural Engineering,* vol. 142, p. C4015015, 2016.

[23] B. S. Vishwanath and S. Banerjee, "Life-Cycle Resilience of Aging Bridges under Earthquakes," *Journal of Bridge Engineering,* vol. 24, p. 04019106, 2019.

[24] P. Bocchini and D. M. Frangopol, "Restoration of bridge networks after an earthquake: Multicriteria intervention optimization," *Earthquake Spectra,* vol. 28, pp. 426-455, 2012.

[25] T. Van der Lei, P. Herder, and Y. Wijnia, *Asset management*: Springer, 2012.

[26] J. E. Amadi-Echendu, K. Brown, R. Willett, and J. Mathew, *Definitions, concepts and scope of engineering asset management*: Springer, 2010.

[27] ISO55000, "Asset management — Overview, principles and terminology," ed, 2014.

[28] N. Group, "International infrastructure management manual," *NAMS Group, Wellington,* 2006.

[29] M. J. Ryall, *Bridge management*: CRC Press, 2010.

[30] K. Golabi, R. B. Kulkarni, and G. B. Way, "A statewide pavement management system," *Interfaces,* vol. 12, pp. 5-21, 1982.

[31] K. Tsunokawa and R. Ul-Islam, "Pitfalls of HDM-4 strategy analysis," *International Journal of Pavement Engineering,* vol. 8, pp. 67-77, 2007.

[32] H. G. Kerali, J. B. Odoki, and E. E. Stannard, "Overview of HDM-4," *The Highway Development and Management Series,* vol. 4, 2000.

[33] G. Stojić, S. Vesković, I. Tanackov, and S. Milinković, "Model for railway infrastructure management organization," *Promet-Traffic&Transportation,* vol. 24, pp. 99-107, 2012.

[34] S. Jovanovic, "Modern railway infrastructure asset management," in *Proceedings of the 24th Southern African Transport Conference*, 2005.

[35] P. V. Paulo, F. Branco, and J. de Brito, "BuildingsLife: a building management system," *Structure and Infrastructure Engineering,* vol. 10, pp. 388-397, 2014.

[36] C. Kielhauser, B. T. Adey, and N. Lethanh, "Investigation of a static and a dynamic neighbourhood methodology to develop work programs for multiple close municipal infrastructure networks," *Structure and Infrastructure Engineering,* vol. 13, pp. 361-389, 2017.

[37] P. Dehghanian, M. Fotuhi-Firuzabad, F. Aminifar, and R. Billinton, "A comprehensive scheme for reliability centered maintenance in power distribution systems—Part I: Methodology," *IEEE Transactions on Power Delivery,* vol. 28, pp. 761-770, 2013.

[38] D. M. Abraham, R. Wirahadikusumah, T. Short, and S. Shahbahrami, "Optimization modeling for sewer network management," *Journal of construction engineering and management,* vol. 124, pp. 402-410, 1998.

[39] K. Morton, "Asset management in the electricity supply industry," *Power Engineering Journal,* vol. 13, pp. 233-240, 1999.

[40] J. Almeida, "Sistema de gestão de pontes com base em custos de ciclo de vida (translation: Life-cycle cost based bridge management systems)," Department of Civil Engineering, University of Porto, PhD Thesis, 2013.

[41] H.Hawk, "Bridge life-cycle cost analysis. NCHRP Report 483: Transportation Research Board," 2003.

[42] V. Patidar, *Multi-objective optimization for bridge management systems* vol. 67: Transportation Research Board, 2007.

[43] M. J. Markow and W. A. Hyman, *Bridge management systems for transportation agency decision making* vol. 397: Transportation Research Board, 2009.

[44] BRIME. (1999). *Bridge Management in Europe*. Available: https://trimis.ec.europa.eu/project/bridge-management-europe#tab-outline

[45] COST345. (2004). *Procedures Required for the Assessment of Highway Structures*. Available: http://cost345.zag.si/

[46] LIFECON. (2003). *Life Cycle Management System* Available: http://lifecon.vtt.fi

[47] DARTS, "Durable and reliable tunnel structures. Project with financial support of the European Commision under the Fifth Framework Program, GROWTH 2000 Project GRDI-25633," 2004.

[48] SAMARIS. (2006). *Sustainable and Advanced Materials for Road Infrastructures*. Available: http://samaris.zag.si/index.htm

[49] SAMCO. (2006). *Structural Assessment, Monitoring and Control*. Available: http://www.samco.org/

[50] S. Bridges. (2007). *Assessment for Future Traffic Demands and Longer Lives*. Available: http://www.sustainablebridges.net/

[51] INNOTRACK. (2011). *Innovative Track Systems*. Available: https://cordis.europa.eu/result/rcn/47369_en.html

[52] duratiNet. (2013). *Network Durable Transport Infrastructure in the Atlantic Area*. Available: durati.lnec.pt

[53] ETSI. (2012). *Bridge Life Cycle Optimisation* Available: http://etsi.aalto.fi/

[54] C. TU1406. (2018). *Quality specifications for roadway bridges, standardization at a European level (BridgeSpec)*. Available: https://www.tu1406.eu/

[55] SBRI. (2013). *Sustainable steel-composite bridges in built environment*. Available: https://publications.europa.eu/en/publication-detail/-/publication/bd7ff219-0fbd-4ff9-8702-44c31fe4b07b/language-en

[56] MAINLINE. (2014). *About MAINLINE project*. Available: http://www.mainline-project.eu/

[57] SustIMS, "Sustainable Infrastructure Management System," 2015.

[58] INFRALERT, "Liner infrastructure efficiency improvement by automated learning and optimized predictive maintenance techniques " 2018.

[59] AM4INFRA. (2018). *Common Framework for a European Life Cycle based Asset Management Approach for transport infrastructure networks* Available: http://www.am4infra.eu

[60] SAFEWAY. (2019). *GIS-based Infrastructure Management System for Optimized Response to Extreme Events on Terrestrial Transport Networks*. Available: https://www.safeway-project.eu/en

[61] P. D. Thompson, E. P. Small, M. Johnson, and A. R. Marshall, "The Pontis bridge management system," *Structural engineering international,* vol. 8, pp. 303-308, 1998.

[62] H. Hawk and E. P. Small, "The BRIDGIT bridge management system," *Structural engineering international,* vol. 8, pp. 309-314, 1998.

[63] R. Hajdin, "Kuba 4.0: The swiss road structure management system," 0097-8515, 2008.

[64] A. W. Hammad, J. Yan, and B. Mostofi, "Recent development of bridge management systems in Canada," in *2007 Annual Conference and Exhibition of the Transportation Association of Canada: Transportation-An Economic Enabler (Les Transports: Un Levier Economique) Transportation Association of Canada*, 2007.

[65] A. Miyamoto and M. Motoshita, "Development and practical application of a Bridge Management System (J-BMS) in Japan," *Civil Engineering Infrastructures Journal,* vol. 48, pp. 189-216, 2015.

[66] P. Nordengen, D. Welthagen, and E. De Fleuriot, "A bridge management system for the South African National Roads Agency," in *Bridge Management for inspection, maintenance, assessment and repair*, 2000.

[67] N. Rail, "Asset Management Strategy," 2014.

[68] T. Mendonça, A. Vieira, and P. Paulo, "Sistema de gestão de um parque de obras de arte," *Betao Estrutural,* 2002.

[69] N. H. Andersen, "DANBRO—A bridge management system for many levels," in *Bridge Evaluation, Repair and Rehabilitation*, ed: Springer, 1990, pp. 11-21.

[70] Z. Mirzaei, B. T. Adey, P. Thompson, and L. Klatter, "Overview of existing Bridge Management Systems-Report by the IABMAS Bridge Management Committee (2014)," in *7th International Conference on Bridge Maintenance, Safety and Management (IABMAS 2014)*, 2014.

[71] L. AASHTO, "Manual for condition evaluation and load and resistance factor rating of highway bridges," *American association of state highways transportation officials, DC, Washington,* 2003.

[72] C. Canadian Highway Bridge Design Code, "CSA-S6-00," *CSA International, Toronto, Ontario, Canada,* 2000.

[73] BD44/95, "The Assessment of Concrete Highway Bridges and Structures. Design Manual for Roads and Bridges. Vol.3, Section 4, Part 14.. HSMO, London," 1995.

[74] BD21/01, "The Assessment of Highway Bridges and Structures. Design Manual for Roads and Bridges. Vol.3, Section 4, Part 3.. HSMO, London," 2001.

[75] SIA-462, "Evaluation de la Sécurite Structurale des Ouvrages Existants. Société Suisse des Ingénieurs et des Architectes, Zürich," 1994.

[76] C. von Scholten, I. Enevoldsen, and T. Arnbjerg-Nielsen, "Reliability-Based Classification of the Load Carrying Capacity of Existing Bridges, Report 291," ed: Road Directorate, Ministry of Transport, Denmark, 2004.

[77] A. O'Connor and I. Enevoldsen, "Probability-based assessment of bridges–European experience," in *Austroads bridge conference, 7th, Auckland, New Zealand* 2009.

[78] J. Schneider, *Introduction to safety and reliability of structures* vol. 5: Iabse, 2006.

[79] J. K. Paik and R. E. Melchers, *Condition assessment of aged structures*: Elsevier, 2014.

[80] W. Rücker, F. Hille, and R. Rohrmann, "Guideline for the assessment of existing structures," *SAMCO Final Report,* 2006.

[81] EN CEN 1992, "Eurocode 2: Design of concrete structures. Part 1-1:General rules and rules for buildings," 2010.

[82] J. R. Casas and D. Wisniewski, "Safety requirements and probabilistic models of resistance in the assessment of existing railway bridges," *Structure and Infrastructure Engineering,* vol. 9, pp. 529-545, 2013.

[83] A. Strauss, A. Vidovic, I. Zambon, F. Dengg, N. Tanasic, and J. C. Matos, "Performance indicators for roadway bridges," in *IABMAS Conference 2016*, 2016, pp. 965-970.

[84] A. Mandić Ivanković, D. Skokandić, A. Žnidarič, and M. Kreslin, "Bridge performance indicators based on traffic load monitoring," *Structure and Infrastructure Engineering,* pp. 1-13, 2017.

[85] M. Sánchez-Silva, D. M. Frangopol, J. Padgett, and M. Soliman, "Maintenance and Operation of Infrastructure Systems: Review," *Journal of Structural Engineering,* vol. 142, p. F4016004, 2016.

[86] R. E. Melchers, *Structural Reliability: Analysis and Prediction*: Horwood, 1987.

[87] A. S. Nowak and K. R. Collins, *Reliability of structures*: CRC Press, 2012.

[88] P. Thoft-Cristensen and M. J. Baker, *Structural reliability theory and its applications*: Springer Science & Business Media, 2012.

[89] M. H. Faber, *Statistics and probability theory: in pursuit of engineering decision support* vol. 18: Springer Science & Business Media, 2012.

[90] A. Olsson, G. Sandberg, and O. Dahlblom, "On Latin hypercube sampling for structural reliability analysis," *Structural safety,* vol. 25, pp. 47-68, 2003.

[91] C. J. Sallaberry, J. C. Helton, and S. C. Hora, "Extension of Latin hypercube samples with correlated variables," *Reliability Engineering & System Safety,* vol. 93, pp. 1047-1059, 2008.

[92] C. G. Bucher and U. Bourgund, "A fast and efficient response surface approach for structural reliability problems," *Structural safety,* vol. 7, pp. 57-66, 1990.

[93] N. Gayton, J. Bourinet, and M. Lemaire, "CQ2RS: a new statistical approach to the response surface method for reliability analysis," *Structural safety,* vol. 25, pp. 99-121, 2003.

[94] X. S. Nguyen, A. Sellier, F. Duprat, and G. Pons, "Adaptive response surface method based on a double weighted regression technique," *Probabilistic Engineering Mechanics,* vol. 24, pp. 135-143, 2009.

[95] N. Roussouly, F. Petitjean, and M. Salaun, "A new adaptive response surface method for reliability analysis," *Probabilistic Engineering Mechanics,* vol. 32, pp. 103-115, 2013.

[96] J. R. Casas, "Reliability-based assessment of masonry arch bridges," *construction and building materials,* vol. 25, pp. 1621-1631, 2011.

[97] A. S. Nowak, C.-H. Park, and J. R. Casas, "Reliability analysis of prestressed concrete bridge girders: comparison of Eurocode, Spanish Norma IAP and AASHTO LRFD," *Structural safety,* vol. 23, pp. 331-344, 2001.

[98] S. N. IAP -98, "Actions in highway bidges. Road Directorate, Spanish Ministry of Public Works, Madrid," ed, 1998.

[99] E. E. 1991-3, "Eurocode 1: Basis of Design and Actions on Structures. Part 3: Traffic Loads on Bridges," ed, 1994.

[100] AASHTO, "AASHTO LRFD Bridge design specifications. American Association of State Highway and Transportation Officials," ed, 2012.

[101] V. N. Moreira, J. Fernandes, J. C. Matos, and D. V. Oliveira, "Reliability-based assessment of existing masonry arch railway bridges," *Construction and Building Materials,* vol. 115, pp. 544-554, 2016.

[102] J. C. Matos, P. J. Cruz, I. B. Valente, L. C. Neves, and V. N. Moreira, "An innovative framework for probabilistic-based structural assessment with an application to existing reinforced concrete structures," *Engineering Structures,* vol. 111, pp. 552-564, 2016.

[103] E. S. Cavaco, L. A. Neves, and J. R. Casas, "Reliability-based approach to the robustness of corroded reinforced concrete structures," *Structural Concrete,* vol. 18, pp. 316-325, 2017.

[104] H. Guimarães, J. C. Matos, and A. A. Henriques, "An innovative adaptive sparse response surface method for structural reliability analysis," *Structural Safety,* vol. 73, pp. 12-28, 2018.

[105] I. 13822, "Bases for design of structures. Assessment of existing structures," ed, 2012.

[106] ISO 13822, "Bases for design of structures. Assessment of existing structures," 2012.

[107] EN1990, "Eurocode 0-Basis of structural design Commité Européen de Normalisation (CEN), Brussels, Belgium," ed, 2002.

[108] ISO 2394, "General principles on reliability for structures. Ethiopian Standards Agency," 2012.

[109] JCSS. (2001). *Joint Committee on Structural Safety. Probabilistic Model Code*. Available: https://www.jcss-lc.org/jcss-probabilistic-model-code/

[110] *fib*, "Bulletin 22: Monitoring and Safety Evaluation of Existing Concrete Structures, State-of-Art Report, pp. 297," 2003.

[111] P. Hopkin, *Fundamentals of risk management: understanding, evaluating and implementing effective risk management*: Kogan Page Publishers, 2014.

[112] M. H. Faber and M. G. Stewart, "Risk assessment for civil engineering facilities: critical overview and discussion," *Reliability engineering & system safety,* vol. 80, pp. 173-184, 2003.

[113] B. R. Ellingwood, "Risk-informed condition assessment of civil infrastructure: state of practice and research issues," *Structure and infrastructure engineering,* vol. 1, pp. 7-18, 2005.

[114] M. Faber, "Risk assessment in engineering: principles, system representation & risk criteria," *JCSS Joint Committee of Structural Safety,* 2008.

[115] D. Imhof, "Risk assessment of existing bridge structures," University of Cambridge, 2004.

[116] A. Al-Wazeer, "Risk-based bridge maintenance strategies," PhD Thesis, University of Maryland, 2007.

[117] J. E. Padgett, K. Dennemann, and J. Ghosh, "Risk-based seismic life-cycle cost–benefit (LCC-B) analysis for bridge retrofit assessment," *Structural Safety,* vol. 32, pp. 165-173, 2010.

[118] A. Decò and D. M. Frangopol, "Life-cycle risk assessment of spatially distributed aging bridges under seismic and traffic hazards," *Earthquake Spectra,* vol. 29, pp. 127-153, 2013.

[119] B. Zhu and D. M. Frangopol, "Reliability, redundancy and risk as performance indicators of structural systems during their life-cycle," *Engineering Structures,* vol. 41, pp. 34-49, 2012.

[120] D. Saydam, P. Bocchini, and D. M. Frangopol, "Time-dependent risk associated with deterioration of highway bridge networks," *Engineering Structures,* vol. 54, pp. 221-233, 2013.

[121] D. Saydam and D. M. Frangopol, "Risk-based maintenance optimization of deteriorating bridges," *Journal of Structural Engineering,* vol. 141, p. 04014120, 2014.

[122] D. Saydam, D. M. Frangopol, and Y. Dong, "Assessment of risk using bridge element condition ratings," *Journal of Infrastructure Systems,* vol. 19, pp. 252-265, 2012.

[123] G. Barone and D. M. Frangopol, "Reliability, risk and lifetime distributions as performance indicators for life-cycle maintenance of deteriorating structures," *Reliability Engineering & System Safety,* vol. 123, pp. 21-37, 2014.

[124] Y. Dong and D. Frangopol, "Risk-Informed Performance Assessment and Updating of Fatigue-Sensitive Structures at Component and System Level," in *Life-Cycle of Engineering Systems: Emphasis on Sustainable Civil Infrastructure: Proceedings of the Fifth International Symposium on Life-Cycle Civil Engineering (IALCCE 2016), 16-19 October 2016, Delft, The Netherlands*, 2016, p. 435.

[125] B. Zhu and D. M. Frangopol, "Time-variant risk assessment of bridges with partially and fully closed lanes due to traffic loading and scour," *Journal of Bridge Engineering,* vol. 21, p. 04016021, 2016.

[126] M. A. Zanini, F. Faleschini, and C. Pellegrino, "Probabilistic seismic risk forecasting of aging bridge networks," *Engineering Structures,* vol. 136, pp. 219-232, 2017.

[127] T. Yilmaz, S. Banerjee, and P. A. Johnson, "Uncertainty in risk of highway bridges assessed for integrated seismic and flood hazards," *Structure and Infrastructure Engineering,* vol. 14, pp. 1182-1196, 2017.

[128] A. Mondoro and D. M. Frangopol, "Risk-based cost-benefit analysis for the retrofit of bridges exposed to extreme hydrologic events considering multiple failure modes," *Engineering Structures,* vol. 159, pp. 310-319, 2018.

[129] J. W. Baker, M. Schubert, and M. H. Faber, "On the assessment of robustness," *Structural Safety,* vol. 30, pp. 253-267, 2008.

[130] T. Canisius, J. Sorensen, and J. Baker, "Robustness of structural systems–a new focus for the joint committee on structural safety (JCSS)," in *Proc., 10th Int. Conf. on Application of Statistic and Probability in Civil Engineering (ICASP10)*, 2007.

[131] U. Starossek and M. Haberland, "Disproportionate collapse: terminology and procedures," *Journal of performance of constructed facilities,* vol. 24, pp. 519-528, 2010.

[132] EN1991-1-7, "General Actions – Accidental Actions. Commité Européen de Normalisation (CEN), Brussels, Belgium," 2006.

[133] D. M. Frangopol and J. P. Curley, "Effects of damage and redundancy on structural reliability," *Journal of structural engineering,* vol. 113, pp. 1533-1549, 1987.

[134] U. Starossek and M. Haberland, "Approaches to measures of structural robustness," *Structure and Infrastructure Engineering,* vol. 7, pp. 625-631, 2011.

[135] F. Biondini and S. Restelli, "Damage propagation and structural robustness," in *Life-Cycle Civil Engineering: Proceedings of the International Symposium on Life-Cycle Civil Engineering, IALCCE'08, Held in Varenna, Lake Como, Italy on June 11-14 2008*, 2008, p. 131.

[136] E. Cavaco, "Robustness of corroded bridges," PhD thesis, Universidade Nova de Lisboa, 2013.

[137] G. Fu and D. M. Frangopol, "Balancing weight, system reliability and redundancy in a multiobjective optimization framework," *Structural Safety,* vol. 7, pp. 165-175, 1990.

[138] N. C. Lind, "A measure of vulnerability and damage tolerance," *Reliability Engineering & System Safety,* vol. 48, pp. 1-6, 1995.

[139] M. Ghosn and F. Moses, *Redundancy in highway bridge superstructures* vol. 406: Transportation Research Board, 1998.

[140] M. Ghosn, D. Frangopol, T. McAllister, M. Shah, S. Diniz, B. Ellingwood*, et al.*, "Reliability-based performance indicators for structural members," *Journal of Structural Engineering,* vol. 142, p. F4016002, 2016.

[141] P. Thoft-Christensen, "Assessment of the reliability profiles for concrete bridges," *Engineering structures,* vol. 20, pp. 1004-1009, 1998.

[142] A. O'Connor and I. Enevoldsen, "Probability-based assessment of bridges-European experience," in *Australian Structural Engineering Conference*, 2005.

[143] D. F. Wiśniewski, "Safety formats for the assessment of concrete bridges: with special focus on precast concrete," Department of Civil Engineering, University of Minho, PhD Thesis, 2007.

[144] A. A. Henriques, "Aplicação de novos conceitos de segurança no dimensionamento do betão estrutural (translation: Application of new safety concepts on the structural concrete design, PhD Thesis)," Department of Civil Engineering, University of Porto, 1998.

[145] C. Brett and Y. Lu, "Assessment of robustness of structures: Current state of research," *Frontiers of Structural and Civil Engineering,* vol. 7, pp. 356-368, 2013.

[146] RSA, "Regulamento de Segurança e Ações para Estruturas de Edifícios e Pontes (translation "Code of safety and actions for building structures and bridges")," 1983.

[147] SB-LRA, "Guideline for load and resistance assessment of existing European railway bridges – advices on the use of advanced methods. Integrated project sustainable bridges. VI Framework programme. Brussels. Available in: http://www.sustainablebridges.net," 2007.

[148] EN-CEN, "Eurocode 2: Design of concrete structures. Part 1-1:General rules and rules for buildings," 2010.

[149] J. Almeida, "Sistema de gestão de pontes com base em custos de ciclo de vida (translation: Life-cycle cost based bridge management systems)," Universidade do Porto, 2013.

[150] Matlab, "Design of Experiments Toolbox," 2016.

[151] European Committee for Standardization (CEN), "EN 1991-2, Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges," ed. Brussels, 2003.

[152] L. F. Martha. (2017). *Ftool - Two-Dimensional Frame Analysis Tool, Version 4.00 Basic*. Available: https://www.ftool.com.br/Ftool/

[153] CEN, "Eurocode 3 - Design of steel structures, Part 1-1: General rules and rules for buildings," ed, 2010.

[154] C. M. R. Costa, "Análise numérica e experimental do comportamento estrutural de pontes em arco de alvenaria de pedra (translation: Numerical and experimental analysis of the structural behavior of masonry arch bridges)," Department of Civil Engineering, University of Porto, PhD Thesis, 2009.

[155] S. Bridges, "Safety and Probabilistic Modelling Background document, D4.4. Sustainable Development Global Change & Ecosystems Integrated Project. ," 2007.

[156] D. V. Oliveira, P. B. Lourenço, and C. Lemos, "Geometric issues and ultimate load capacity of masonry arch bridges from the northwest Iberian Peninsula," *Engineering Structures,* vol. 32, pp. 3955-3965, 2010.

[157] N. M. F. Araújo, "High-speed trains on ballasted railway track: dynamic stress field analysis," PhD Thesis, School of Engineering, Civil Engineering, University of Minho, 2010.

[158] N. Rail, "The structural assessment of underbridges," *Network Rail, London, UK, Guidance Note NR/GN/CIV/025,* 2006.

[159] S. De Santis, "Load-carrying capability and seismic assessment of masonry bridges," Doctoral Thesis, Sezione di Scienze dell'Ingegneria Civile - Scuola Dottorale in Ingegneria - Department of Structures Roma Tre University, 2011.

[160] L. S. RING, "Masonry arch analysis software, version 3.2," 2020.

[161] M. Gilbert, C. Smith, W. Darwich, and A. Tyas, "LimitState:RING Manual," vol. 1, 3.1.a ed. United Kingdom: LimitState Ltd, 2014.

[162] M. Gilbert, "Gross displacement mechanism analysis of masonry bridges and tunnels," presented at the 11th International Brick Block Masonry Conference, Tongji University, Shanghai, China, 1997.

[163] M. Gilbert, "The behaviour of masonry arch bridges containing defects," Doctoral Thesis, University of Manchester, 1993.

[164] M. Gilbert, "Limit analysis applied to masonry arch bridges: state-of-the-art and recent developments," in *5th International arch bridges conference*, 2007, pp. 13-28.

[165] R. R. D.Novák, M.Vorechovsky, "FReET Program Documentation Part -2 User Manual, 1.4 ed., Cervenka Consulting, Prague, Czech Republic," 2006.

[166] M. Sánchez-Silva and G.-A. Klutke, *Reliability and life-cycle analysis of deteriorating systems*: Springer, 2016.

[167] G. Morcous, "Case-based reasoning for modeling bridge deterioration," Concordia University, PhD Thesis, 2000.

[168] P. Albrecht and A. H. Naeemi, "Performance of weathering steel in bridges," *NCHRP report,* 1984.

[169] J. R. Kayser and A. S. Nowak, "Capacity loss due to corrosion in steel-girder bridges," *Journal of Structural Engineering,* vol. 115, pp. 1525-1537, 1989.

[170] J. Ghosh and P. Sood, "Consideration of time-evolving capacity distributions and improved degradation models for seismic fragility assessment of aging highway bridges," *Reliability Engineering & System Safety,* vol. 154, pp. 197-218, 2016.

[171] K. A. T. Vu and M. G. Stewart, "Structural reliability of concrete bridges including improved chloride-induced corrosion models," *Structural safety,* vol. 22, pp. 313-333, 2000.

[172] M. Torres-Luque, E. Bastidas-Arteaga, F. Schoefs, M. Sánchez-Silva, and J. F. Osma, "Non-destructive methods for measuring chloride ingress into concrete: State-of-the-art and future challenges," *Construction and building materials,* vol. 68, pp. 68-81, 2014.

[173] I. Zambon, M. P. Santamaria Ariza, J. Campos e Matos, and A. Strauss, "Value of Information (VoI) for the Chloride Content in Reinforced Concrete Bridges," *Applied Sciences,* vol. 10, p. 567, 2020.

[174] E. B. EuRam III, "DuraCrete final technical report, probabilistic performance based durability design of concrete structures," *Document BE95-1347,* 2000.

[175] D.-E. Choe, P. Gardoni, D. Rosowsky, and T. Haukaas, "Probabilistic capacity models and seismic fragility estimates for RC columns subject to corrosion," *Reliability Engineering & System Safety,* vol. 93, pp. 383-393, 2008.

[176] A. Hatami and G. Morcous, "Developing deterioration models for Nebraska bridges," 2011.

[177] T. M. Zayed, L.-M. Chang, and J. D. Fricker, "Statewide performance function for steel bridge protection systems," *Journal of performance of constructed facilities,* vol. 16, pp. 46-54, 2002.

[178] Y. Shen, Y. Zeng, L. Zhu, and T. Huang, "A deformation analysis method of stepwise regression for bridge deflection prediction," in *International Conference on Intelligent Earth Observing and Applications 2015*, 2015, p. 98082Q.

[179] M. Y. Shahin, *Pavement management for airports, roads, and parking lots* vol. 501: Springer New York, 2005.

[180] R. Farebrother, "Nonlinear regression modelling: A unified practical approach: David A. Ratkowsky,(Dekker, New York, 1983) $39.00, pp. 276," ed: Elsevier, 1986.

[181] D. M. Bates and D. G. Watts, *Nonlinear regression analysis and its applications*: Wiley, 1988.

[182] G. Morcous, "Case-B ased Reasoning for Modeling Bridge Deterioration," Concordia University Montreal, Quebec, Canada, 2000.

[183] Y. Shen, Y. Zeng, L. Zhu, and T. Huang, "A deformation analysis method of stepwise regression for bridge deflection prediction," in *International Conference on Intelligent Earth Observing and Applications*, 2015, pp. 98082Q-98082Q-10.

[184] G. Morcous, H. Rivard, and A. Hanna, "Modeling bridge deterioration using case-based reasoning," *Journal of Infrastructure Systems,* vol. 8, pp. 86-95, 2002.

[185] G. Morcous, "Developing deterioration models for Nebraska bridges," 2011.

[186] M. Pandey and X.-X. Yuan, "A comparison of probabilistic models of deterioration for life cycle management of structures," in *Advances in Engineering Structures, Mechanics & Construction*, ed: Springer, 2006, pp. 735-746.

[187] D. M. Frangopol, M. J. Kallen, and J. M. Van Noortwijk, "Probabilistic models for life-cycle performance of deteriorating structures: review and future directions," *Progress in Structural Engineering and Materials,* vol. 6, pp. 197-212, 2004.

[188] P. Thoft-Christensen, "Advanced bridge management systems," *Structural Engineering Review,* vol. 3, pp. 151-163, 1995.

[189] D. Frangopol and P. C. Das, "Management of bridge stocks based on future reliability and maintenance costs," *Bridge design, construction, and maintenance,* pp. 45-58, 1999.

[190] D. Frangopol, E. Gharaibeh, J. Kong, and M. Miyake, "Optimal network-level bridge maintenance planning based on minimum expected cost," *Transportation Research Record: Journal of the Transportation Research Board,* pp. 26-33, 2000.

[191] D. M. Frangopol, J. S. Kong, and E. S. Gharaibeh, "Reliability-based life-cycle management of highway bridges," *Journal of computing in civil engineering,* vol. 15, pp. 27-34, 2001.

[192] G. Gaal, "Prediction of deterioration of concrete bridges," TU Delft, Delft University of Technology, 2004.

[193] D. Frangopol and L. Neves, "Life-cycle maintenance of structures by condition, reliability and cost oriented probabilistic optimization," *Innovation in computational structures technology,* pp. 95-110, 2006.

[194] G. Morcous, "Performance prediction of bridge deck systems using Markov chains," *Journal of performance of Constructed Facilities,* vol. 20, pp. 146-155, 2006.

[195] Y. Tsuda, K. Kaito, K. Aoki, and K. Kobayashi, "Estimating Markovian transition probabilities for bridge deterioration forecasting," *Structural Engineering/Earthquake Engineering,* vol. 23, pp. 241s-256s, 2006.

[196] M. Sánchez-Silva, D. M. Frangopol, J. Padgett, and M. Soliman, "Maintenance and operation of infrastructure systems," *Journal of Structural Engineering,* vol. 142, p. F4016004, 2016.

[197] A. A. Butt, M. Y. Shahin, K. J. Feighan, and S. H. Carpenter, *Pavement performance prediction model using the Markov process*, 1987.

[198] J. Yang, M. Gunaratne, J. J. Lu, and B. Dietrich, "Use of recurrent Markov chains for modeling the crack performance of flexible pavements," *Journal of Transportation Engineering,* vol. 131, pp. 861-872, 2005.

[199] K. Kobayashi, M. Do, and D. Han, "Estimation of Markovian transition probabilities for pavement deterioration forecasting," *KSCE Journal of Civil Engineering,* vol. 14, pp. 343-351, 2010.

[200] K. A. Abaza, "Back-calculation of transition probabilities for Markovian-based pavement performance prediction models," *International Journal of Pavement Engineering,* vol. 17, pp. 253-264, 2014.

[201] R. Hassan, O. Lin, and A. Thananjeyan, "Probabilistic modelling of flexible pavement distresses for network management," *International Journal of Pavement Engineering,* vol. 18, pp. 216-227, 2015.

[202] A. V. Moreira, J. Tinoco, J. R. Oliveira, and A. Santos, "An application of Markov chains to predict the evolution of performance indicators based on pavement historical data," *International Journal of Pavement Engineering,* pp. 1-12, 2016.

[203] Y. Jiang, M. Saito, and K. C. Sinha, *Bridge performance prediction model using the Markov chain*, 1988.

[204] M. A. Cesare, C. Santamarina, C. Turkstra, and E. H. Vanmarcke, "Modeling bridge deterioration with Markov chains," *Journal of Transportation Engineering,* vol. 118, pp. 820-833, 1992.

[205] A. D. Orcesi and C. F. Cremona, "A bridge network maintenance framework for Pareto optimization of stakeholders/users costs," *Reliability Engineering & System Safety,* vol. 95, pp. 1230-1243, 2010.

[206] A. D. Orcesi and C. F. Cremona, "Optimization of maintenance strategies for the management of the national bridge stock in France," *Journal of bridge engineering,* vol. 16, pp. 44-52, 2010.

[207] F. Sobreiro, "Modelos de previsão de deterioração de pontes existentes: processos de Markov (translation: Prediction degradation models of existing bridges: Markov processes)," Faculdade de Ciências e Tecnologia, Master Thesis, 2011.

[208] N. K. W. Wellalage, T. Zhang, and R. Dwight, "Calibrating Markov Chain–Based Deterioration Models for Predicting Future Conditions of Railway Bridge Elements," *Journal of Bridge Engineering,* vol. 20, p. 04014060, 2014.

[209] L. Li, F. Li, Z. Chen, and L. Sun, "Bridge Deterioration Prediction Using Markov-Chain Model Based on the Actual Repair Status in Shanghai," in *Transportation Research Board 95th Annual Meeting*, 2016.

[210] L. Bai, R. Liu, Q. Sun, F. Wang, and P. Xu, "Markov-based model for the prediction of railway track irregularities," *Proceedings of the Institution of Mechanical Engineers, Part F: Journal of rail and rapid transit,* vol. 229, pp. 150-159, 2015.

[211] Y. Shafahi and R. Hakhamaneshi, "Application of a maintenance management model for Iranian railways based on the Markov chain and probabilistic dynamic programming," *International Journal of Science and Technology. Transaction A: Civil Engineering,* vol. 16, pp. 87-97, 2009.

[212] J.-A. Zakeri and S. Shahriari, "Developing a deterioration probabilistic model for rail wear," *International Journal of Traffic and Transportation Engineering,* vol. 1, pp. 13-18, 2012.

[213] T. A. F. Regado, "Desenvolvimento de um modelo de desempenho para infraestruturas ferroviárias aplicado à linha férrea," 2014.

[214] P. D. DeStefano and D. A. Grivas, "Method for estimating transition probability in bridge deterioration models," *Journal of Infrastructure Systems,* vol. 4, pp. 56-62, 1998.

[215] S. Mašović, S. Stošić, and R. Hajdin, "Application of Semi-Markov Decision Process in Bridge Management," in *IABSE Symposium Report*, 2015, pp. 1-8.

[216] M. Nasrollahi and G. Washer, "Estimating inspection intervals for bridges based on statistical analysis of national bridge inventory data," *Journal of Bridge Engineering,* vol. 20, p. 04014104, 2014.

[217] A. K. Agrawal, A. Kawaguchi, and Z. Chen, "Deterioration rates of typical bridge elements in New York," *Journal of Bridge Engineering,* vol. 15, pp. 419-429, 2010.

[218] Y. Kleiner, "Scheduling inspection and renewal of large infrastructure assets," *Journal of infrastructure systems,* vol. 7, pp. 136-143, 2001.

[219] L. E. S. Oliveira and M. E. Morita, "Introduçao aos modelos escondidos de markov (hmm) (translation: Introduction to Hidden Markov Models)," *Pontifícia Universidade Católica do Paraná,* p. 16, 2000.

[220] L. Rabiner and B. Juang, "An introduction to hidden Markov models," *ieee assp magazine,* vol. 3, pp. 4-16, 1986.

[221] L. MacDonald Iain and Z. Walter, *Hidden Markov and other models for discrete-valued time series* vol. Vol 110: Chapman and Hall/CRC; 1st edition (January 1, 1997), 1997.

[222] K. Kobayashi, K. Kaito, and N. Lethanh, "A statistical deterioration forecasting method using hidden Markov model for infrastructure management," *Transportation Research Part B: Methodological,* vol. 46, pp. 544-561, 2012.

[223] N. Lethanh and B. T. Adey, "A hidden Markov model for modeling pavement deterioration under incomplete monitoring data," *World Academy of Science, Engineering and Technology,* vol. 6, pp. 46-53, 2012.

[224] N. Lethanh, K. Kaito, and K. Kobayashi, "Infrastructure deterioration prediction with a Poisson hidden Markov model on time series data," *Journal of Infrastructure Systems,* vol. 21, p. 04014051, 2014.

[225] M. Santamaria Ariza, I. Zambon, H. S. Sousa, J. A. Campos e Matos, and A. Strauss, "Comparison of forecasting models to predict concrete bridge decks performance," *Structural Concrete,* 2020.

[226] J. L. Peterson, "Petri nets," *ACM Computing Surveys (CSUR),* vol. 9, pp. 223-252, 1977.

[227] P. C. Yianni, D. Rama, L. C. Neves, J. D. Andrews, and D. Castlo, "A Petri-Net-based modelling approach to railway bridge asset management," *Structure and Infrastructure Engineering,* vol. 13, pp. 287-297, 2017.

[228] B. Le and J. Andrews, "Petri net modelling of bridge asset management using maintenance-related state conditions," *Structure and Infrastructure Engineering,* vol. 12, pp. 730-751, 2016.

[229] C. Ferreira, "Use of Petri Nets to Manage Civil Engineering Infrastructures," Faculdade de Ciências e Tecnologia, University of Lisbon, PhD Thesis, 2018.

[230] J. Sobanjo, "A neural network approach to modeling bridge deterioration," in *Computing in Civil Engineering*, 1997, pp. 623-626.

[231] Y.-H. Huang, "Artificial neural network model of bridge deterioration," *Journal of Performance of Constructed Facilities,* vol. 24, pp. 597-602, 2010.

[232] A. Kosgodagan, O. Morales-Nápoles, J. Maljaars, T. Yeung, and B. Castanier, "Bayesian networks to quantify transition rates in degradation modeling: application to a set of steel bridges in the Netherlands," in *12th International Conference on Applications of Statistics and Probability in Civil Engineering, ICASP12, Vancouver, Canada, July 12-15, 2015*, 2015.

[233] R. Wang, L. Ma, C. Yan, and J. Mathew, "Condition deterioration prediction of bridge elements using Dynamic Bayesian Networks (DBNs)," in *Quality, Reliability, Risk, Maintenance, and Safety Engineering (ICQR2MSE), 2012 International Conference on*, 2012, pp. 566-571.

[234] M. I. Rafiq, M. K. Chryssanthopoulos, and S. Sathananthan, "Bridge condition modelling and prediction using dynamic Bayesian belief networks," *Structure and Infrastructure Engineering,* vol. 11, pp. 38-50, 2015.

[235] A. Petcherdchoo, "Maintaining condition and safety of deteriorating bridges by probabilistic models and optimization," University of Colorado, 2004.

[236] L. Collins, "An introduction to Markov chain analysis," 1975.

[237] J. Andrews, "A modelling approach to railway track asset management," *Proceedings of the Institution of Mechanical Engineers, Part F: Journal of Rail and Rapid Transit,* vol. 227, pp. 56-73, 2013.

[238] Z.-H. Lu, Y.-G. Zhao, and K. Yu, "Stochastic modeling of corrosion propagation for service life prediction of chloride contaminated RC structures," in *Proceedings of the International Symposium on Life-Cycle Civil Engineering, IALCCE'08, held in Varenna, Lake Como, Italy*, 2008, pp. 195-201.

[239] E. Portela, T. Bittencourt, and A. Beck, "Time-dependent probabilistic modeling of corrosion propagation for service life prediction in RC structures," *Maintenance, Monitoring, Safety, Risk and Resilience of Bridges and Bridge Networks,* p. 375, 2016.

[240] E. Minaie and F. Moon, "Practical and Simplified Approach for Quantifying Bridge Resilience," *Journal of Infrastructure Systems,* vol. 23, p. 04017016, 2017.

[241] C. Ferreira, L. C. Neves, J. C. Matos, and J. M. S. Soares, "A degradation and maintenance model: Application to Portuguese context," *Proceedings of Bridge Maintenance, Safety, Management and Life Extension,* pp. 483-489, 2014.

[242] A. S. Nowak and M. M. Szerszen, "Reliability profiles for steel girder bridges with regard to corrosion and fatigue," *Journal of Theoretical and Applied Mechanics,* vol. 39, pp. 339-352, 2001.

[243] M. Sánchez-Silva and G.-A. Klutke, *Reliability and life-cycle analysis of deteriorating systems* vol. 182: Springer, 2016.

[244] G. Brundtland, M. Khalid, S. Agnelli, S. Al-Athel, B. Chidzero, L. Fadika*, et al.*, "Our common future (brundtland report)," 1987.

[245] E. K. Zavadskas, J. Antucheviciene, T. Vilutiene, and H. Adeli, "Sustainable decision-making in civil engineering, construction and building technology," *Sustainability,* vol. 10, p. 14, 2017.

[246] W. Kloepffer, "Life cycle sustainability assessment of products," *The International Journal of Life Cycle Assessment,* vol. 13, p. 89, 2008.

[247] J.Elkington, "Cannibals with forks, the triple bottom line of 21st century business, Capstone, Oxford, U.K.," 1997.

[248] P. Bocchini, D. M. Frangopol, T. Ummenhofer, and T. Zinke, "Resilience and sustainability of civil infrastructure: Toward a unified approach," *Journal of Infrastructure Systems,* vol. 20, p. 04014004, 2014.

[249] H. Gervásio and L. S. Da Silva, "A probabilistic decision-making approach for the sustainable assessment of infrastructures," *Expert Systems with Applications,* vol. 39, pp. 7121-7131, 2012.

[250] Y. Dong, D. M. Frangopol, and D. Saydam, "Sustainability of highway bridge networks under seismic hazard," *Journal of earthquake engineering,* vol. 18, pp. 41-66, 2014.

[251] S. Engert, R. Rauter, and R. J. Baumgartner, "Exploring the integration of corporate sustainability into strategic management: a literature review," *Journal of cleaner production,* vol. 112, pp. 2833-2850, 2016.

[252] Z. Lounis and T. P. McAllister, "Risk-based decision making for sustainable and resilient infrastructure systems," *Journal of Structural Engineering,* vol. 142, p. F4016005, 2016.

[253] S. I. Assessment, "Guidelines and principles for social impact assessment," *Environmental Impact Assessment Review,* vol. 15, pp. 11-43, 1994.

[254] G. Du and R. Karoumi, "Life cycle assessment framework for railway bridges: literature survey and critical issues," *Structure and Infrastructure Engineering,* vol. 10, pp. 277-294, 2014.

[255] D. Nielsen, G. Chattopadhyay, and D. Raman, "Life Cycle Cost Estimation for Railway Bridge Maintenance," in *International Heavy Haul Association Conference*, 2013, pp. 311-318.

[256] F. Biondini and D. M. Frangopol, "Life-cycle performance of deteriorating structural systems under uncertainty," *Journal of Structural Engineering,* vol. 142, p. F4016001, 2016.

[257] M. Missimer, K.-H. Robèrt, and G. Broman, "A strategic approach to social sustainability–Part 1: exploring the social system," *Journal of Cleaner Production,* vol. 140, pp. 32-41, 2017.

[258] L. A. Sierra, V. Yepes, and E. Pellicer, "A review of multi-criteria assessment of the social sustainability of infrastructures," *Journal of Cleaner Production,* vol. 187, pp. 496-513, 2018.

[259] S. Di Cesare, F. Silveri, S. Sala, and L. Petti, "Positive impacts in social life cycle assessment: state of the art and the way forward," *The International Journal of Life Cycle Assessment,* vol. 23, pp. 406-421, 2018.

[260] R. Valdes-Vasquez and L. E. Klotz, "Social sustainability considerations during planning and design: framework of processes for construction projects," *Journal of construction engineering and management,* vol. 139, pp. 80-89, 2012.

[261] H. M. d. S. Gervásio, "Sustainable design and integral life-cycle analysis of bridges," 2010.

[262] P. Thoft-Christensen, "Life-cycle cost-benefit (LCCB) analysis of bridges from a user and social point of view," *Structures & Infrastructure Engineering,* vol. 5, pp. 49-57, 2009.

[263] H. Sundquist and R. Karoumi, "Life Cycle Cost Methodology and LCC Tools," *Structure Design and Bridges, Stockholm,* 2012.

[264] S. A. Simson, L. Ferreira, and M. H. Murray, "Rail track maintenance planning: An assessment model," *Transportation research record,* vol. 1713, pp. 29-35, 2000.

[265] D. M. Frangopol, K.-Y. Lin, and A. C. Estes, "Life-cycle cost design of deteriorating structures," *Journal of Structural Engineering,* vol. 123, pp. 1390-1401, 1997.

[266] N. M. Okasha and D. M. Frangopol, "Lifetime-oriented multi-objective optimization of structural maintenance considering system reliability, redundancy and life-cycle cost using GA," *Structural Safety,* vol. 31, pp. 460-474, 2009.

[267] A. D. Orcesi and D. M. Frangopol, "Optimization of bridge maintenance strategies based on structural health monitoring information," *Structural Safety,* vol. 33, pp. 26-41, 2010.

[268] D. Nielsen, D. Raman, and G. Chattopadhyay, "Life cycle management for railway bridge assets," *Proceedings of the Institution of Mechanical Engineers, Part F: Journal of Rail and Rapid Transit,* vol. 227, pp. 570-581, 2013.

[269] G. Barone and D. M. Frangopol, "Life-cycle maintenance of deteriorating structures by multi-objective optimization involving reliability, risk, availability, hazard and cost," *Structural Safety,* vol. 48, pp. 40-50, 2014.

[270] M. Soliman and D. M. Frangopol, "Life-cycle cost evaluation of conventional and corrosion-resistant steel for bridges," *Journal of Bridge Engineering,* vol. 20, p. 06014005, 2014.

[271] S. Kim and D. M. Frangopol, "Efficient multi-objective optimisation of probabilistic service life management," *Structure and Infrastructure Engineering,* vol. 13, pp. 147-159, 2017.

[272] H. Gervásio and L. S. da Silva, "Life-cycle social analysis of motorway bridges," *Structure and Infrastructure Engineering,* vol. 9, pp. 1019-1039, 2013.

[273] I. J. Navarro, V. Yepes, and J. V. Martí, "Social life cycle assessment of concrete bridge decks exposed to aggressive environments," *Environmental Impact Assessment Review,* vol. 72, pp. 50-63, 2018.

[274] H. Gervásio and L. S. e. da Silva, "Comparative life-cycle analysis of steel-concrete composite bridges," *Structure and Infrastructure Engineering,* vol. 4, pp. 251-269, 2008.

[275] J. Hammervold, M. Reenaas, and H. Brattebø, "Environmental life cycle assessment of bridges," *Journal of Bridge Engineering,* vol. 18, pp. 153-161, 2013.

[276] T. García-Segura, V. Yepes, D. M. Frangopol, and D. Y. Yang, "Lifetime reliability-based optimization of post-tensioned box-girder bridges," *Engineering Structures,* vol. 145, pp. 381-391, 2017.

[277] V. Penadés-Plà, J. V. Martí, T. García-Segura, and V. Yepes, "Life-cycle assessment: A comparison between two optimal post-tensioned concrete box-girder road bridges," *Sustainability,* vol. 9, p. 1864, 2017.

[278] K. Steele, G. Cole, G. Parke, B. Clarke, and J. Harding, "Highway bridges and environment—Sustainable perspectives," in *Proceedings of the Institution of Civil Engineers-Civil Engineering*, 2003, pp. 176-182.

[279] A. J. Martin, "Concrete bridges in sustainable development," in *Proceedings of the Institution of Civil Engineers-Engineering Sustainability*, 2004, pp. 219-230.

[280] G. A. Keoleian, A. Kendall, J. E. Dettling, V. M. Smith, R. F. Chandler, M. D. Lepech*, et al.*, "Life cycle modeling of concrete bridge design: Comparison of engineered cementitious

composite link slabs and conventional steel expansion joints," *Journal of infrastructure systems,* vol. 11, pp. 51-60, 2005.

[281] D. Collings, "An environmental comparison of bridge forms," in *Proceedings of the Institution of Civil Engineers-Bridge Engineering*, 2006, pp. 163-168.

[282] G. Du and R. Karoumi, "Life cycle assessment of a railway bridge: comparison of two superstructure designs," *Structure and Infrastructure Engineering,* vol. 9, pp. 1149-1160, 2013.

[283] J. N. Rodrigues, P. Providência, and A. M. Dias, "Sustainability and lifecycle assessment of timber-concrete composite bridges," *Journal of Infrastructure Systems,* vol. 23, p. 04016025, 2016.

[284] K. F. Bizjak, A. Šajna, K. Slanc, and F. Knez, "Environmental life cycle assessment of railway bridge materials using UHPFRC," *Materials and Geoenvironment,* vol. 63, pp. 183-198, 2016.

[285] Z. Lounis and L. Daigle, "Environmental benefits of life cycle design of concrete bridges," in *Proceedings of the 3rd International Conference on Life Cycle Management, Zurich, Switzerland*, 2007, pp. 1-6.

[286] L. Bouhaya, R. Le Roy, and A. I. d. Feraille-Fresnet, "Simplified environmental study on innovative bridge structure," *Environmental science & technology,* vol. 43, pp. 2066-2071, 2009.

[287] Y. Itoh, M. Wada, and C. Liu, "Lifecycle environmental impact and cost analyses of steel bridge piers with seismic risk," in *9th international conference on structural safety and reliability, Rome, Italy*, 2005, pp. 1581-1588.

[288] J. E. Padgett and C. Tapia, "Sustainability of natural hazard risk mitigation: Life cycle analysis of environmental indicators for bridge infrastructure," *Journal of Infrastructure Systems,* vol. 19, pp. 395-408, 2013.

[289] L. Costa, "Algoritmos evolucionários em optimização uni e multi-objectivo (translation: Evolutionary Algorithms in uni and multi-objective optimization)," 2003.

[290] S. S. Rao, "Engineering optimization: theory and practice 3rd ed., New York: John Wiley," 1996.

[291] R. L. Cheu, Y. Wang, and T. F. Fwa, "Genetic Algorithm-Simulation Methodology for Pavement Maintenance Scheduling," *Computer-Aided Civil and Infrastructure Engineering,* vol. 19, pp. 446-455, 2004.

[292] D. E. Golberg, "Genetic algorithms in search, optimization, and machine learning," *Addion wesley,* vol. 1989, 1989.

[293] K. Deb, "An introduction to genetic algorithms," *Sadhana,* vol. 24, pp. 293-315, 1999.

[294] A. V. Moreira, "Development of an optimization methodology for pavement management systems," Department of Civil Engineering, University of Minho, PhD Thesis, 2018.

[295] D. Methods. (2012). *Multiobjective optimization of a laminar transonic airfoil for swept wing*. Available: http://www.designmethods.eu/en/?portfolio=multiobjective-optimization-of-a-laminar-transonic-nlf-for-a-swept-wing

[296] A. Zhou, B.-Y. Qu, H. Li, S.-Z. Zhao, P. N. Suganthan, and Q. Zhang, "Multiobjective evolutionary algorithms: A survey of the state of the art," *Swarm and Evolutionary Computation,* vol. 1, pp. 32-49, 2011.

[297] Q. Zhang and H. Li, "MOEA/D: A multiobjective evolutionary algorithm based on decomposition," *IEEE Transactions on evolutionary computation,* vol. 11, pp. 712-731, 2007.

[298] Y. Yusoff, M. S. Ngadiman, and A. M. Zain, "Overview of NSGA-II for optimizing machining process parameters," *Procedia Engineering,* vol. 15, pp. 3978-3983, 2011.

[299] K. Deb, *Multi-objective optimization using evolutionary algorithms* vol. 16: John Wiley & Sons, 2001.

[300] H. Li and Q. Zhang, "Multiobjective optimization problems with complicated Pareto sets, MOEA/D and NSGA-II," *IEEE transactions on evolutionary computation,* vol. 13, pp. 284-302, 2008.

[301] L. Costa, "Multi-objective optimization: Lecture notes to the course of multi-objective optimization for the graduation in Applied Math - Operational Research - 4th year," Departament of Production and Systems, School of Engineering, University of Minho, Braga, 2013.

[302] P. Bocchini and D. M. Frangopol, "A probabilistic computational framework for bridge network optimal maintenance scheduling," *Reliability Engineering & System Safety,* vol. 96, pp. 332-349, 2011.

[303] S. Kim and D. M. Frangopol, "Cost-based optimum scheduling of inspection and monitoring for fatigue-sensitive structures under uncertainty," *Journal of Structural Engineering,* vol. 137, pp. 1319-1331, 2010.

[304] S. Kim, D. M. Frangopol, and M. Soliman, "Generalized probabilistic framework for optimum inspection and maintenance planning," *Journal of Structural Engineering,* vol. 139, pp. 435-447, 2013.

[305] S. Kim and D. M. Frangopol, "Inspection and monitoring planning for RC structures based on minimization of expected damage detection delay," *Probabilistic Engineering Mechanics,* vol. 26, pp. 308-320, 2011.

[306] S. M. Soliman and D. M. Frangopol, "Life-cycle management of fatigue-sensitive structures integrating inspection information," *Journal of Infrastructure Systems,* vol. 20, p. 04014001, 2013.

[307] R. Denysiuk, A. V. Moreira, J. C. Matos, J. R. Oliveira, and A. Santos, "Two-stage multiobjective optimization of maintenance scheduling for pavements," *Journal of Infrastructure Systems,* vol. 23, p. 04017001, 2017.

[308] C. Torres-Machi, E. Pellicer, V. Yepes, and A. Chamorro, "Towards a sustainable optimization of pavement maintenance programs under budgetary restrictions," *Journal of cleaner production,* vol. 148, pp. 90-102, 2017.

[309] C. Tapia and J. E. Padgett, "Multi-objective optimisation of bridge retrofit and post-event repair selection to enhance sustainability," *Structure and Infrastructure Engineering,* vol. 12, pp. 93-107, 2016.

[310] V. Penadés-Plà, T. García-Segura, J. V. Martí, and V. Yepes, "An optimization-LCA of a prestressed concrete precast bridge," *Sustainability,* vol. 10, p. 685, 2018.

[311] T. García-Segura and V. Yepes, "Multiobjective optimization of post-tensioned concrete box-girder road bridges considering cost, CO2 emissions, and safety," *Engineering Structures,* vol. 125, pp. 325-336, 2016.

[312] T. García-Segura, V. Yepes, and D. M. Frangopol, "Multi-objective design of post-tensioned concrete road bridges using artificial neural networks," *Structural and Multidisciplinary Optimization,* vol. 56, pp. 139-150, 2017.

[313] A. Mondoro, D. M. Frangopol, and M. Soliman, "Optimal risk-based management of coastal bridges vulnerable to hurricanes," *Journal of Infrastructure Systems,* vol. 23, p. 04016046, 2016.

[314] S. Kim and D. M. Frangopol, "Decision making for probabilistic fatigue inspection planning based on multi-objective optimization," *International Journal of Fatigue,* vol. 111, pp. 356-368, 2018.

[315] D. M. Frangopol, "Life-cycle performance, management, and optimisation of structural systems under uncertainty: accomplishments and challenges 1," *Structure and Infrastructure Engineering,* vol. 7, pp. 389-413, 2011.

[316] MAtlab, "Optimization Toolbox," 2016.

[317] B. Åkesson, *Understanding bridge collapses*: CRC Press, 2008.

[318] J. Scheer, *Failed bridges: case studies, causes and consequences*: John Wiley & Sons, 2010.

[319] G. C. Lee, S. Mohan, C. Huang, and B. N. Fard, "A Study of US Bridge Failures (1980-2012)," MCEER Technical Report 13-0008, Buffalo, NY,2013.

[320] M. A. Khan, *Bridge and highway structure rehabilitation and repair*, 2010.

[321] M.Knott, "Vessel Collision Design - Risk analysis and deep foundation issues for bridges over navigable waterways. Technical Report May 2003, Moffatt & Nichol, 2005.," 2005.

[322] N. Rail. (2019). *Prevention of bridge strikes*. Available: https://www.networkrail.co.uk/running-the-railway/looking-after-the-railway/bridges-tunnels-and-viaducts/the-risk-of-bridge-strikes/prevention-of-bridge-strikes/

[323] E. Figueiredo, "Damage identification in civil engineering infrastructure under operational and environmental conditions," Faculty of Engineering of University of Porto, PhD Thesis, 2012.

[324] C. S. Holling, "Resilience and stability of ecological systems," *Annual review of ecology and systematics,* vol. 4, pp. 1-23, 1973.

[325] M. Bruneau, S. E. Chang, R. T. Eguchi, G. C. Lee, T. D. O'Rourke, A. M. Reinhorn*, et al.*, "A framework to quantitatively assess and enhance the seismic resilience of communities," *Earthquake spectra,* vol. 19, pp. 733-752, 2003.

[326] E. P. Dalziell and S. T. McManus, "Resilience, vulnerability, and adaptive capacity: implications for system performance," 2004.

[327] S. E. Chang and M. Shinozuka, "Measuring improvements in the disaster resilience of communities," *Earthquake spectra,* vol. 20, pp. 739-755, 2004.

[328] T. McDaniels, S. Chang, D. Cole, J. Mikawoz, and H. Longstaff, "Fostering resilience to extreme events within infrastructure systems: Characterizing decision contexts for mitigation and adaptation," *Global Environmental Change,* vol. 18, pp. 310-318, 2008.

[329] G. P. Cimellaro, C. Fumo, A. M. Reinhorn, and M. Bruneau, *Quantification of disaster resilience of health care facilities*: MCEER Buffalo, NY, 2009.

[330] G. P. Cimellaro, A. M. Reinhorn, and M. Bruneau, "Framework for analytical quantification of disaster resilience," *Engineering structures,* vol. 32, pp. 3639-3649, 2010.

[331] M. Ouyang, L. Dueñas-Osorio, and X. Min, "A three-stage resilience analysis framework for urban infrastructure systems," *Structural safety,* vol. 36, pp. 23-31, 2012.

[332] K. Barker, J. E. Ramirez-Marquez, and C. M. Rocco, "Resilience-based network component importance measures," *Reliability Engineering & System Safety,* vol. 117, pp. 89-97, 2013.

[333] R. Francis and B. Bekera, "A metric and frameworks for resilience analysis of engineered and infrastructure systems," *Reliability Engineering & System Safety,* vol. 121, pp. 90-103, 2014.

[334] K. Adjetey-Bahun, B. Birregah, E. Châtelet, and J.-L. Planchet, "A model to quantify the resilience of mass railway transportation systems," *Reliability Engineering & System Safety,* vol. 153, pp. 1-14, 2016.

[335] A. D. González, L. Dueñas-Osorio, M. Sánchez-Silva, and A. L. Medaglia, "The interdependent network design problem for optimal infrastructure system restoration," *Computer-Aided Civil and Infrastructure Engineering,* vol. 31, pp. 334-350, 2016.

[336] P. Lin and N. Wang, "Stochastic post-disaster functionality recovery of community building portfolios I: Modeling," *Structural Safety,* vol. 69, pp. 96-105, 2017.

[337] P. Lin and N. Wang, "Stochastic post-disaster functionality recovery of community building portfolios II: Application," *Structural Safety,* vol. 69, pp. 106-117, 2017.

[338] A. Barabadi and Y. Z. Ayele, "Post-disaster infrastructure recovery: Prediction of recovery rate using historical data," *Reliability Engineering & System Safety,* vol. 169, pp. 209-223, 2018.

[339] J. E. Padgett and R. DesRoches, "Bridge functionality relationships for improved seismic risk assessment of transportation networks," *Earthquake Spectra,* vol. 23, pp. 115-130, 2007.

[340] P. Bocchini and D. M. Frangopol, "Optimal resilience-and cost-based postdisaster intervention prioritization for bridges along a highway segment," *Journal of Bridge Engineering,* vol. 17, pp. 117-129, 2012.

[341] W. Lokuge and S. Setunge, "Evaluating disaster resilience of bridge infrastructure when exposed to extreme natural events," *Proceedings of the 3rd International Conferencon Building Resilience,* pp. 1-12, 2013.

[342] A. Deco, P. Bocchini, and D. M. Frangopol, "A probabilistic approach for the prediction of seismic resilience of bridges," *Earthquake Engineering & Structural Dynamics,* vol. 42, pp. 1469-1487, 2013.

[343] S. Chandrashekaran and S. Banerjee, "Optimal Retrofit Strategy for Disaster Resilience of Highway Bridges," in *10th US National Conference on Earthquake Engineering*, 2014, pp. 21-25.

[344] A. Venkittaraman and S. Banerjee, "Enhancing resilience of highway bridges through seismic retrofit," *Earthquake Engineering & Structural Dynamics,* vol. 43, pp. 1173-1191, 2014.

[345] Y. Dong and D. M. Frangopol, "Risk and resilience assessment of bridges under mainshock and aftershocks incorporating uncertainties," *Engineering Structures,* vol. 83, pp. 198-208, 2015.

[346] A. Karamlou and P. Bocchini, "Computation of bridge seismic fragility by large-scale simulation for probabilistic resilience analysis," *Earthquake Engineering & Structural Dynamics,* vol. 44, pp. 1959-1978, 2015.

[347] G. Cimellaro, "Resilience-based design (RBD) modelling of civil infrastructure to assess seismic hazards," in *Handbook of Seismic Risk Analysis and Management of Civil Infrastructure Systems*, ed: Elsevier, 2013, pp. 268-303.

[348] C. Kafali and M. Grigoriu, "Rehabilitation decision analysis," in *Proceedings of the Ninth International Conference on Structural Safety and Reliability (ICOSSAR'05)*, 2005.

[349] M. P. S. Ariza, H. S. Sousa, J. N. D. Fernandes, and J. C. Matos, "Reliability analysis of a post-tensioned railway bridge exposed to corrosion effects," 2019.

[350] CEN, "Eurocode 2: Design of concrete structures - Part 1-1 : General rules and rules for buildings ", ed, 2004.

[351] M. Dolce, D. Cardone, and F. Croatto, "Frictional behavior of steel-PTFE interfaces for seismic isolation," *Bulletin of earthquake engineering,* vol. 3, pp. 75-99, 2005.

[352] Matlab, "Design of Experiments Toolbox," ed, 2016.

[353] A. Abel Henriques, "Aplicação de novos conceitos de segurança no dimensionamento do betão estrutural (translation: Application of new safety concepts on the structural concrete design)," Department of Civil Engineering, University of Porto, PhD Thesis, 1998.

[354] G. M. Luís C.Silva, Paulo B. Lourenço, "A probabilistic framework using discrete FE-based homogenized model for the in-and out-of-plane analysis of masonry structures," *Conference: 7th International Conference on Computational Methods in Structural Dynamics and Earthquake Engineering Methods in Structural Dynamics and Earthquake Engineering,* 2019.

[355] CEN, "Eurocode 8. Design provisions for earthquake resistance of structures. Part 1-1: General rules – seismic actions and general requirements for structures. Brussels, Belgium Eur Commitee Stand," ed, 2004.

[356] Gelfi, "SIMQKE-GR - Software for generating artificial accelerograms compatible with the response spectrum," 2006.

[357] K. Pitilakis, H. Crowley, and A. M. Kaynia, "SYNER-G: typology definition and fragility functions for physical elements at seismic risk," *Geotechnical, Geological and Earthquake Engineering,* vol. 27, pp. 1-28, 2014.

[358] A. Lupoi, P. Franchin, P. Pinto, and G. Monti, "Seismic design of bridges accounting for spatial variability of ground motion," *Earthquake engineering & structural dynamics,* vol. 34, pp. 327-348, 2005.

[359] K. Mackie and B. Stojadinovic, "Improving probabilistic seismic demand models through refined intensity measures," in *Proc. 13th World Conf. Earthquake Eng. Int. Assoc. for Earthquake Eng. Japan*, 2004.

[360] J. E. Padgett, B. G. Nielson, and R. DesRoches, "Selection of optimal intensity measures in probabilistic seismic demand models of highway bridge portfolios," *Earthquake engineering & structural dynamics,* vol. 37, pp. 711-725, 2008.

[361] O. Avsar and A. Yakut, "Evaluation of ground motion intensity measures for the fragility curves of ordinary highway bridges in Turkey," in *Proceedings of the 9th US national & 10th Canadian conference on earthquake engineering, Toronto*, 2010.

[362] FEMA, "Hazus MH MR5 technical manual. Federal Emergency Management Agency, Washington, DC," ed, 2010.

[363] CEN, "Eurocode 8: Design of structures for earthquake restistance - Part 3: Assessment and retrofiting of buildings," 2004.

[364] I. F. Moschonas, A. J. Kappos, P. Panetsos, V. Papadopoulos, T. Makarios, and P. Thanopoulos, "Seismic fragility curves for Greek bridges: methodology and case studies," *Bulletin of Earthquake Engineering,* vol. 7, p. 439, 2009.

[365] C. Jiang, B. Wei, D. Wang, L. Jiang, and X. He, "Seismic vulnerability evaluation of a three-span continuous beam railway bridge," *Mathematical Problems in Engineering,* vol. 2017, 2017.

[366] F. industriale. (2020). Available: https://www.fipindustriale.it/

[367] J. W. Baker, "Efficient analytical fragility function fitting using dynamic structural analysis," *Earthquake Spectra,* vol. 31, pp. 579-599, 2014.

[368] E. Choi, R. DesRoches, and B. Nielson, "Seismic fragility of typical bridges in moderate seismic zones," *Engineering structures,* vol. 26, pp. 187-199, 2004.

[369] J. Zhang and Y. Huo, "Evaluating effectiveness and optimum design of isolation devices for highway bridges using the fragility function method," *Engineering Structures,* vol. 31, pp. 1648-1660, 2009.

[370] J. Ghosh and J. E. Padgett, "Aging considerations in the development of time-dependent seismic fragility curves," *Journal of Structural Engineering,* vol. 136, pp. 1497-1511, 2010.

[371] M. Sánchez-Silva and R. Rackwitz, "Socioeconomic implications of life quality index in design of optimum structures to withstand earthquakes," *Journal of Structural Engineering,* vol. 130, pp. 969-977, 2004.

[372] B. Zhu and D. M. Frangopol, "Risk-based approach for optimum maintenance of bridges under traffic and earthquake loads," *Journal of Structural Engineering,* vol. 139, pp. 422-434, 2012.

[373] D. M. Frangopol and M. Soliman, "Life-cycle of structural systems: recent achievements and future directions," *Structure and infrastructure engineering,* vol. 12, pp. 1-20, 2016.

Annex

Annex A – Chapter 2

This annex is referred to the questionnaire provided by the IABMAS report in the different bridge management systems.

Table A.1 - Performance indicators adopted in the different bridge management systems

		Performance Indicators				
Designation	**Country**	**Condition**	**Load carrying capacity**	**Safety**	**Risk**	**Condition States (CSs)**
MRWA	Australia	✓	×	×	×	-
NSW	Australia	✓	×	✓	✓	4
OBMS	Canada	✓	✓	✓	✓	4
QBMS	Canada	✓	✓	✓	✓	4
eBMS	Canada	✓	✓	✓	✓	5
PEI BMS	Canada	✓	✓	✓	✓	4
GNWT	Canada	✓	✓	✓	✓	4
DANBRO	Denmark	✓	×	×	×	5
FBMS	Finland	✓	✓	✓	✓	4
GBMS	Germany	✓	✓	✓	✓	4
Eirspan	Ireland	✓	✓	✓	✓	4
APTBMS	Italy	✓	✓	✓	✓	4
RPIBMS	Japan	✓	✓	✓	✓	4
KRBMS	Korea	✓	✓	✓	✓	4
Lat Brutus	Latvia	✓	✓	✓	✓	4
DISK	Netherlands	✓	✓	✓	✓	6
BRUTUS	Norway	✓	✓	✓	✓	4
SMOK	Poland	✓	✓	✓	✓	4
SZOK	Poland	✓	✓	✓	✓	5
SGP	Spain	✓	×	✓	✓	4
BatMan	Sweden	✓	✓	✓	✓	3
KUBA	Switzerland	✓	✓	✓	×	5
ABIMS	USA	✓	✓	✓	×	9
AASHTOWare	USA	✓	×	✓	✓	5
Bridgeman	Vietnam	✓	✓	✓	✓	-

Table A.2 - Prediction models adopted in the bridge management systems

		Predictive Capabilities			Planning time frames (years)	
Designation	**Country**	**Deterioration Model**	**Improvement**	**Optimal Intervention Strategies (IOS)**	**Short Term**	**Long Term**
MRWA	Australia	Prob and Det	✓	✗	6 to 10	N/A
NSW	Australia	Prob and Det	✗	✓	N/A	N/A
OBMS	Canada	Markovian	✓	✓	6 to 10	56 to 60
QBMS	Canada	Markovian	✓	✓	6 to 10	56 to 60
eBMS	Canada	Markovian	✓	✓	6 to 10	56 to 60
PEI BMS	Canada	Markovian	✓	✓	6 to 10	56 to 60
GNWT	Canada	Markovian	✓	✓	6 to 10	N/A
DANBRO	Denmark	Yes	✓	✗	6 to 10	N/A
FBMS	Finland	Yes	✓	✓	6 to 10	N/A
GBMS	Germany	Physical	✓	✓	6 to 10	16 to 20
Eirspan	Ireland	No	✗	✓	6 to 10	N/A
APTBMS	Italy	Markovian	✓	✓	1 to 5	46 to 50
RPIBMS	Japan	Speed curves	✓	✓	N/A	96 to 100
KRBMS	Korea	Regression	✓	✓	N/A	N/A
Lat Brutus	Latvia	Prob	✓	✓	N/A	N/A
DISK	Netherlands	N/A	✗	✗	6 to 10	N/A
BRUTUS	Norway	Prob and Det	✓	✓	6 to 10	N/A
SMOK	Poland	Yes	✗	✗	6 to 10	N/A
SZOK	Poland	Det	✓	✓	6 to 10	N/A
SGP	Spain	No	✗	✗	N/A	16 to 20
BatMan	Sweden	Det	✓	✓	16-20	N/A
KUBA	Switzerland	Markovian	✓	✓	1 to 5	96 to 100
ABIMS	USA	No	✗	✓	1 to 5	N/A
AASHTOWare	USA	Markovian	✓	✓	N/A	N/A
Bridgeman	Vietnam	No	✗	✗	N/A	N/A

Det – Deterministic Prob – Probabilistic N/A - Not Answered

Table A.3 - Different types of costs adopted in the bridge management systems

		Cost Information				
Designation	**Country**	**Inspection Cost**	**Intervention Cost**	**Traffic Delay Cost**	**Accident Cost**	**Environmental Costs**
MRWA	Australia	✗	✓	✗	✗	✗
NSW	Australia	✗	✓	✗	✗	✗
OBMS	Canada	✓	✓	✓	✓	✓
QBMS	Canada	✓	✓	✓	✗	✓
eBMS	Canada	✓	✓	✓	✓	✓
PEI BMS	Canada	✗	✓	✓	✓	✓
GNWT	Canada	✓	✓	✓	✓	✓
DANBRO	Denmark	✗	✓	✗	✗	✗
FBMS	Finland	✗	✓	✗	✗	✗
GBMS	Germany	✗	✓	✓	✓	✓
Eirspan	Ireland	✗	✓	✗	✗	✗
APTBMS	Italy	✓	✓	✗	✗	✗
RPIBMS	Japan	✗	✓	✓	✗	✓
KRBMS	Korea	✓	✓	✓	✗	✗
Lat Brutus	Latvia	✗	✓	✗	✗	✗
DISK	Netherlands	✗	✓	✗	✗	✗
BRUTUS	Norway	✗	✓	✓	✓	✓
SMOK	Poland	✗	✓	✗	✗	✗
SZOK	Poland	✗	✗	✗	✗	✗
SGP	Spain	✗	✓	✓	✗	✗
BatMan	Sweden	✗	✓	✓	✓	✗
KUBA	Switzerland	✗	✓	✗	✗	✗
ABIMS	USA	✗	✓	✗	✗	✗
AASHTOWare	USA	✗	✓	✗	✗	✗
Bridgeman	Vietnam	✗	✓	✗	✗	✗

Annex B – Chapter 3

This annex refers to the auxiliar calculations to buckling issues of the steel bridge.

Table B.1 – Auxiliar calculations

Bar	$\boldsymbol{\lambda_l}$	$\boldsymbol{i_x}$	$\boldsymbol{i_y}$	$\boldsymbol{L_{cr,x}}$	$\boldsymbol{L_{cr,y}}$
aB	98.815	0.327	0.335	7.560	5.292
BC		0.352	0.343	4.330	3.033
Cc		0.0537	0.101	6.200	4.340
CD		0.352	0.343	4.330	3.033
Dd		0.0536	0.101	6.200	4.340

Table B.2 – Auxiliar Calculations

Bar	$\boldsymbol{A_s}$	$\boldsymbol{\bar{\lambda}_x}$	$\boldsymbol{\bar{\lambda}_y}$	$\boldsymbol{\alpha_x}$	$\boldsymbol{\alpha_y}$
aB	0.0178	23.153	15.796	0.34	0.49
BC	0.0161	12.311	8.841		
Cc	0.009	115.518	42.905		
CD	0.0161	12.311	8.84		
Dd	0.009	115.518	42.905		

Table B-3 – Auxiliar Calculations

Bar	$\boldsymbol{\phi_{sx}}$	$\boldsymbol{\phi_{sy}}$	$\boldsymbol{\chi_x}$	$\boldsymbol{\chi_y}$	$\boldsymbol{\chi}$
aB	0.533	0.506	0.988	1.014	0.99
BC	0.495	0.477	1.027	1.058	1.00
Cc	1.348	0.652	0.495	0.879	0.50
CD	0.494	0.485	1.028	1.040	1.00
Dd	1.348	0.634	0.495	0.912	0.50

Annex C – chapter 7

This annex contains the annex C1 and C2 that refers to the Pareto front solutions, the risk and direct and indirect costs of maintenance without and with the retrofit action effect for the different damage limit state (DLS) and intensity measure (IM). Annex C3 refers to the time-dependent resilience estimation for the different IM.

Annex C1 – Pareto front, Risk and Costs without retrofit actions for different DLS

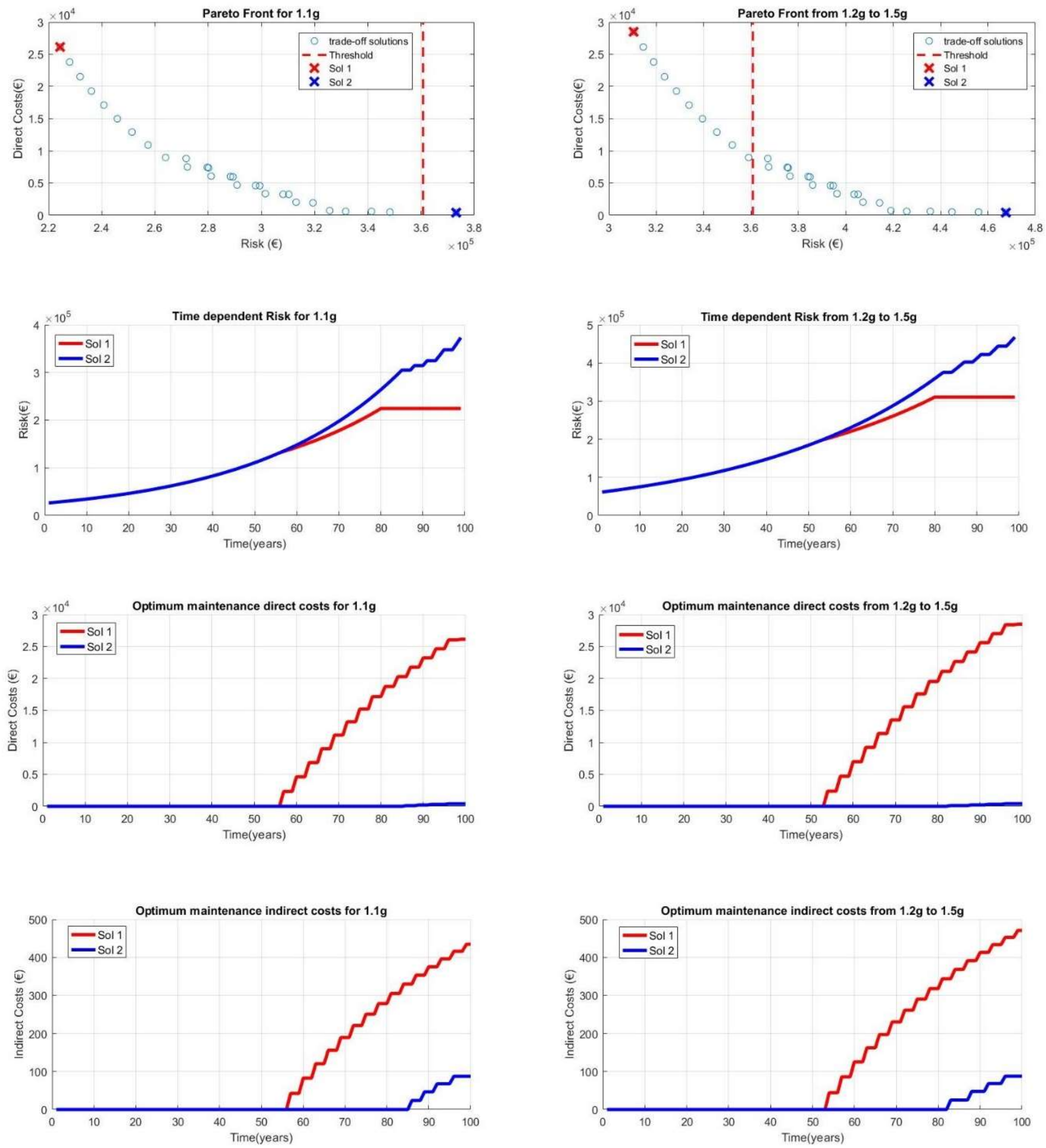


Figure C1.1 - DLS 2 for quantile 5 without considering retrofit action

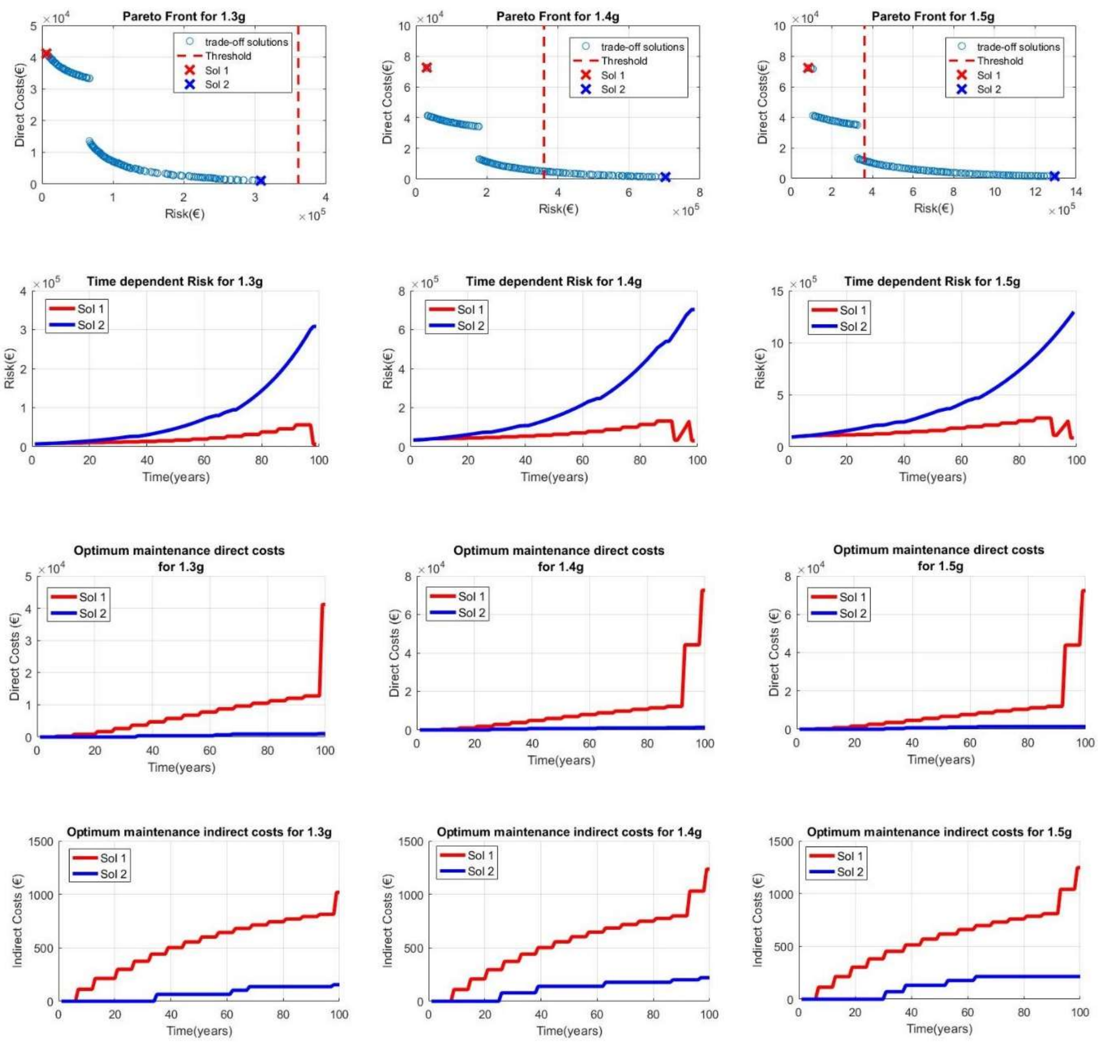


Figure C1.2 - DLS3 for quantile 5 without considering retrofit action

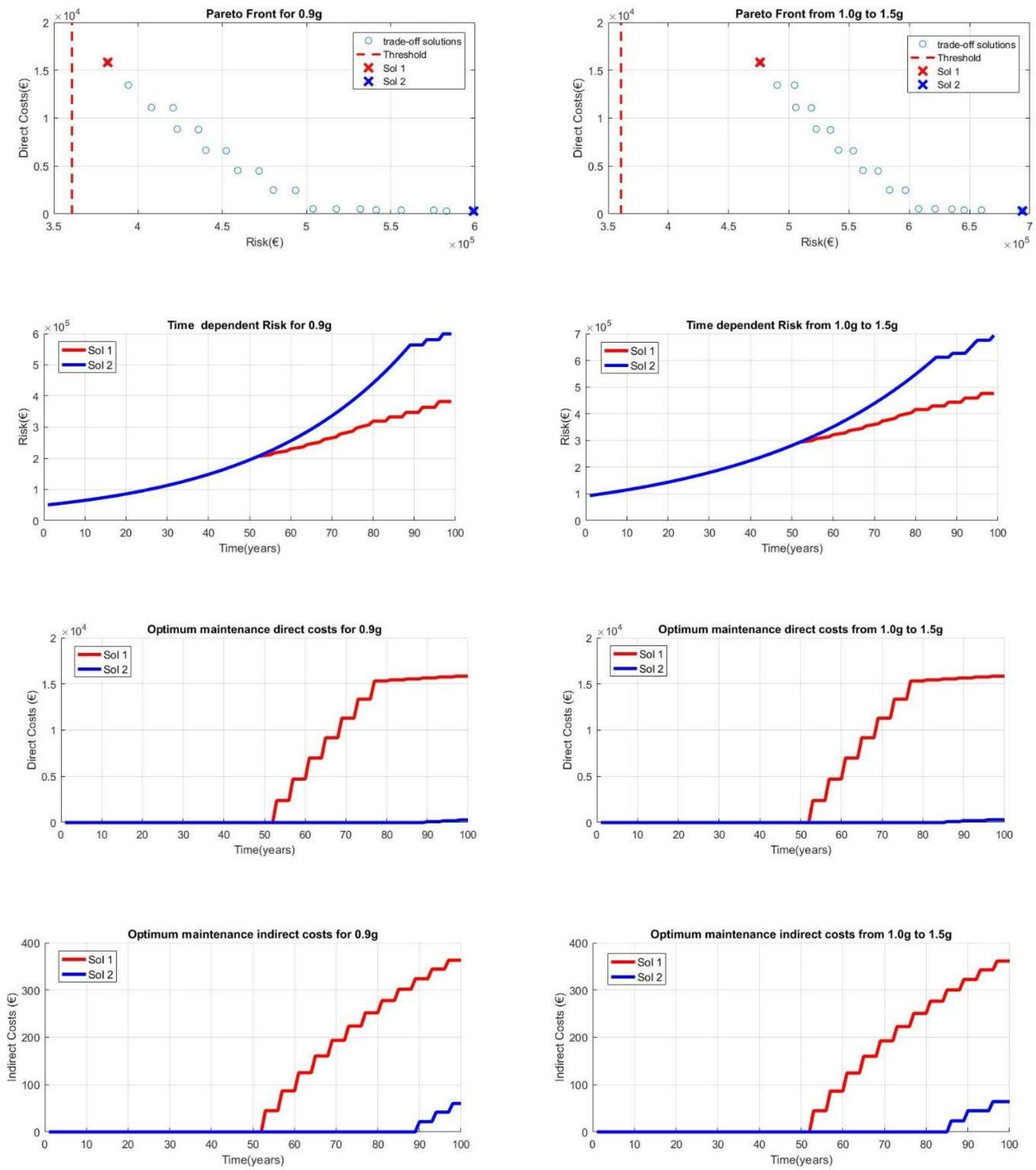


Figure C1.3 - Figure for DLS 2 mean wihtout considering retrofit action

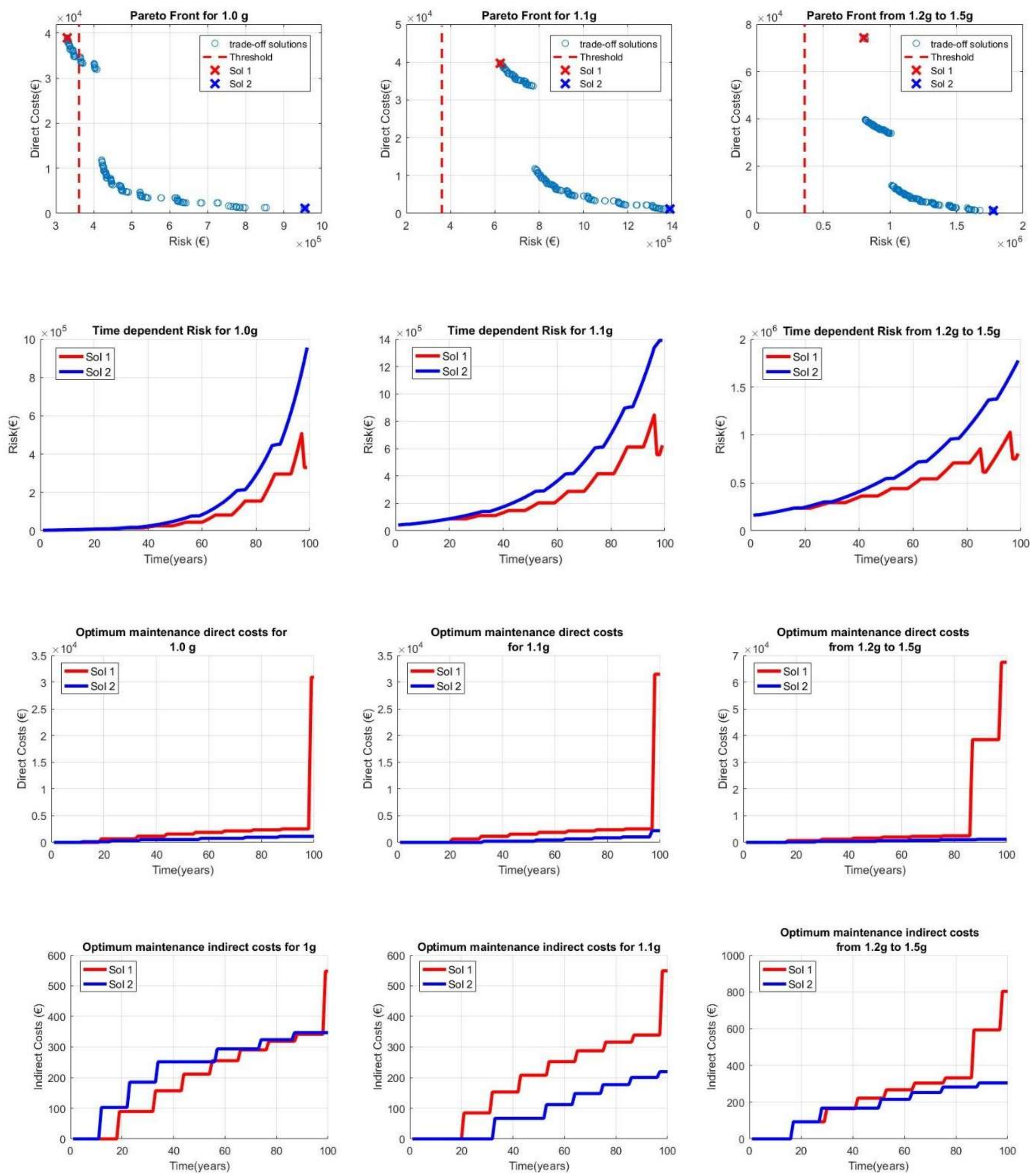
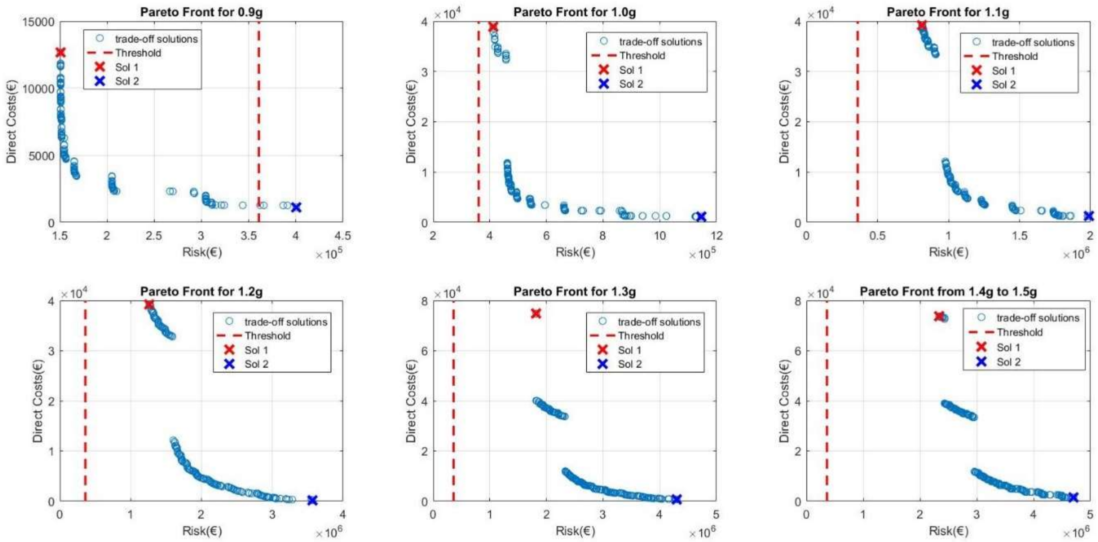
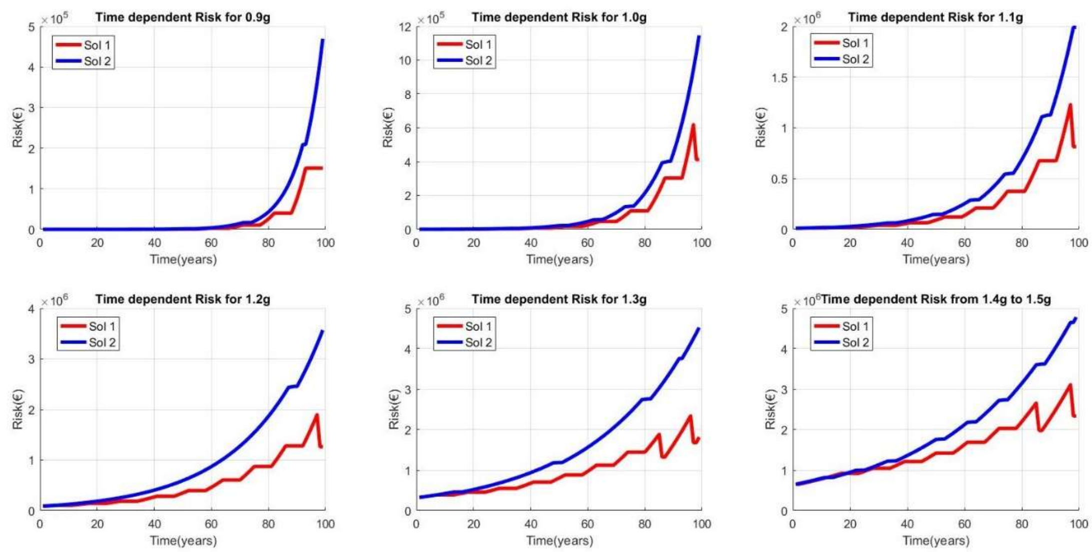


Figure C1.4 - Figure for DLS3 mean value without considering retrofit action


Pareto Front for 0.9g
Pareto Front for 1.0g
Pareto Front for 1.1g
Pareto Front for 1.2g
Pareto Front for 1.3g
Pareto Front from 1.4g to 1.5g
trade-off solutions
Threshold
Sol 1
Sol 2
Direct Costs(€)
Risk(€)


Time dependent Risk for 0.9g
Time dependent Risk for 1.0g
Time dependent Risk for 1.1g
Time dependent Risk for 1.2g
Time dependent Risk for 1.3g
Time dependent Risk from 1.4g to 1.5g
Sol 1
Sol 2
Risk(€)
Time(years)

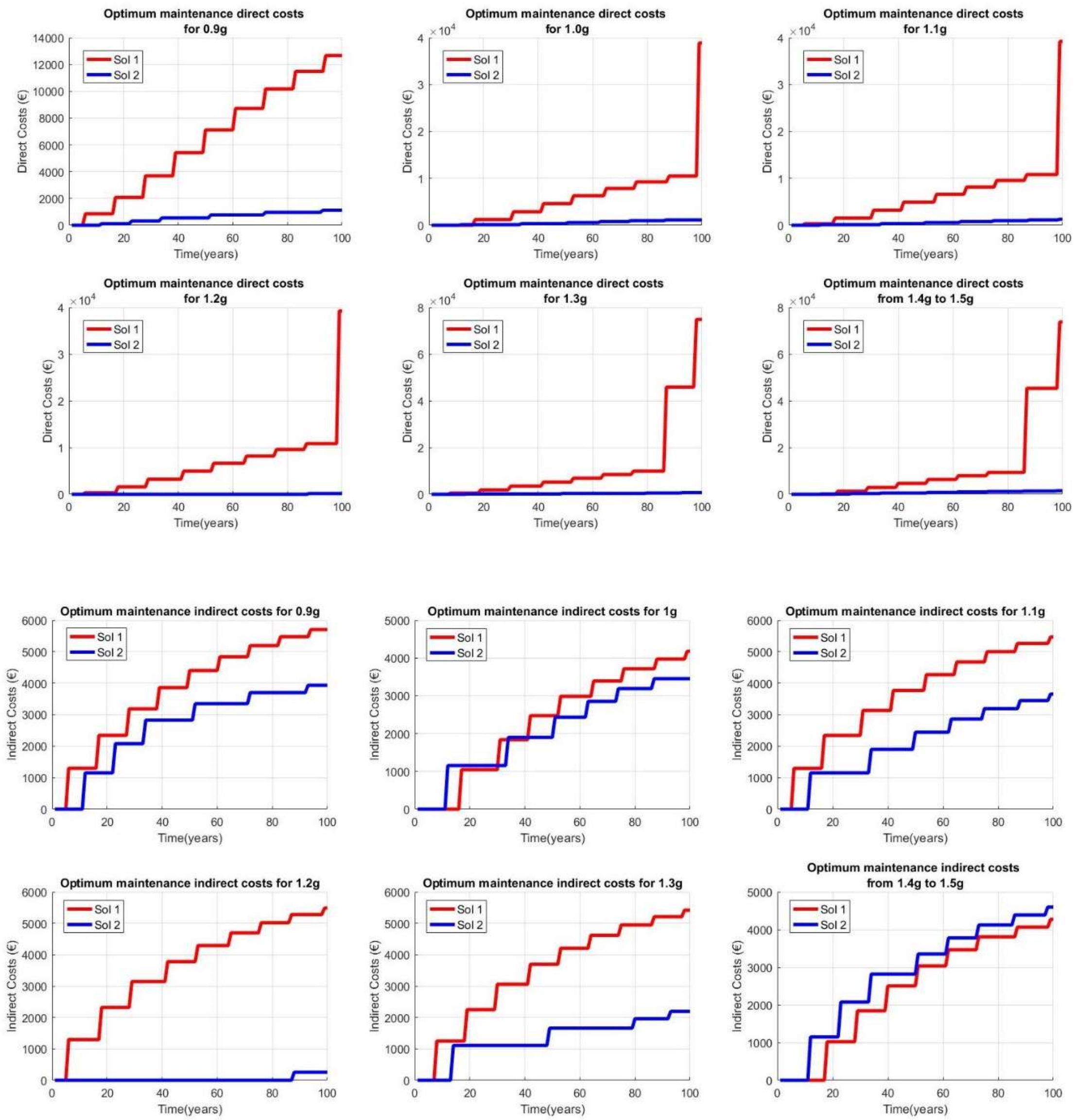


Figure C1.5 - DLS4 for mean value without considering retrofit action

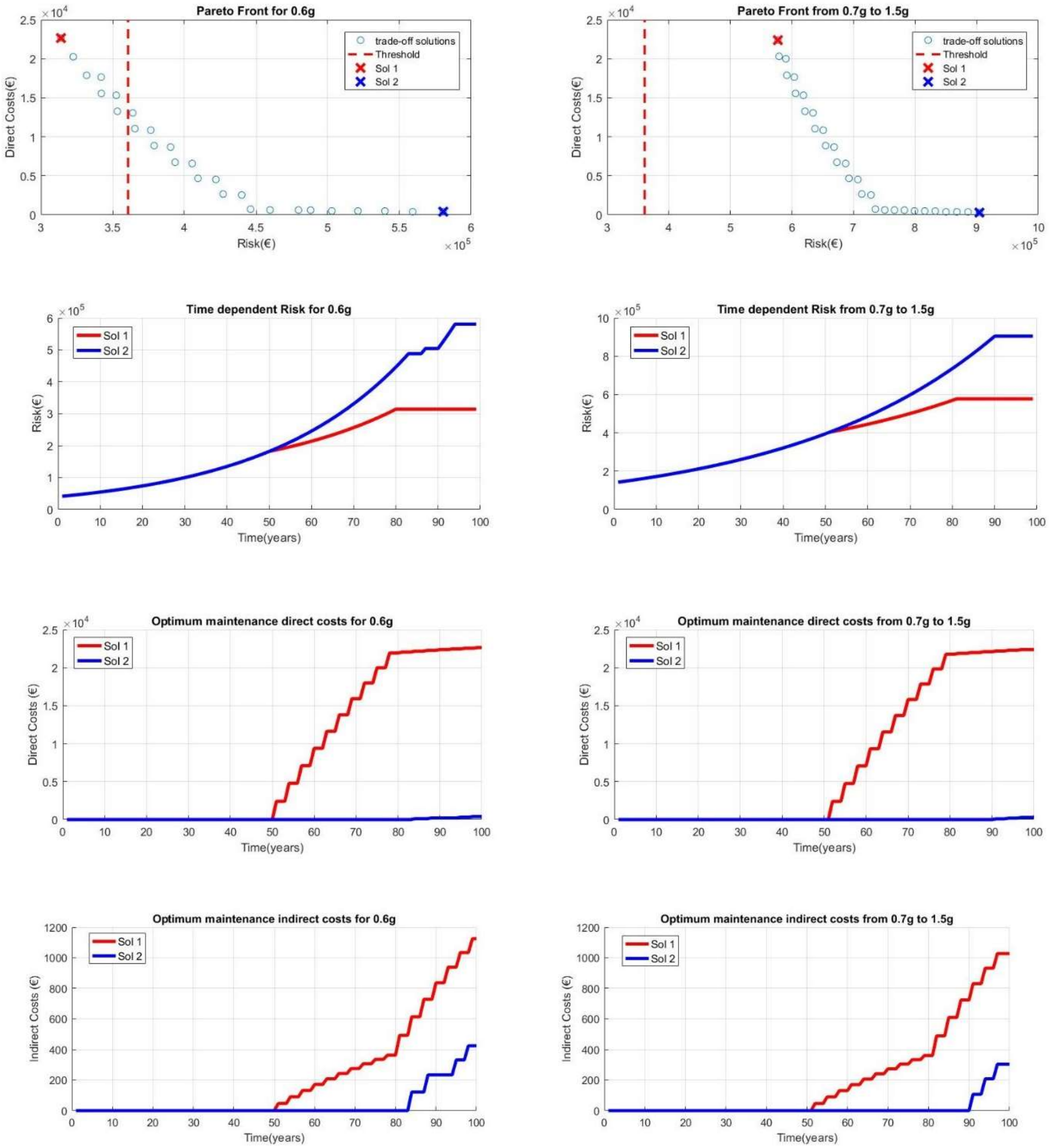


Figure C1.6 - DLS 2 for 95 without retrofit action

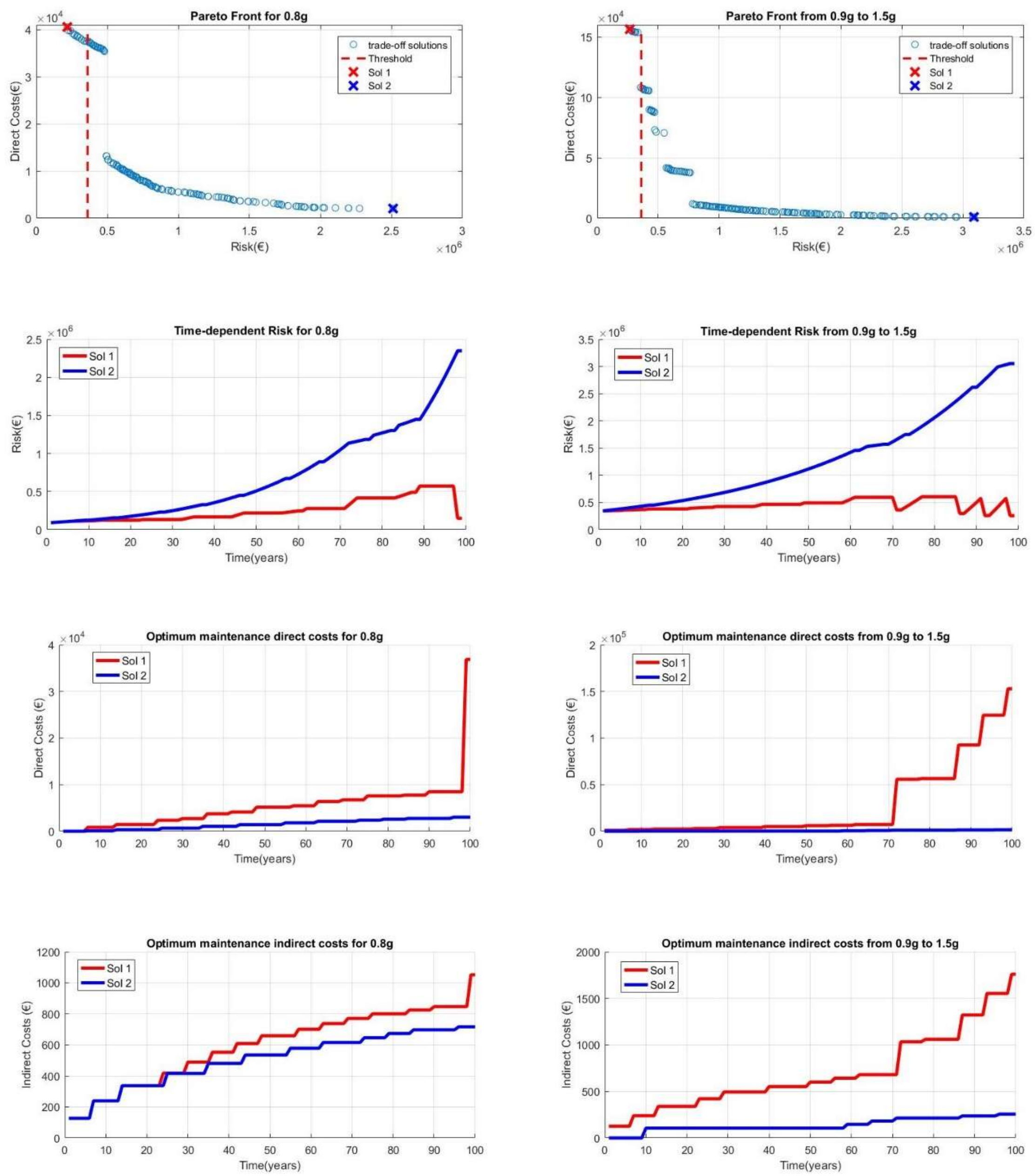


Figure C1.7 - DLS3 for 95 without retrofit action

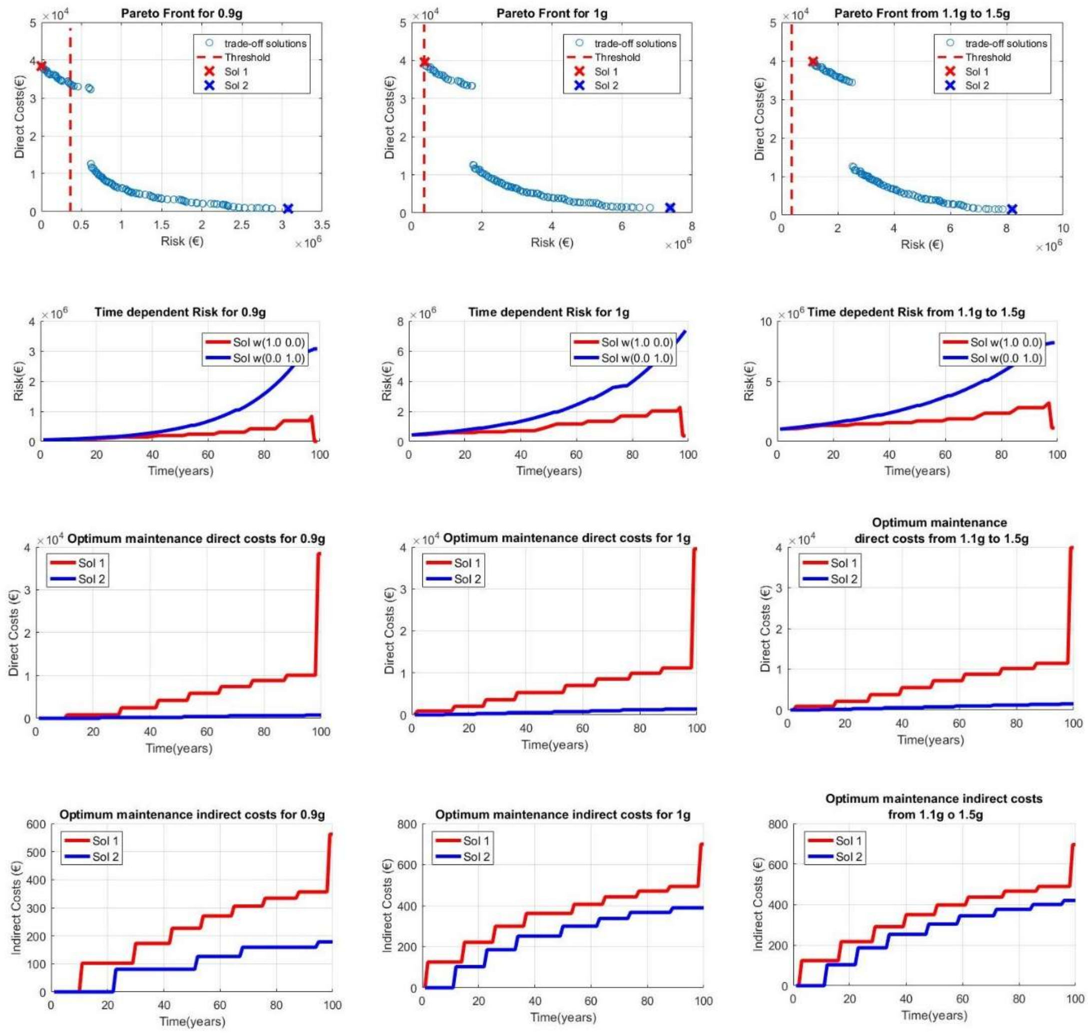


Figure C1.8 - DLS 4 for 95 without retrofit action

Annex C2 – Pareto front, Risk and Costs with retrofit action

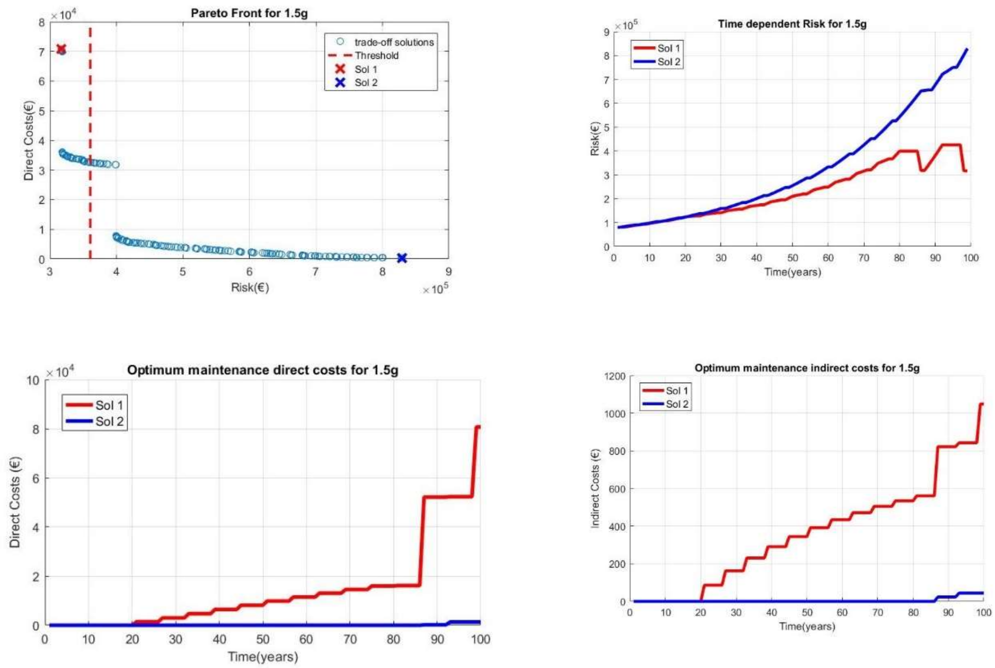


Figure C2.1 - DLS3 for mean value considering retrofit action

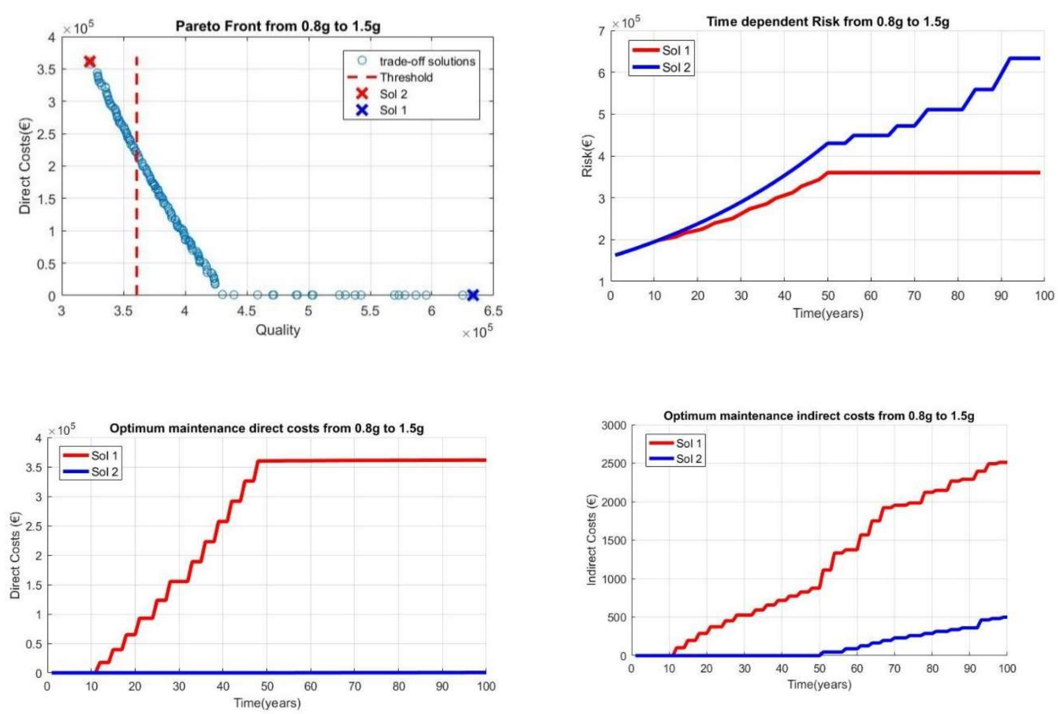


Figure C2.2 - DLS 2 for 95 quantile considering retrofit action

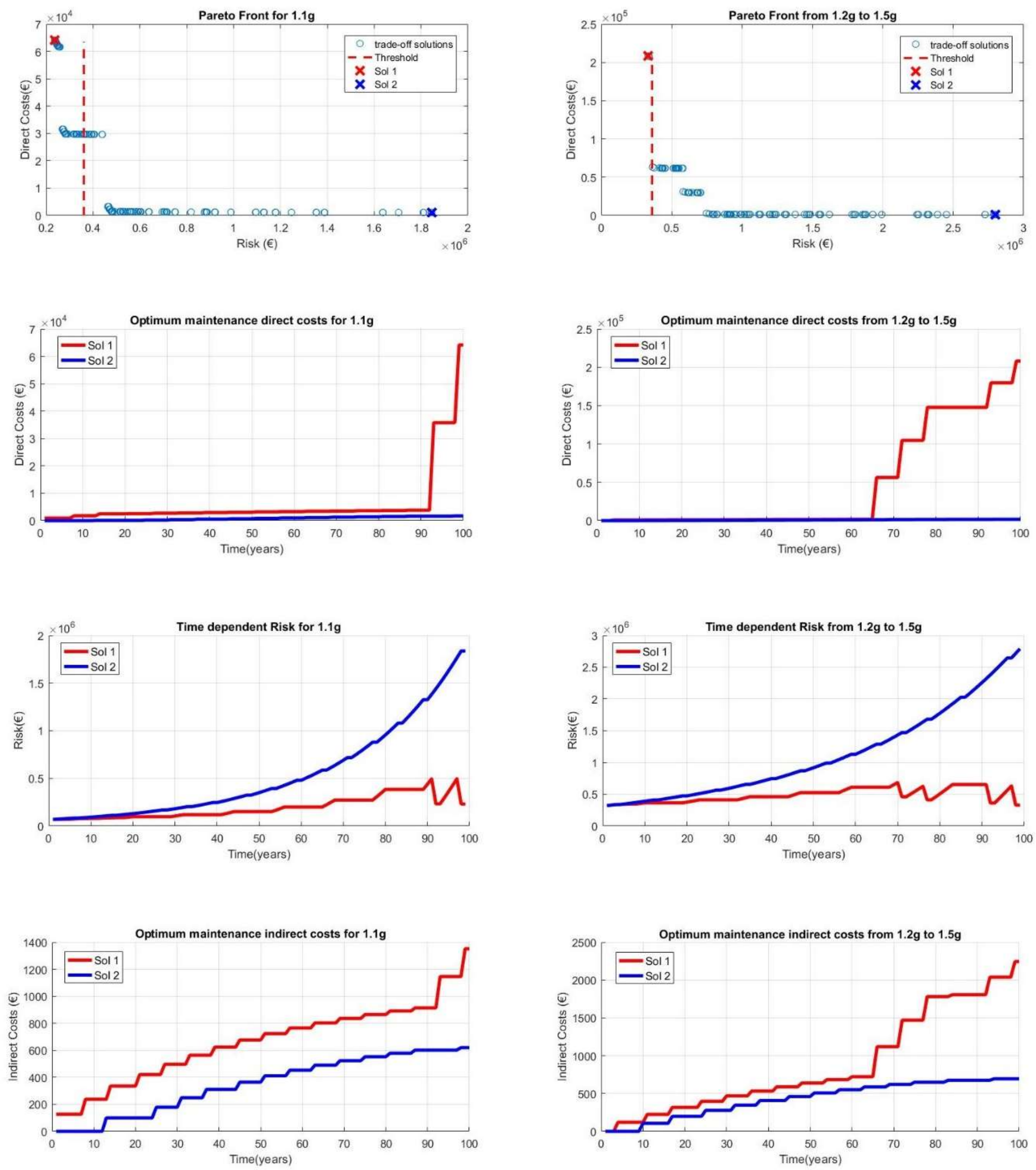
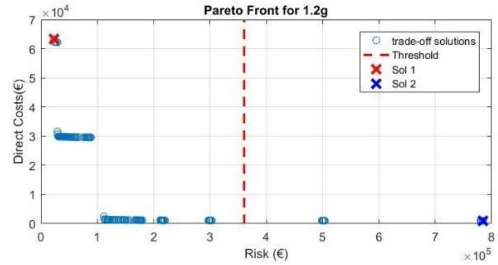
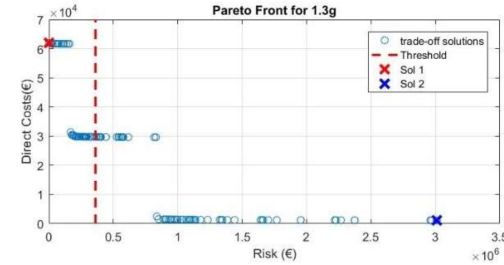
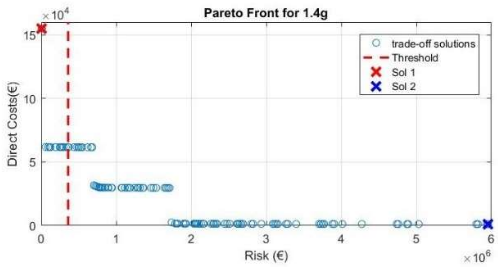
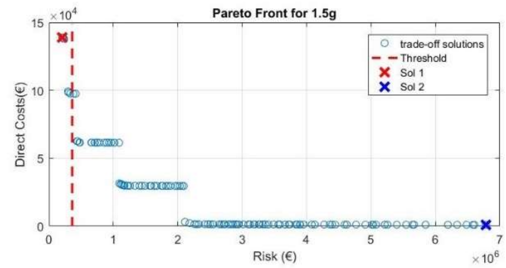


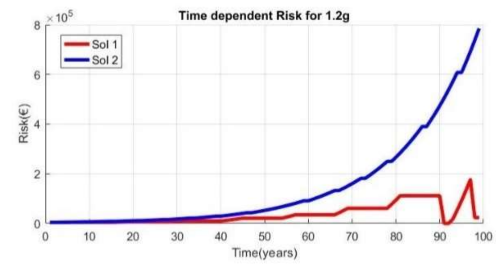
Figure C2.3 - DLS3 for 95 quantile considering retrofit action

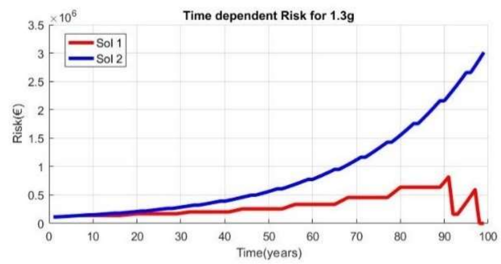

Pareto Front for 1.2g
Direct Costs(€)
Risk (€)
trade-off solutions
Threshold
Sol 1
Sol 2

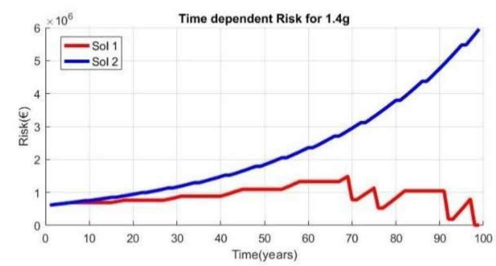

Pareto Front for 1.3g
Direct Costs(€)
Risk (€)
trade-off solutions
Threshold
Sol 1
Sol 2

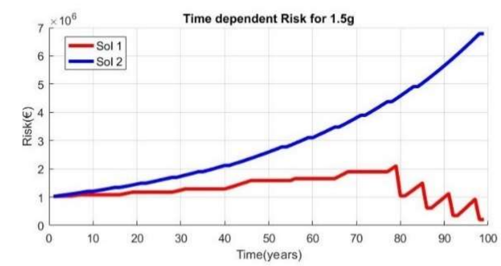

Pareto Front for 1.4g
Direct Costs(€)
Risk (€)
trade-off solutions
Threshold
Sol 1
Sol 2


Pareto Front for 1.5g
Direct Costs(€)
Risk (€)
trade-off solutions
Threshold
Sol 1
Sol 2


Time dependent Risk for 1.2g
Risk(€)
Time(years)
Sol 1
Sol 2


Time dependent Risk for 1.3g
Risk(€)
Time(years)
Sol 1
Sol 2


Time dependent Risk for 1.4g
Risk(€)
Time(years)
Sol 1
Sol 2


Time dependent Risk for 1.5g
Risk(€)
Time(years)
Sol 1
Sol 2

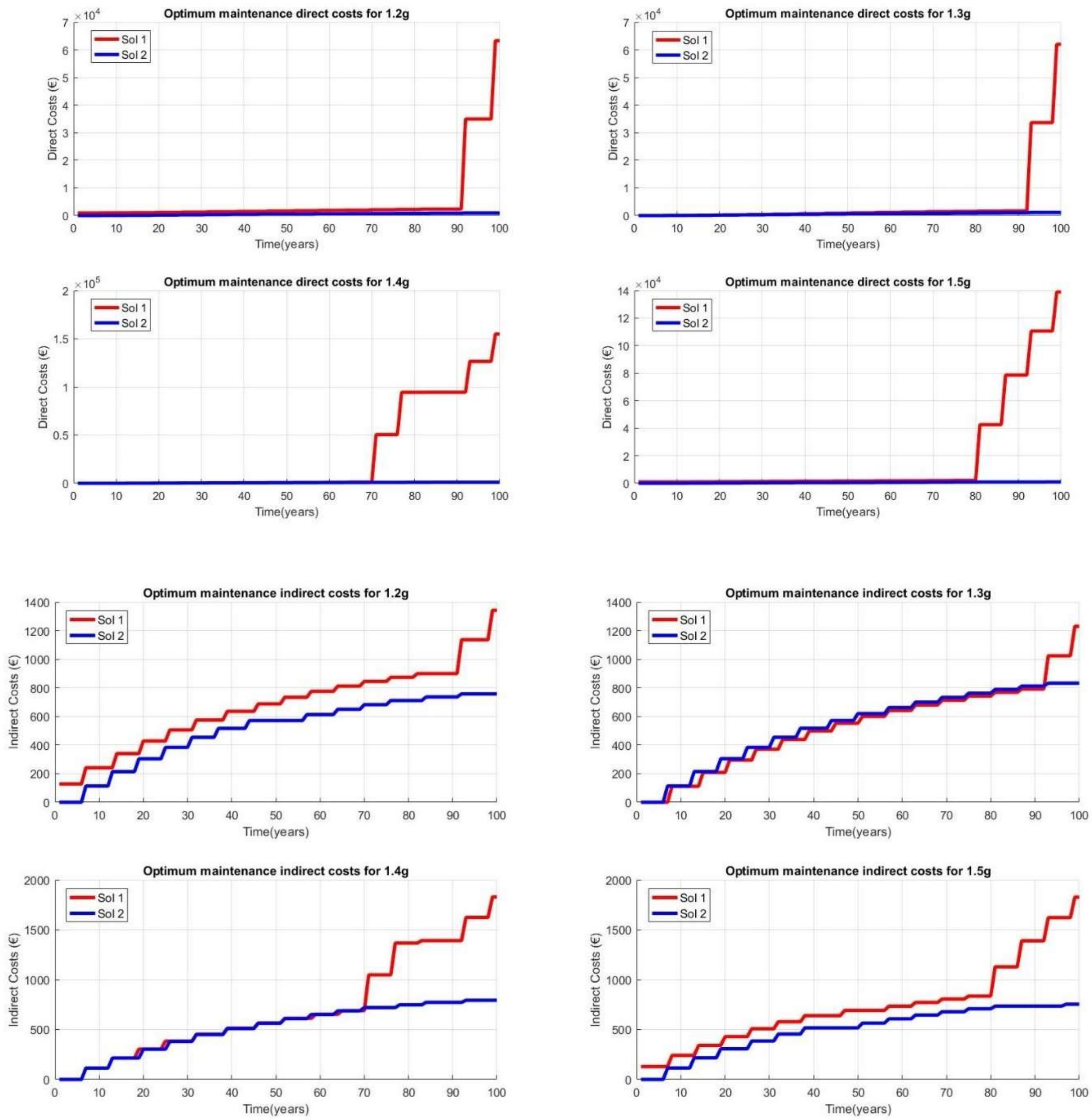


Figure C2.4 - DLS4 for quantile 95 considering retrofit action

Annex C3 – Time-dependent resilience estimation

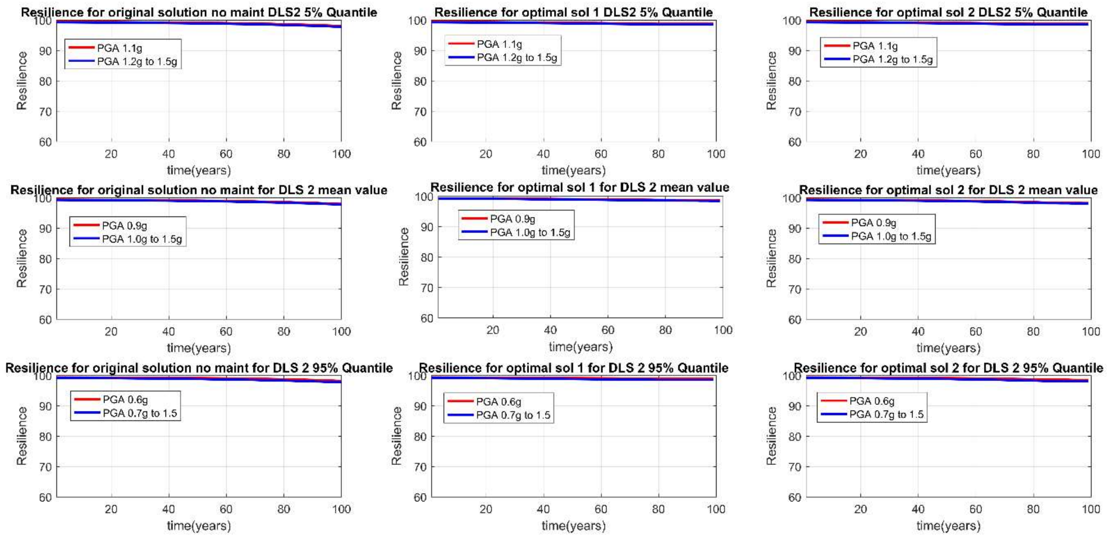


Figure C3.1 - Time dependent resilience for DLS 2 without considering retrofit actions

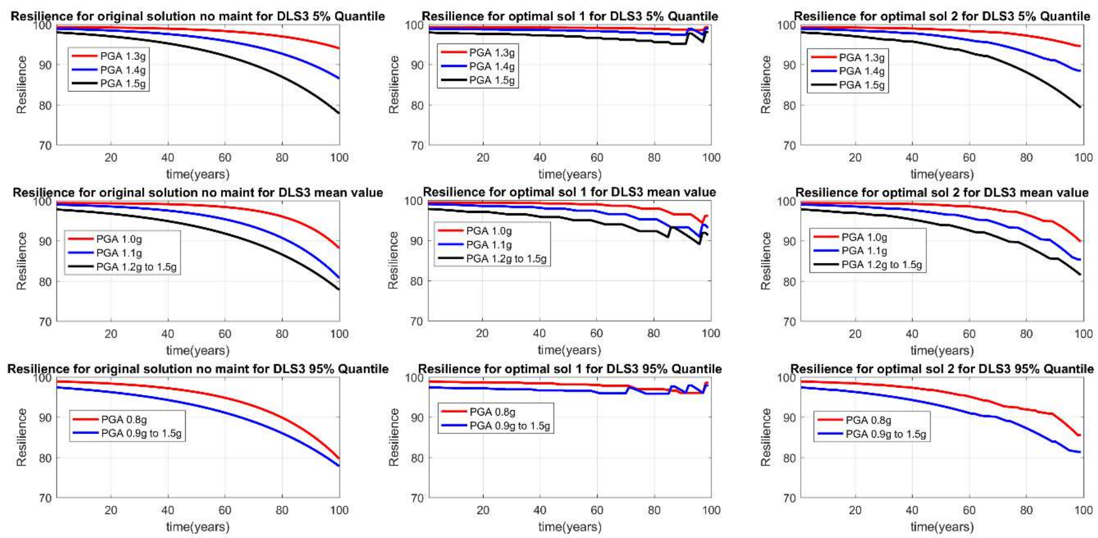


Figure C3.2 - Time dependent resilience for DLS 3 without considering retrofit actions

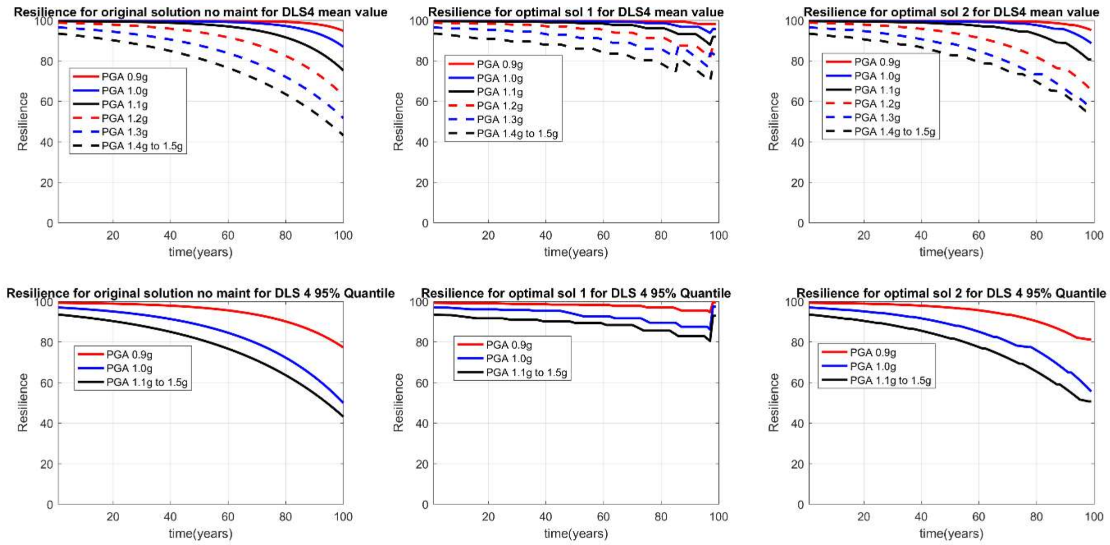


Figure C3.3 - Time dependent resilience for DLS 4 without considering retrofit actions

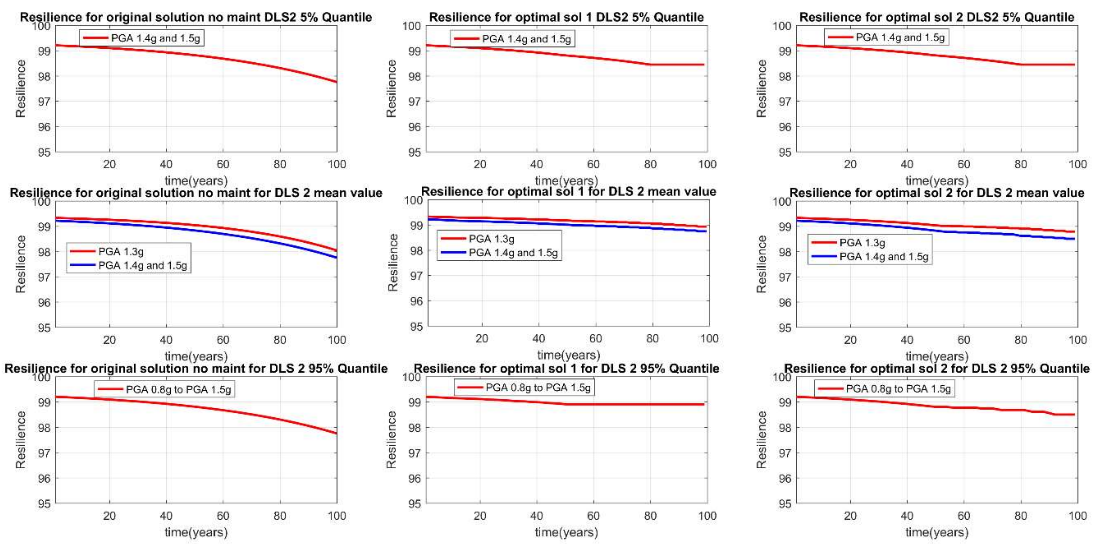


Figure C3.4 - Time dependent resilience for DLS 2 considering retrofit actions

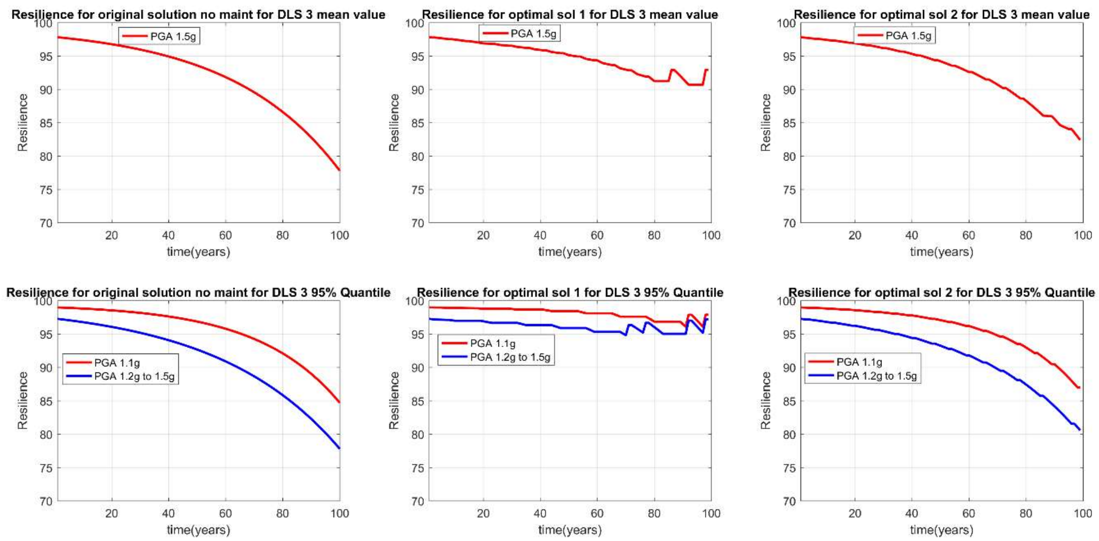


Figure C3.5 - Time dependent resilience for DLS 3 considering retrofit actions

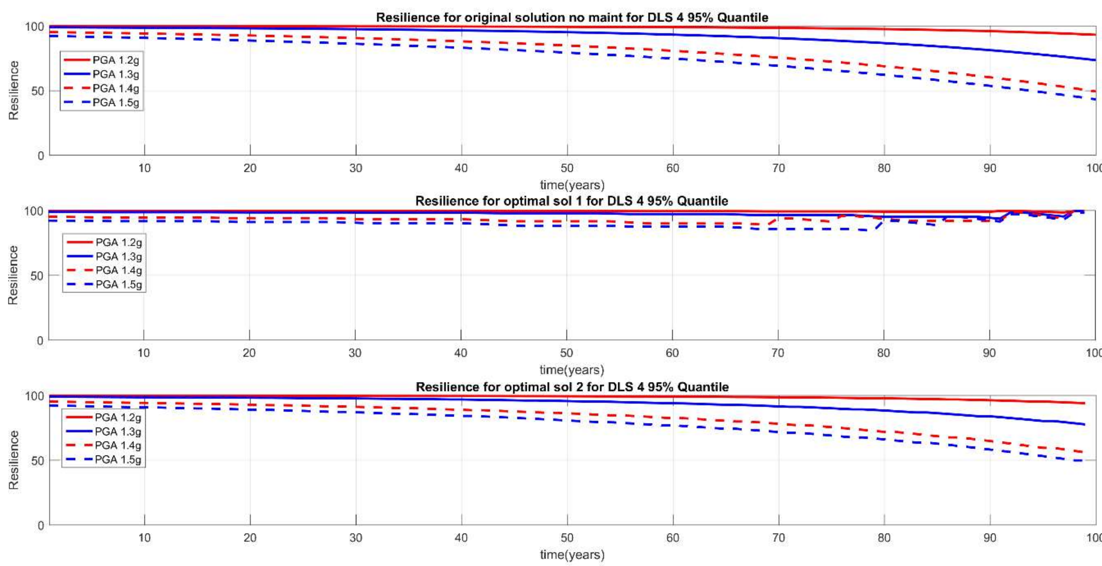


Figure C3.6 - Time dependent resilience for DLS 4 considering retrofit actions