DESIGN OF COMPUTATIONAL EXPERIMENTS FOR FATIGUE ASSESSMENT IN STEEL BRIDGES

Várzeas bridge Case Study

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Abstract

Bridges are among the most important structures in a country's road network. Despite its importance, the financial crisis of 2008 caused a decrease of resources available for bridge management. This reality is transversal to several countries and creates the conditions that may enable the occurrence of catastrophic results.

This Master dissertation proposes a design of computational experiments for fatigue assessment of the longitudinal beams of the Várzeas bridge in order to deal with its response to static loadings from two vehicles of the same type and with similar characteristics, with geometric centers coinciding with the mid-span section, as well as the own weight of the bridge's components. The resulting stress values were obtained using an optimized computational model of the structure with the software Autodesk ROBOT Structural Analysis and compared with the stress value deemed as the limit state in the original design of the bridge.

The probability of failure was obtained using the Monte Carlo Simulation methodology. Using the values obtained by computational simulations, a daily stress spectrum was created, and the respective yearly stress curves were extrapolated considering the traffic of each year and the cumulative traffic effects over the years and compared with the results obtained from a fatigue curve for the material S355 in order to assess the expected time until fatigue failure. Therefore, the proposed design of computational experiments aims at evaluating the structural response of the Várzeas bridge with a high level of precision and can be considered a useful tool for future studies.

Keywords: Design of Experiments, Monte Carlo Simulation, Fatigue assessment, Structural reliability, Bridge condition assessment, Fatigue curve

Resumo

Pontes são das estruturas mais importantes nua rede rodoviária. Apesar da sua importância, a crise financeira de 2008 originou uma redução dos recursos disponíveis para manutenção, realidade transversal a diversos países e que cria condições propícias para a ocorrência de catástrofes, como o colapso da ponte Morandi no verão de 2018.

Esta dissertação de Mestrado propõe um modelo de projeto experimental computacional para avaliação de fadiga aplicado às longarinas longitudinais da ponte das Várzeas, considerando esforços estáticos resultantes de dois veículos do mesmo tipo e com as mesmas características, com os centros geométricos coincidentes com a secção a meio vão da ponte, e do peso dos seus componentes. Os valores de tensão foram obtidos através de um modelo computacional da ponte com recurso ao software Autodesk ROBOT Structural Analysis, e comparados com o estado limite de tensão estabelecido no projeto original da estrutura.

Para a determinar a probabilidade de falha, foi implementada a metodologia de Simulação de Monte Carlo e, posteriormente, elaborado um espectro diário de tensões, extrapolando os valores necessários para desenvolver as respetivas curvas de fadiga anuais, considerando o tráfego de cada ano e os efeitos cumulativos ao longo dos anos. Posteriormente, foram comparados com os valores de uma curva de fadiga para o material S355 obtida através da literatura para avaliar o tempo expectável até ocorrer falha por fadiga. O projeto experimental computacional proposto visa avaliar a resposta das longarinas da ponte das Várzeas com elevada precisão, podendo ser considerado uma ferramenta útil para análises futuras.

Palavras-chave: Projeto de Experiências, Simulação de Monte Carlo, Avaliação de fadiga, Fiabilidade estrutural, Avaliação de pontes, Curva de fadiga

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List of acronyms

BCA	Bridge Condition Assessment
BMS	Bridge Maintenance System
BWIM	Bridge Weigh-in-Motion
DE	Destructive Evaluation
DoCE	Design of Computational Experiments
DoE	Design of Experiments
EU	European Union
FAD	Free-of-Axle-Detector
FEM	Finite Element Modeling
FMECA	Failure Modes, Effect and Criticality Analysis
FORM	First-Order Reliability Method
GWV	Gross Weight Vehicle
JAE	Junta Autónoma de Estradas
LCA	Life-Cycle Assessment
LTR	Load Testing Response
MCS	Monte Carlo Simulation
NDE	Non-Destructive Evaluation
NOR	Nothing-on-Road
RSM	Response Surface Method
SHM	Structural Health Monitoring
SORM	Second-Order Reliability Method
USA	United States of America
VI	Visual Inspection
WIM	Weigh-in-Motion

List of formulas

List of symbols

$\Delta\sigma$	Applied stress range
ADC	Average daily cycles
AW_{ai}	Axle of the i^{th} axle of vehicle type *a*
C	Confidence level to determine the sample size
A	Constant dependent on the detailing category
$F_X(x)$	Cumulative distribution function
x	Design input variable
D	Domain
σ_e	Endurance limit of the material
n_s	Factor of safety
α	Factor of traffic increase
$I(X_i)$	Failure indicator for X_i in Monte Carlo Simulation
a	Generic point
$\sigma_i(\varepsilon)$	i^{th} applied stress component calculated to act at the generic point a
AP_{ai}	i^{th} axle distance of vehicle type *a*
S_{iD}	i^{th} dead load
S_{iL}	i^{th} live load
$R_i(a)$	i^{th} resistance at location a
$S_i(a)$	i^{th} stress resultant at location a (internal action)
S_{iW}	i^{th} wind load
f_{RS}	Joint density function
$f_{\boldsymbol{X}}(\boldsymbol{X})$	Joint probability density function of $\boldsymbol{X}$ in X-space
$G(x)$	Limit state function
S	Loadings
x^U	Lower bound of the domain
$f_{X_i}(x_i)$	Marginal probability density function for the basic variable X_i
σ_{max}	Maximum cyclic stress
$\bar{x}_a$	Mean value of variable *a*
$\bar{x}_b$	Mean value of variable *b*
x_{ai}	Measured *i* value for variable *a*
x_{bi}	Measured *i* value for variable *b*
$y(x)$	Measured response of an experiment
σ_{min}	Minimum cyclic stress
n_i	Number of cycles at $\Delta\sigma$
$N(t)$	Number of cycles on a period *t*
N	Number of cycles to failure
N_i	Number of cycles to failure at $\Delta\sigma$
N_H	Number of successful simulations where failure occur

N_S	Number of total simulations made.
ρ	Pearson's linear correlation coefficient
σ_{pi}	Permissible stress for the *i*^th^ stressed component
$\bar{r}$	Predetermined critical value of a variable x
p_f	Probability of failure
ε	Random error
R	Resistance of a structure
$r(x)$	Response to a stochastic input x
$Y_S^{(K)}$	Response set of size *K*
$X_N^{(K)}$	Sample set of size *K*
m	Slope of the fatigue curve
R_σ	Stress ratio
σ_{sim}	Stress value obtained through computational simulation
y	System response
$y_t(x)$	True response of an experiment
x^L	Upper bound of the domain
$\sigma^2[p_f]$	Variance of the Monte Carlo Simulation problem
$\boldsymbol{X}$	Vector of values for all variables x
$\boldsymbol{X}_a$	Vector of values of variable *a*
$\boldsymbol{X}_b$	Vector of values of variable *b*
t	Year to which will be assessed the number of cycles

1. Introduction

The first chapter presents an outline of the research methodology conducted throughout this Master dissertation. First, it is presented a context of *bridge structural reliability* and its importance to the normal functioning of a structure, as well as the context of bridge failures in Europe regarding the current state of these infrastructures and the measures mentioned by each country's government to address these problems. Finally, it is presented the intended objectives of this Master dissertation, research methodologies and the structure of this document.

1.1. Context

1.1.1. Importance of bridge safety

Bridges are among the most important structures built for public use and are considered an important geopolitical asset. First used to settle unexplored areas, bridges are currently a key component of a country's road network and its continuous use, along with the normal aging of the structure, the increase of traffic flow and the environmental conditions to which a bridge is exposed need to be addressed in order to mitigate the possibility that damage can result in failure or, in critical situations, collapse. In this regard, it is important that the high investments made in a bridge's construction are optimized so the structure can perform in a safe way while being economically viable to the company.

Since the turn of the century, maintenance and structural integrity assessment of public infrastructures has been a main goal of governments throughout the world. The 2008 economic crisis that was originated in the United States of America (USA) and shook the world's financial institutions, as well as the rise of social media, created the perfect storm in which bad spending habits from public institutions and the lack of safety precautions in the continuous use of public infrastructures were on the center of it. From budgetary restrictions to a new social conscience in terms of life-threatening situations when using these structures, the need for new and modern inspection and maintenance procedures have been the subject of an increasing number of studies and articles throughout the last few years.

Nowadays, most bridges worldwide are the target of replacement and strengthening actions due to its lack of reliability and functionality, inevitably creating economic and environmental impacts. In order to mitigate these consequences, it is important to create an optimized maintenance plan, as well as extending the bridge's lifespan in order to avoid its demolition and the construction of a new structure. This is achieved through the bridge's structural assessment, comprised by both simplified structural models (developed through material testing applied to several components, as well as the original documentation) and the structure's intrinsic load carrying capacity (in order to determine a bridge's structural response).

Having in mind that it is very complex to analyze the structural reliability of a bridge, even with the advent of computing evolution, the selection of which structural components are critical is of paramount importance, since the failure of a non-critical component may not necessarily result in the failure of the structure. This factor, along with the incorrect use of standards developed for new bridges and the simplifications assumptions from the design phase, need to be taken into consideration in order to correctly access the state of a bridge in an efficient and cost-effective way.

Most older bridges have been designed based on a criterion based on a limit state of stress, strain and/or other parameters, in which is considered that the structure fails when this limit state is violated, i.e., when the stress value in a certain moment exceeds the value deemed as a limit. This approach is the cornerstone of *bridge structural reliability*, and it is still presently used when accessing the structural integrity of a bridge when subjected to certain load conditions.

Although a widely accepted method, this is considered a conservative approach, resulting in a waste of unnecessary resources (human, material and monetary) on a structure that is not really in danger of failure. To overcome these problems, several methodologies have been proposed in order to achieve results as thorough as possible to the actual structural behavior when subjected to certain loads. With the increase of computational power available, these methodologies are becoming even more based on finite element models, allowing to correctly simulate structural behavior in a correct manner while avoiding the allocation of resources to the location where the structure is. Some of these methods are described further in this Master dissertation.

1.1.2. Public perception of bridge failures

The advent of social media, along with an increasing group conscience from the world as a society, has increased the general oversight of the public in matters related to the safe use of public infrastructures. Situations regarding the lack of maintenance and inspections in these structures are widely criticized, putting the bridge's owners and/or responsible entities on notice.

In Europe, the most prominent aspect is the inexistence of a centralized database for the bridges located in the member states of the European Union (EU). This has been addressed especially since the 14th August 2018 collapse of the Morandi bridge in Genoa, Italy, creating a sense of awareness in the collective mind of the EU member states. An image of this bridge after collapsing is presented in Figure 1.1. Addressing the dangers of uninspected structures, several bridge experts have been on the record saying that the dangers of a lack of inspection and maintenance of older bridges has increased since the economic crisis of 2008. The Economist, quoting a study from 1999, revealed that about 30% of the bridges in Europe have some kind of problem, mostly due to material deterioration [1].

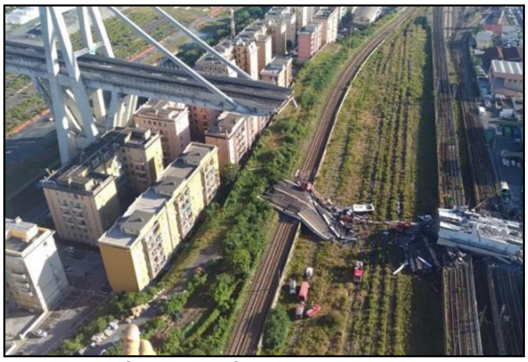

Figure 1.1- Aftermath of the Morandi Bridge collapse

In Portugal, the most notable situation regarding a bridge collapse occurred on 4th March 2001, when the Hintze Ribeiro bridge collapsed, causing the death of 59 people. An image of this bridge after collapsing is presented in Figure 1.2. This failure was the result of several days of intense rain, resulting in an increase of the stream flow of the Douro river which caused the collapse of one of the pillars and the fall of a portion of the bridge's deck. In the aftermath of this collapse, the Social Equipment Minister at the time resigned and a report was made, determining that the collapse was a result of the excess of sand extraction around the fallen pillar, causing its structural weakening, as well as due to the lack of maintenance and inspection of the bridge's pillars. This bridge was constructed in 1884 and was 336 meters long and approximately 6 meters wide, which made the crossing of heavy vehicles subject of traffic restrictions. This structure was replaced less than a year later by a structural reinforced replica of the previous bridge.

Figure 1.2- Aftermath of the Hintze Ribeiro bridge collapse

Several news outlets have tried to understand the current condition of several bridges throughout several countries, coming to alarming conclusions. Quoting a news report from the Portuguese site Observador, the current situation of several countries regarding its bridge's condition and approved measures to address this problem are presented in Appendix A.

Despite several actions that took place since the collapse of the Morandi bridge, the lack of a European centralized entity responsible for data compilation and with the power of creating fiscal packages destined to the maintenance and reparation of the EU member state's bridges is something that needs to be addressed soon.

1.2. Problem statement and objectives

The main purpose of this Master dissertation is to develop a design of computational experiments to assess the fatigue behavior and detect the probability of failure of the Várzeas bridge, a girder (beam) bridge located in the Mealhada municipality, when subjected to certain traffic loads adapted from the literature. For the purpose of this study, the components analyzed were the longitudinal beams, considered as the critical components of the Várzeas bridge, which were subjected to the own weight of the bridge's components and to static loadings, considering two vehicles of the same type at mid span.

The static loads were obtained for six different types of vehicles, whose characteristics regarding the axle loads and distance between axles were obtained through random numbers generated and the different variable's inverse cumulative distribution and the Monte Carlo Simulation methodology was implemented to analyze these sample points. The correspondent stress values were obtained through an optimized computational model of the structure with the software Autodesk ROBOT Structural Analysis. The obtained value was compared to the value deemed as a *limit state* for static loading.

The final step of this Master dissertation was to assess the fatigue behavior of the structure for different years. The daily traffic spectrum was set as a constant and obtained by achieving the requirements of the literature and the yearly traffic loads were then extrapolated, considering a traffic increase of 3 percent per year. These loads were analyzed for both the traffic loadings of the years analyzed and the cumulative traffic loadings up to the year analyzed. These values allowed to develop the fatigue curve for each case, which was then compared with a stress curve of the longitudinal beam's material obtained from the literature.

These analyzes allowed to not only correctly assess the behavior of the longitudinal beams when subjected to static loads, but also to assess the degree of safety from the original design of the Várzeas bridge.

1.3. Research methodology

The subject of structural reliability is a vast area of expertise, making it necessary to establish a research methodology in order to correctly achieve the objectives of this study. The research methodology adopted for this Master dissertation is comprised by:

- Literature review on Bridge Condition Assessment, Design of Experiments, Monte Carlo Simulation and other relevant information.

- Selection of publications with descriptive and innovative information for the purpose of the present study .
- Establishing a connection between this information and the present case-study.
- Selection of a thorough method for the objectives proposed.
- Implementing the adopted methodology.
- Analyze and discuss the obtained results.
- Obtain conclusions from the results obtained, refer limitations of the study and propose future research approaches.

Figure 1.3 shows the schematic methodological framework adopted in terms of information research for the purpose of this Master dissertation.

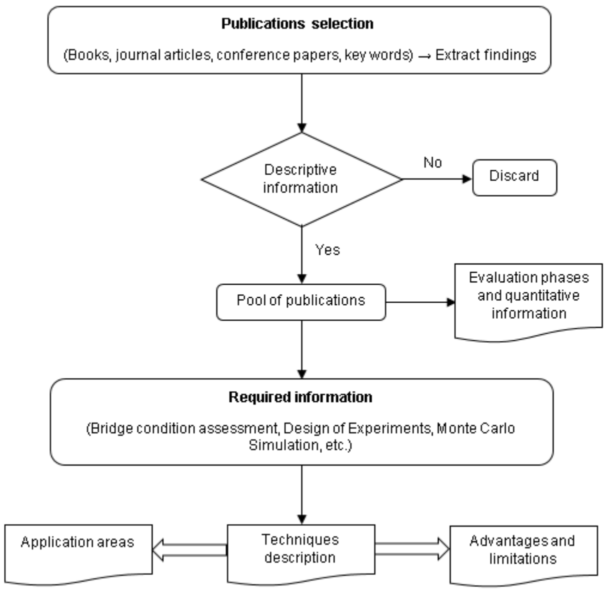

Figure 1.3- Methodological framework adopted for literature review

In this regard, the literature review presented in Figure 1.3 allowed to understand the progresses in the field of structural reliability and bridge condition assessment, as well as to understand several innovative approaches made in a recent past. From the premise that each kind of bridge has a specific type of dominant behavior, it is important to select which information is of value for the case study of this Master dissertation and to determine how to adapt it to this structure.

1.4. Document structure

This Master dissertation is divided as follows:

1. Introduction- This chapter presents some information to contextualize the topic of bridge safety and the way the public views the probability of bridge failures and collapses. Besides these topics, it presents the problem and the objectives of this Master

dissertation, along with the research methodology developed in order to pursue a solution for the problem in hands.

2. Related work- State of the art- The second chapter relates to the literature review made to address the work formerly made by several authors regarding bridge failures and collapses. It includes several approaches to characterize a bridge by type, former statistical studies regarding bridge failures and collapses, the way bridge condition assessment is made, and the role of Structural Health Monitoring and Bridge Weigh-in-Motion have on assessing the structural response of a bridge.
3. Research methods and techniques- The third chapter presents the several approaches adopted for this study. It includes a detailed description of Design of Experiments, Structural Reliability and Monte Carlo Simulation.
4. Case study- The fourth chapter presents a detailed description of the Várzeas bridge and the structural information of its components. Besides these topics, the sensors installed in the Várzeas bridge, and the traffic load and fatigue considerations for this study are presented.
5. Results and discussion- The fifth chapter presents the results obtained by this study and its discussion are presented. The mid span considerations, sample size required for determining the probability of failure with a certain precision and confidence level, the assessment of failure occurrences and the fatigue assessment for the presented traffic conditions are presented. Finally, these results are put into context through its discussion.
6. Conclusions, limitations and future work- The sixth chapter presents the conclusions of this study, its limitations and suggests future approaches to assess the structural reliability of this structure.

1. Related work – State of the art

The second chapter presents the literature review made. It presents the way a bridge can be characterized by type and a review of data compiled by several authors in the past. Besides these topics, the bridge condition assessment methodology is presented, as well as the Structural Health Monitoring approach for bridge automation ant the Bridge Weigh-in-Motion approach to compile a database of vehicle load loadings.

2.1. Bridge types

When dealing with a bridge, it is important to first determine the type of structure to analyze in order to correctly access the kind of behavior it will present. Several categories have been presented in order to characterize a bridge by its type, with the most common ones dealing with its structural configuration, the materials used and its end-use.

2.1.1. Structural configuration

When categorizing a bridge by its structural configuration, the most important factor to analyze is the distribution of tension, compression, torsion, bending and shear forces throughout the structure. Although most bridges are subjected to all these kinds of effects, only a few will be considered predominant regarding the kind of structural configuration. The most common bridge structural types are beam (or girder) bridges, truss bridges, cantilever bridges, arch bridges and tied-arch bridges.

For the present study, it will be analyzed a short-span beam (or girder) bridge, a type characterized by horizontal beams supported in each end, either simply (when the beams only connect across a single span) or continuously (when the beams are connected across at least two spans).

2.1.2. Materials used

When referring to the materials used, the most recent bridges are built in steel, concrete, stainless steel, fiber reinforced polymers (currently under development) or combinations of the materials. These materials, along with the introduction of welding processes in the late 1920's, are among the greater improvements in structural building achieved since the 19th century.

In the early beginnings, the use of wood and stone were predominant when building a bridge. The first major improvement was introduced by the ancient Romans when using cement, which allowed the reduction of the variation of strength typical of natural stone. The use of cement started to decline after the Roman Era, and the next major improvement in materials happened only in the late 1800's, with the use of cast iron as arches when building the Iron Bridge (Shropshire, England) across the Severn river.

In the early 19th century, with the advent of the Industrial Revolution blooming in England, several truss systems were developed for large-span bridges using wrought iron, but it was later observed that it could not support large loads due to its low tensile strength. In order to overcome this obstacle, the use of steel, which has a higher tensile strength than iron made it possible to build large span bridges.

For the present study, the components to be analyzed are made of structural steel.

2.1.3. End-use

When referring to the type of application, a bridge can be categorized as a road, railway and pedestrian bridge, or as a combination of these end-uses. Along with this information, it is possible to determine an adequate mathematical model capable of analyzing the loading scenarios that result from the structure's use.

For the present study, the bridge considered is a road bridge, although several pedestrians use it as well. Since the use of this road by pedestrians is scarce, it will only be considered load cases resulting from vehicles crossing the mid-section of the bridge.

2.2. Data compilation regarding bridge failures

As engineering practices and standards evolve with the knowledge acquired from bridge failures, data compilation from previous occurrences is important in order to create databases that can help to predict patterns and trends, which can help to minimize and/or avoid any kind of failure and collapse. Unfortunately, the creation of these databases come as a result of previous catastrophes, resulting in loss of human lives, material damages and economic, social and environmental impacts. This gives a new meaning to the popular saying that, when we finally have a system that can totally avoid catastrophes of any kind, we will have no more subjects left to observe.

Throughout the years, several attempts to create these types of databases have been achieved, both from a technical and an academic point of view. Although very extensive in the goals its creators aimed to achieve, this preventive approach has only been considered as an important factor in the last few decades. The most interesting attempts at creating these databases, as well as some interesting results will be presented further on.

2.2.1. Types of bridge failures

Throughout the years, engineers have studied a plethora of occurrences regarding the type of damage in bridges in order to acquire knowledge, seeking the reasons for a structure's demise so they could predict the structural behavior of several bridge types and to build more reliable and durable bridges, as well as to avoid costly mistakes in the future. When a bridge experiences damage above a certain limit, these events can be categorized as a failure. When a failure reaches a certain level, this can result in the collapse of the structure. The difference between these occurrences is that, when a bridge experiments a failure situation, there is not a need to restrict traffic flows, since it can be linked to unprecise monitoring techniques. In the case of bridge collapses, there is a need to restrict traffic flows in order to avoid putting in danger the lives of those crossing the bridge and that are in the vicinity of the structure.

2.2.1.1. Bridge collapse

Collapse can be defined as when one or more structural elements fell from a bridge as a result of the failure rendering the structure incapable of remaining in service [2].

A bridge collapse can be classified as a partial or total collapse. A partial collapse can be defined as when "*structures on which all or some of the primary structural members of a span or multiple spans have undergone severe deformation such that the lives of those traveling on or under the structure would be in danger*" [3], whereas total collapse can be defined as when "*structures which all primary members of a span or several spans have undergone severe deformation such that no travel lanes are passable*" [3].

2.2.1.2. Bridge failure

Failure can be defined as a situation that resulted in loss of function (e.g. fatigue cracking that can result in collapse if left unchecked, stress values that violate the value defined as a limit state), that could result in bridge total or partial closures, repairs or strengthening works [2].

When a bridge failure occurs, a principal cause of failure is reported in order to explain this occurrence. A principal cause of failure can be defined as "*errors in design, detailing, or construction; unanticipated effects of stress concentrations; lack of proper maintenance; the use of improper materials or foundation type; or the insufficient consideration of an extreme event*" [4]. A principal cause can often be divided into two distinct causes of failure, namely enabling and triggering causes.

2.2.1.2.1. Enabling causes of failure

An enabling cause of failure can be defined as "*any issue with the bridge that can be identified as an internal weakness or deficiency that leaves the structure vulnerable to failure*" [5].

Enabling causes can be referred to attributes that relate solely to the bridge structure itself (e.g. materials deficiencies, construction failures, design problems, internal defects which can lead to failure, etc.), and the likelihood of its occurrence can be minimized with improvements in the standards used or construction procedures, among other actions. Although they can often be prevented, they are often overlooked and difficult to detect before viewing the problem when building or using the bridge.

Although they can usually be the same as the principal cause, they can create the conditions to make a bridge failure more probable. The most common enabling causes are presented in Table 2.1.

Table 2.1- Most common enabling causes [6]

Enabling cause	Definition
Design errors	Cases where there was evidence of incorrect design assumptions, wrong estimation of loads and oversight of failure modes, among others.

Table 2.1- Most common enabling causes (cont.) [6]

Construction errors	Failures caused by negligence, ignorance, mistakes in calculations, as well as poor workmanship and wrong assembly sequence, among others.
Limited knowledge	Cases where there was insufficient understanding of a failure mode, such as aerodynamic instability or a structural/material problem such as brittle fracture, fatigue or buckling. Usually taking place when using new materials or new forms of design or due to severe extrapolation of what at the time had proved successful.
Lack of maintenance	Common in countries with financial difficulties, it refers to cases where the inspection, repair and overseeing of maintenance actions are neglected by the entities responsible for the maintenance of a bridge.

2.2.1.2.2. Triggering causes of failure

A triggering cause of failure can be defined as "*the causes which are external to the bridge*" [7]. These causes have a larger range than the enabling causes, making them harder to predict and much more likely to result in failure. The added difficulty in their prediction is a result of the fact that they are external to the controlled and engineered aspects of a bridge's performance, but must be accounted for during the design phase as accurately as possible, using factors of safety while avoiding overdesign of the bridge [8].

The most common triggering causes are presented in Table 2.2.

Table 2.2- Most common triggering causes [6]

Triggering cause	**Definition**
Corrosion	When the failure is a result of the deterioration of the bridge's components.
Natural hazards	When failures have taken place due to extreme loading such as flooding, storms or very high winds.
Collisions and overloads	Accidents pertain to vehicle and ship impacts to the structure, fire and explosions (excluding war actions, vandalism and terrorist attacks), creating horizontal loads that the bridge does not undertake when used normally.

Although it is an easy way of evaluating a cause of failure, most failure and collapse situations are the result of a combination of causes, making its evaluation more difficult than at first sight. Therefore, processing and statistically representing the collected data is a complex process since this evaluation is not as linear as first thought.

2.2.2. Impact of bridge failures

The main goal of an engineer when dealing with this kind of projects is to ensure a correct design of the bridge and its structural integrity throughout the entire life span, accounting for the predictable increases in traffic flow along the years, as well as the probability of occurrence of natural phenomena and the selection of materials capable of withstanding the loads which the bridge will be subjected to, among other factors.

Robustness evaluation of bridges within a risk-based framework requires estimation of the probability of occurrence of different hazards followed by an assessment of the vulnerability of the bridge with respect to those hazards, as well as a quantification of the impact of potential failure [2]. The impact of a bridge's failure is often used as an indicator of the importance of that bridge in terms of location, structural configuration and materials used. These impacts can be translated into four main categories: human, economic, environmental and social [9].

Table 2.3 shows several examples of situations that can arise from these impact categories.

Table 2.3- Impact categories of bridge failures [9]

Impact categories	Examples
Human	• Fatalities • Injuries • Psychological damage
Economic	• Replacement/repair costs • Loss of functionality/downtime • Traffic delay/re-routing costs • Traffic management costs • Clean up costs • Rescue costs • Regional economic effects • Loss of production/business • Investigations/compensations • Infrastructure inter-dependency costs
Environmental	• CO2 Emissions • Energy use • Pollutant releases • Environmental clean-up/reversibility
Social	• Loss of reputation • Erosion of public confidence • Undue changes in professional practice

The consequences of a bridge failure can be further divided into direct or indirect. Direct consequences regard the failure of individual components of a bridge, whereas indirect consequences are triggered by direct consequences and are associated with the reduction or loss of a system's functionality.

Once the likelihood of hazard occurrences that may affect a bridge structure is estimated, the next step should be the assessment of the bridge's capacity to withstand these hazards and an appropriate risk assessment [9]. This makes it possible to evaluate qualitatively and quantitatively the consequences of failure, and consequently the evaluation of a bridge's robustness.

2.2.3. Impact quantification of bridge failures

Often expressed in monetary units, quantifying the impact of a bridge failure is, often, a daunting task. Parameters such as the quantification of injuries and loss of lives from people that were on and/or under the structure at the time of failure, as well the financial, economic and environmental impacts are often hard to determine, and several models have been proposed throughout the years to simplify this task.

In addition to fatalities, a bridge failure can result in human injuries. Quantifying the consequences of injuries is an even more challenging task due to the wide range of different injuries that may result from the accident [9]. This has been addressed through the creation of a qualitative injury scale regarding the severity of injuries, after which a severance package is attributed to the person, quantifying the type of injuries sustained. Readers can refer to [9] for further reading on creating injury scales suited to access the human impacts of a bridge failure.

When evaluating the factors affecting these impacts, a modeling framework for bridge failure impacts should account for their type and the relevant time frame, as well as the system boundaries surrounding the structure [9]. The time frame, whether expressed in days, weeks or months, is the metric in which one can evaluate the impacts in the short and long term. The system's boundaries can be defined as structural domains when the system is only comprised by the bridge, and spatial domain when the road network where the bridge is included needs to be accounted for.

Every possible variable must be taken into consideration when quantifying the impacts of a bridge failure, no matter its nature. Readers can refer to [8] and [9] in order to read about a more precise quantitative and qualitative analysis of bridge failures and collapses.

2.2.4. Previous statistical assessment of bridge failures

Engineers have been studying several past bridge failures throughout the years, trying to acquire the best information possible about a bridge's demise with the goal of gaining additional understanding and avoiding mistakes as most as possible in future situations. In order to achieve these goals, several continuous studies consisting on the data compilation of bridge failures and the creation of databases is essential to identify and analyze trends and patterns in past occurrences in order to predict future hazardous situations before they happen.

Along with the studies ordered by national governments and public entities, the data compilation of previous bridge failures is of paramount importance to universities and research centers, whether it is only in an academic perspective or in terms of acquiring scientific knowledge, both in direct and indirect areas of expertise. Direct areas of expertise include subjects such as structural integrity and maintenance management, among others, while indirect areas of expertise include subjects such as materials behavior and bonding processes, among others. Studies focusing on different bridge types in terms of dimensions, materials and form can lead to identifying the predominant causes and/or failure modes for each kind of bridge. Therefore, the statistical analysis of previous data is a strong way to identify the probability of occurrence of the most significant situations that affect the bridge's structural integrity, enabling the mitigation of their consequences.

Although an effective way of reducing and eliminating several design and structural errors, creating a database of this sort is a daunting task due of the lack of publicly available information regarding partial failures and assessments carried throughout the years on these structures. A very reliable source of information was created by Imhof [10] as a part of his PhD thesis, which initially included 347 failure occurrences from 1813 and 2004 in bridges and has been updated since. Although it is a very detailed database, it does not differentiate the several bridges by type, location and materials.

Several approaches to this thematic have been made, with the most accurate one for the study of the present Master dissertation being created by Iman & Chryssanthopoulos [11]. These authors aimed to review the failure statistics of metallic bridges, comprising a database of 164 metallic bridge failures from several sources of literature, Internet and news reports. This led the authors to create a database comprised of bridges from different parts of the world and establishing a relationship between a bridge's failure or collapse to its causes, failure modes and structural configuration, as well as the year in which the bridges were built and when the occurrence took place. Although a good indicator to assess trends and patterns, it has the limitation of not precisely describing which countries the bridges in question are located, as well as it does not differentiate road, railway and pedestrian bridges.

In the USA, continuous studies made by Wardhana & Hadipriono [9], Hadipriono & Diaz [8] and Harik et. al. [4] in the timeframes from 1977 to 1981, 1982 to 1988 and 1989 to 2000, respectively, are considered relevant from an academical point of view to predict trends and patterns in bridge failures. These works proved that vehicle impacts with the structure, along with floods and scour, are the primary causes of bridge failures in the USA. Several similar approaches of worldwide bridges have been made by Smith [5], with this author obtaining the same conclusions.

In order to continue the work of Wardhana & Hadipriono, an attempt was made by Taricska [2] as a part of his Master dissertation, analyzing road bridge failures in the United States in the period from 2000 to 2012. This study was developed with the support of the New York State Department of Transportation and the author compiled the data from several USA state transportation authorities, but not all of them. To fill in the gaps, the author used information from several news outlets and scientific publications, allowing him to look for features and trends that could result in an increase of the failure probability. By identifying the main causes of bridge failures, Taricska analyzed the data compiled and divided it by state, year of failure and the age of the bridge when the failure took place, cause of failure, if the failure resulted in partial or total collapse, materials and structural configuration. This was presented by total failures and percentage of failures through each evaluated parameter, and several case studies were analyzed using Fuzzy logic in order to determine the aforementioned patterns and trends.

When comparing bridges to other types of infrastructures, the work of Eldukair et.al. [6] shows that bridges are among the riskiest structure types in terms of consequences regarding its failure, resulting in high number of deaths and injuries when compared with other types of infrastructures.

Although these studies have shown the need of several analysis of this sort, there is not an official record of bridge failures and a centralized database of that sort for bridges in the EU. To this date each member state of the EU has an individualized database of its bridges structural state, creating several difficulties to analyze this.

2.2.5. Important results obtained from previous statistical studies

Noting that the prediction of patterns and trends in bridge failures relies on learning from past mistakes, reading the results obtained from past studies is an important factor to know what and where to look for. In this topic, several important results obtained from past studies are presented so that the reader can understand the types of tendencies a researcher is looking for and the explanations for them.

The work of Iman & Chryssanthopoulos [11] gives a clear view of the failure modes leading to failures and collapses. When comparing the results for each of these outcomes, several interesting conclusions are obtained regarding the prominent failure modes. Figure 2.1 a) and b) show the distribution of failure modes for collapsed and non-collapsed metallic bridges obtained by the database created by these authors.

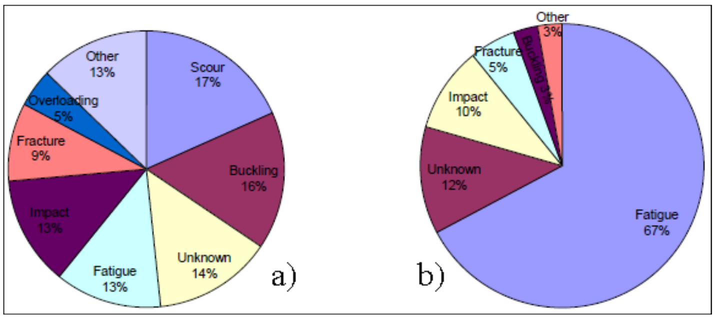

Figure 2.1- Failure modes for metallic bridges. a) collapsed, b) non-collapsed [11]

Figure 2.1 a) shows that there is no single failure mode that can be considered as dominant in metallic bridge collapses, with fatigue and fracture combined being the most accounted for, reaching for a combined 22% of bridge collapses observed, followed by buckling (16%). Meanwhile, Figure 2.1b) shows that fatigue alone is responsible for about two thirds of every non-collapsed bridges observed. This led the authors to conclude that the high fatigue related issues in non-collapsed metallic bridges is a result of the attention paid to this specific failure mode by bridge engineers, resulting in detecting fatigue related problem in early stages through inspections in order to avoid its collapse. Another conclusion achieved by these authors relates to the satisfactory redundancy level regarding fatigue detailed failures.

Also regarding Figure 2.1 b), the authors noted that the high fatigue occurrences in non-collapsed bridges can be partially attributed to the fact that these problems seem to be over-reported, especially for welded bridges constructed in the period 1950-1980 [11]. In this regard, the authors thought that it would be useful to observe in more detail the causes resulting in fatigue cracking. Figure 2.2 shows the distribution of the nature of fatigue cracking in non-collapsed bridges observed in the respective study.

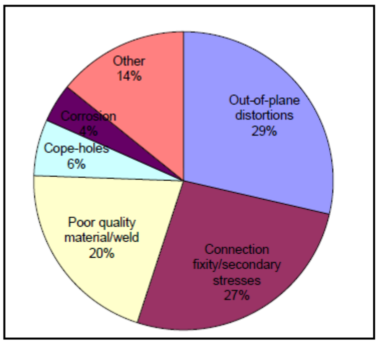

Figure 2.2- Nature in fatigue cracking in non-collapsed metallic bridges [11]

Figure 2.2 shows that the majority of the fatigue cracking observed is attributed to out of-plane distortions in welded structures (29%), unanticipated connection fixity and secondary stresses in riveted structures (27%) and poor quality material/welding (20%) [11], with the former two not being accounted for in the design phase. In this regard, fatigue is considered by the authors as an area that needs to be addressed more thoroughly by researchers in local and global levels in the future.

Regarding overloading issues, these authors observed that this failure mode has a small impact in the occurrences observed by their database, which can be considered as a result of the conservative approach by bridge engineers in the design phase, making these structures durable and safe to the loads that they are subjected to. Another explanation can be the detection of deteriorated components and further substitution before a failure and/or collapse occurs, showing the importance that correct inspection and maintenance actions have in the structural integrity of a bridge.

When looking for tendencies in bridge failures in the USA by location, Tariscka [2] concluded it was important when considering weather patterns in a particular area, number of load cycles experienced by a bridge through the average daily cycles (ADC) experienced on the bridge, as well as the maintenance and inspection costs each state's Department of Transportation has budgeted regarding costs associated with bridge maintenance per year [2]. This author came to the same conclusions regarding the number of bridge partial and total collapses by fatigue. Figure 2.3 shows the total number of occurrences for each failure mode considered.

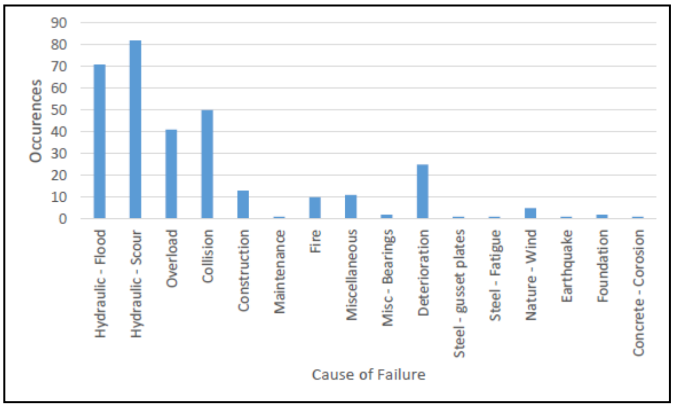

Figure 2.3- Bridge failure occurrences by failure mode in the period 2000-2012 in the USA [2]

From Figure 2.3 it is possible to observe that the number of occurrences caused by fatigue related issues (including steel fatigue, failure in steel-gusset plates, maintenance and several miscellaneous issues) had a weak role to play in the overall database, corroborating the conclusions of Iman & Chryssanthopoulos.

Regarding overloading issues, the results from the database created by Tariscka show that this failure mode has a higher impact when compared to the database of Iman & Chryssanthopoulos. This can be explained by over-reporting from the responsible entities in the USA, as well as the fact that this author considered both partial and total collapses and non-collapse occurrences in the same analysis, as well as the bigger size of the database considered and the fact that all USA states do not have the same approach regarding their bridge's inspection and maintenance.

In this regard, it is possible to observe that fatigue and overloading are two failure modes that are subjected to constant and thorough analysis, especially in the design phase, making it one of the main focuses of a bridge's structural integrity by engineers.

2.3. Bridge condition assessment

2.3.1. Context

The correct assessment of the condition that a bridge is operating in is of paramount importance to mitigate its failure risks and to evade problems in the road network, with all the aforementioned impacts.

In this regard, Bridge Condition Assessment (BCA) has become vital to predict a bridge's future performance and optimizing bridge maintenance, rehabilitation and replacement needs [12]. In order to achieve these goals, several monitoring techniques are used, both individually and simultaneously, in order to create an efficient database of information to predict future failure situations.

2.3.2. Bridge maintenance

Throughout the bridge's lifespan, the structure is maintained by a bridge administrator and its team. During the bridge management phase, there are three types of measurements that can be distinguished [13]:

- Inspection: Activities planned and repeated with predicted intervals. They usually include visual inspection, but they can also include testing and measurements. In some cases, continuous monitoring using built-in or permanently installed sensors is used.
- Assessment: Only made when called for. It can be a structural assessment with respect to the safety or the function of the bridge. It can also be an assessment of the condition of the bridge.
- Maintenance and repair: This can either be periodical maintenance or consist of measures called for by an assessment.

In order to correctly design these structures, a computational structural model is developed to determine a bridge's safety or function, as well as its repair and reconstruction needs. Although this is a powerful tool in bridge assessment and maintenance, these computational models are usually based on several simplification assumptions, which needs to be minimized in order to obtain optimized results. The reasons to perform a structural assessment of a bridge can be subdivided in four main categories [13]:

- Changed requirements: Requirements for increased traffic loads are the dominating reason for structural assessments. Other examples in this category can be changes in codes and regulations or changed requirements due to a change in use.
- Planned reconstruction: Often involves interventions into the load carrying structure, which requires a structural evaluation of the bridge.
- Damage: Can occur due to extreme events like floods, storms and earthquakes. Damage can also occur due to events that the bridge was not designed for, such as overloading, traffic or ship impact, fire and explosions.
- Deterioration: Can be caused by external environmental loading, (e.g. chloride penetration, corrosion, frost, carbonation, fatigue). It can also be caused by reactions inside the material.

These reasons can be applied in order to achieve the goals regarding structural safety, function or condition. The safety of a bridge is evaluated in terms of its possible failure or collapse when subjected to external loading. A bridge's function is evaluated through effects such as a component's deformation. The condition of a bridge is evaluated through, among others, its deterioration state. The measures or activities included in a structural assessment vary from case to case and may consist of one or more of the following parts [13]:

- Structural modeling and analysis: To be able to evaluate safety and function, or to be able to do a closer evaluation of the condition, structural analysis and calculations are needed. A structural assessment of the load carrying capacity includes traditionally this part only.

- Accurate inspections: The regularly inspections made may need to be complemented (e.g. for a more careful survey of the extension and cause for damage or deterioration).
- Testing and measurements: These can include determination of material properties, real geometry, bridge condition, damage extensions, traffic and other loads, among others. Testing and measurements can also be made in order to verify, calibrate and improve the bridge's structural models. This way, also properties that are hard to measure directly can be evaluated (e.g. boundary conditions, and stiffness of internal connections and of damaged structural elements.
- Safety evaluation: With probabilistic methods a more detailed evaluation of the safety can be made. Probabilistic methods are also opened to consider objective specific data (e.g. traffic situation on the bridge).

Depending on the outcome of the structural assessment analysis, the intervention strategy can be defined and implemented depending if the bridge can be used without any kind of restriction (with the actions taking place preferably during periods when traffic flow is lower), if the bridge can be used with reduced traffic flow, or if it needs to be closed for traffic during the intervention (e.g. when strengthening, repair and replacement actions are needed). A bridge's structural assessment is a function of the resistance or capacity of the structure, the actions or loads of the structure and the evaluation and analysis models adopted.

In this regard, the bridge needs to be continuously monitored and its structural evaluation should be considered as an integrated part of the decision-making process, with an adequate level of precision in order to achieve its goals, intention and safety requirements.

2.3.3. Bridge life-cycle assessment

An important part of a bridge life-cycle assessment (LCA) is the implementation of a Bridge Monitoring System (BMS), developed to assist decision makers in maximizing the safety, serviceability (condition where a structure is still considered useful) and functionality levels (levels under which a structure must perform correctly) of bridges within the available budgets [12]. Figure 2.4 shows a representation of a basic framework of a BMS.

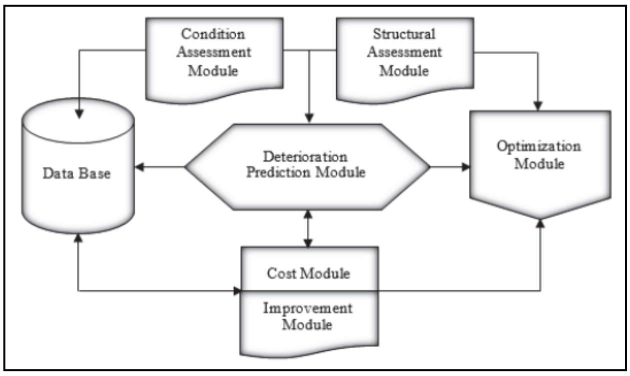

Figure 2.4- Basic components of a BMS [12]

Figure 2.4 shows that the basic framework of a BMS is comprised of several components responsible for the data collection from bridge loading. The first component is a database, responsible for storing inventory and appraisal data obtained from a bridge's monitoring system, as well as feeding this data to the other BMS components. The information obtained is used in several modules, each one with an essential task in this system. Condition and structural assessment modules are used to evaluate a bridge's current health condition, while a deterioration prediction module is used to assess the structure's components future condition, accounting for the present loads and mathematically evaluating these components evolution. From here, a life-cycle improvement module is responsible for the evaluation of agency and user-costs regarding a plethora of maintenance plans. In order to determine which maintenance plan is most cost-effective and simultaneously achieve the desirable maintenance goals, a maintenance optimization module is used. This information is then used continuously throughout the bridge's life-span in order to update not only the structural integrity of the bridge, but also to take into consideration other factors such as the natural evolution of the structure in terms of its aging.

2.3.4. Monitoring approaches

A BCA implementation through bridge monitoring aims to evaluate if the structure will safely operate over a specific period. In this regard, several monitoring approaches are used, preferably simultaneously, following several predetermined guidelines. These guidelines are commonly separated in phases, starting with a preliminary evaluation, followed by a detailed investigation, expert investigation, and finally an advanced assessment, depending on the structural condition of the investigated bridge.

Figure 2.5 illustrates several monitoring approaches in order to achieve the goals of BCA.

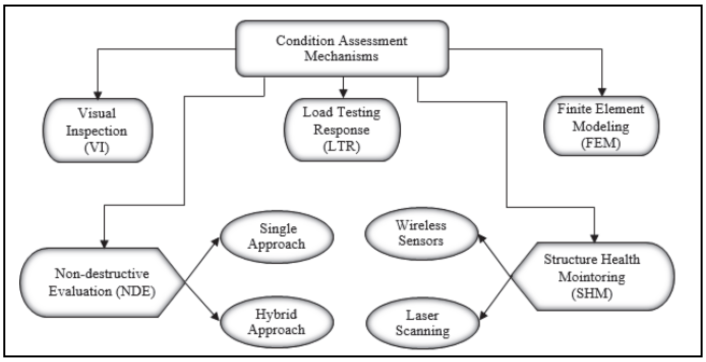

Figure 2.5- Monitoring approaches of BCA [12]

Figure 2.5 shows the five most used monitoring approaches that, when used simultaneously, are able to obtain critical information of a bridge's current state and help develop a strategy in order to mitigate its failure. These structures are Visual Inspection (VI), Load Testing Response (LTR), Finite Element Modeling (FEM), Non-Destructive Evaluation (NDE) (as an alternate approach to Destructive Evaluation (DE)) and Structural Health Monitoring (SHM). Their definitions, as well as the advantages and limitations related to each of these approaches are described in Appendix B.

2.4. Bridge weigh-in-Motion

2.4.1. Context

Many bridges show signs of substantial deterioration due to adverse environmental conditions and increased loading, the last factor being related to the increase of traffic throughout the years. Knowing that the first adversity needs to be dealt with in the design phase and continuously monitored, a bridge's structural capacity when subjected to vehicle loading is assessed through loading monitoring systems, which is a difficult task to implement due to the loading's high randomness between sections, which can lead to an excessive level of safety in some situations, resulting in costly and unnecessary maintenance and repair actions. Besides being the most important factor in fatigue deterioration, traffic also represents the most significant contribution to the total value of the external action to consider for ultimate limit state analysis [14].

In order to correctly and thoroughly monitor and assess a bridge's structural response, the most commonly accepted ways are using Eurocode and Bridge Weigh-in-Motion (BWIM) techniques. Eurocodes are a set of standards created by the EU in order to establish a common standard concept to be used by every EU member-states and adapted to each country's legal specifications, in which calibration trucks are used in order to assess a bridge's structural integrity and fatigue behavior. On the other hand, BWIM is a method by which the axle weights of a vehicle travelling at a certain speed can be determined using a bridge instrumented with sensors [15]. Inserting this information into a mathematical algorithm or a computational model, it is possible to determine the structural behavior of a bridge when subjected to certain loading conditions and enabling the designers to correct the troublesome situations that may occur. BWIM systems consist of axle detectors, devices for measuring strain and data acquisition equipment and are based on the principle that an instrumented bridge is used as a weighing scale [16].

First introduced by Moses, Weigh-in-Motion (WIM) systems were initially composed by sensors placed in the road surface (usually piezoelectric sensors and bending plates embedded into the pavement, normally requiring pneumatic tubes or tape switches) that measured the axle weight and axle distance when in contact with a vehicle passing by, using the bridge as an improved scale to this effect. Nevertheless, these sensors were not durable due to this contact and have been currently replaced by sensors embedded under the structure (e.g. in beams and other critical structural components), allowing the unobtrusive and continuous data recollection without the need of traffic restrictions in order to install and replace these sensors. This variation of BWIM is called Free-of-Axle-Detector (FAD) or Nothing-on-Road (NOR) system).

The performance of each BWIM technology depends on several factors, including its application (e.g. cost-effective design, planning and maintenance, research, prediction of pollution levels, structural enforcement and axle load monitoring and screening), environmental impact, cost and accuracy [17]. Because a BWIM system is used in order to assess certain parameters of a moving vehicle, its sensors measure dynamic loads. The acting loads (the actual loads that need to be measured) are subsequently estimated using the dynamic measured load and appropriate calibration parameters [17].

2.4.2. Theoretical background

In a simple way, the BWIM system is composed by three essential components [16]:

- Axle detectors: used to obtain information about a vehicle's velocity, axle spacing and classification.
- Strain transducers/gauges: attached to the soffit of the bridge, generally at the longitudinal position in order to obtain information about longitudinal strain (maximum strain), which provides information about axle weights, gross weight vehicle (GWV), impact factors and lateral distribution loads.
- Data acquisition equipment: used to store the collected information, making it available for scrutiny.

The most common strategy for BWIM system implementation on a bridge is to locate sensors near the quarter points in a simply supporting span, detecting the axles passing overhead by the data acquisition system, which typically operates at a scan rate in excess of 250 Hz [18]. The measured strain is a linear combination between axle weight, the factored axles and an influence ordinate appropriate to the vehicle location, in which the number of unknowns are the same as the number of axles [16]. The effectiveness of the system will depend on the bridge's geometry and the location of the wheels over the main structural components [18].

BWIM algorithms can be divided into static and dynamic algorithms. Static algorithms include the Moses' algorithm, the influence area method, the reaction force method and the orthotropic BWIM algorithm, all of them aiming the static axle weights of vehicles crossing the instrumented section. Meanwhile, dynamic algorithms, also known as the moving force identification methods, aim at determining the time history of axle forces. Readers can refer to [19] to read more about these algorithms.

2.4.3. Creating a traffic flow database

The needs of correctly modeling traffic effects are directly related to a well-established equilibrium between the accuracy of the implemented algorithms for traffic simulation and the available computational power. Although there have been several studies regarding the development of algorithms as general as possible, most of them only regard the prediction of maximum effects within a predetermined period or the definition of fatigue effects. Even if an algorithm can assess both these topics, the assumptions and simplifications adopted will make it susceptible to errors.

In order to develop a model to simulate the traffic flow over a bridge, the first step is to determine if the traffic flow is free or, in most critical situations, congested. For the purpose of this study, it will be considered that the traffic flow is free, with only one vehicle on each lane at a time, both crossing the critical section (mid-span) at the same time. To establish the period in which the traffic flow will be analyzed, some hypothesis of the stationary character of the studied phenomenon must be drawn to shorten the period of simulation [14]. Therefore, it is possible to determine a certain period (e.g. one day, one week) as the basic period to simulate, due to its stationary character. From here, it is possible to determine parameters such as hourly traffic intensity and daily traffic intensity, among others, since the basic period to simulate is considered constant.

The next step is to statistically analyze the uncertainties regarding the most important variables involved. Among these variables, the most significant are the type of vehicle, intervals between vehicles, GWV of each vehicle and its axle load distribution, spacing between any pair axles and the extreme axles, and the daily intensity of vehicles per lane [14]. This allows to group the vehicles in groups, each one with a statistical distribution depending on the number and characteristics of the sample.

From here, the next step consists in determining the velocity in which a vehicle crosses the critical section, since the static effects over a bridge are obtained through the dynamic effects measured. Knowing the weight per axle of every vehicle crossing the critical section, it is possible to determine the static effect of these loads at each instant.

In terms of analyzing the traffic effects on the bridge, knowing the axle weight of all vehicles crossing the structure and accounting for an impact factor of a vehicle (a deterministic value characterized by each type of vehicle), it is possible to determine the resulting stresses in the section. However, for fatigue analysis, only local extremes (maximum and minimum values) are considered. In this regard, it is important to correctly screen the data available. With these data, it is possible to determine the maximum traffic effects within the established time-span [14].

One result of economic development is the increase of vehicles registered. In Portugal, this situation was predicted by the former *Junta Autónoma de Estradas* (JAE) on its standards for each bridge type, establishing a traffic increase factor for the heavy vehicles that are expected to cross the bridge in one day.

In order to assess traffic increases, it is possible to analyze two different situations: traffic referring to only a specific year; and the cumulative traffic up to a specific year. The number of cycles on a section of a bridge over one year can be determined by:

$$N(t) = ADC * 365 * (1+\alpha)^{t-1} \tag{2.1}$$

Where ADC represents the average daily cycles on the bridge (the number of vehicles crossing a bridge), α is the factor of traffic increase and t the year to analyze.

In order to assess the cumulative number of vehicles that crossed a bridge up to a certain year, this can be done by [20]:

$$N(t) = ADC * 365 * \int_{0}^{t-1} (1+\alpha)^t \, .dt \qquad (2.2)$$

The cumulative effect that traffic loading has on a bridge throughout the years is an important factor to assess the fatigue behavior of the structure, as it will be seen further in this document.

2.5. Structural Health Monitoring

2.5.1. Context

With its usage, civil infrastructures start to deteriorate and the need of parameter measuring, as well as correct maintenance procedures are important to correctly assess their structural integrity in terms of usage and age. These factors take into consideration the structure's level of safety in order to withstand limit loading from, among other factors, overloading situations, impact loads and environmental occurrences. As the state of the art in bridge design is advancing towards a performance-based or a reliability-based design, it becomes increasingly important to monitor and evaluate the long-term structural performance of bridges [21].

In order to implement a correct structural assessment model, it is important to correctly instrument the structure, as well as implementing correct methodologies to interpret the measured data, in order to obtain meaningful about the structure's integrity and performance. In this regard, Structural Health Monitoring (SHM) can be defined as a system able to track the responses of a structure along with inputs, if possible, over a sufficiently long duration to determine anomalies, detect deterioration and identify damage for decision making [22].

SHM aims at creating a diagnosis of every components and materials of a structure, as well as of the structure throughout its lifetime. The structure and its components and materials must remain in a predefined domain and, through usage monitoring, it is possible to obtain a prognosis of key parameters such as residual life and damage evolution, among others. The domain in which the subject must be updated throughout time as a result of the aging resulting from usage, environmental actions and accidental events, among others.

A SHM system is comprised by the structure in question, sensors and data acquisition, management, transfer, interpretation and diagnosis systems. A well selected sensor measures the physical quantity of a damage and sends it to a computer, being sensitive enough to the measured property and insensitive to any other property that can be encountered in its application and that does not influence the measured property [23].

Being an important method to thoroughly assess a structure's health, several studies have been made throughout the last three decades in order to continuously develop SHM systems framework.

From an academic perspective, Catbas [24] wrote a comprehensive article regarding the SHM approach, its components and how to apply a SHM system to new and existing structures, as well as how to efficiently screen the data obtained. Ko & Ni [25] wrote about the technological developments of SHM systems implemented to long-span bridges, including how bridge maintenance and management are linked, as well as how to treat environmental effects in data treatment. Chang et al. [26] wrote about global-based SHM approaches, as well as the way how to implement sensors and future challenges regarding SHM system implementation.

In terms of applying SHM to real structures, Catbas et al. [27] proposed an implementation to the longest bridge in the USA, combining sensor networks and data acquisition systems with FEM along with Monte Carlo simulations, as well as determining the bridge reliability regarding the vehicle loads applied to the structure, and environmental and wind effects. Nie, et al. [28] proposed an in-service condition assessment based on traffic load effects using SHM data. Ye, et al. [29] proposed a fatigue life assessment with SHM data and the way how bridge maintenance and management are related. Regarding a SHM model updating point of view, Schommer et al. [30] proposed an updating model for SHM through static and dynamic inputs.

2.5.2. Complete SHM system development

When implemented on a structure, SHM will allow early interventions in order to solve several kinds of structural problems, which in turn can mitigate economic and social impacts for both the users and the responsible entities. This is achieved by a set of documents that take into consideration the type of structure, location, geographical and atmospheric conditions and the loads that the structure will be subjected to throughout its lifespan. In this regard, SHM should be based on a monitoring plan, an instrumentation plan, an observation plan and a forecast model [31].

A monitoring plan can be defined as a group of specifications that will allow the correct instrumentation of a structure, as well as the regular monitoring of several predetermined parameters, structural behavior prediction and structural management and maintenance actions.

An instrumentation plan comprises a group of specifications that will allow correctly determining which elements should be instrumented, as well as the kind of sensors should be used, and which effects should be monitored.

An observation plan will include the specifications that will determine the correct methods to observe the aforementioned parameters. Finally, a forecast model is comprised of a set of specifications that will determine the correct procedures for data processing and behavior prediction throughout the structure's lifetime [31].

These documents can be implemented to new and existing structures, each one of them needing a different approach in which the desired goals can be obtained.

When trying to develop a complete SHM system, the first step is defining the monitoring goals. The need of correctly defining the terms "health" and "performance" must be discussed since they will have a high impact on the success of the SHM system. Although important, these definitions can be somewhat difficult to obtain due to the difficulty of determining commonly accepted descriptions. In this regard, a SHM system is typically employed to track health and evaluate performance, symptoms of operational incidents and anomalies due to deterioration and damage during regular operation and/or after an extreme event through continuous monitoring of the external loads applied, both ordinary and extraordinary, as well as analyzing the structural response to these events [24].

When discussing the health of a structure, this term can be defined as a deviation from a sound condition as a result of damage and deterioration that would warrant repair, retrofit or strengthening of the structure [24]. This can be continuously monitored through both tests and experimental measurements of parameters such as bending, frequency changes and curvature, among others.

When discussing the performance of a structure, it is important to formulate the analysis objectives, as well as quantitatively defining a limit state (e.g. safety, serviceability, durability). The desired performance of a structure is obtained if it does not exceed this limit state. Along with health monitoring data, several simulations and performance methods are used in order to correctly define a structure's performance.

With these key terms defined, Figure 2.6 shows a schematic representation of the basic needs of a complete SHM system, as well as the inherent challenges facing each need.

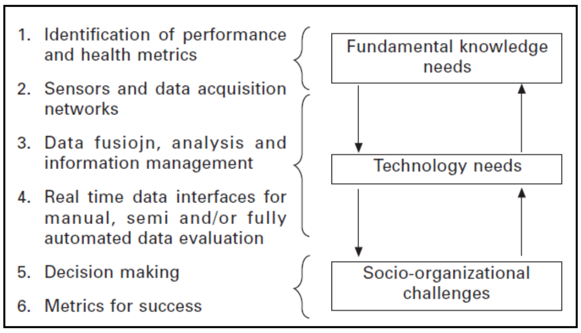

Figure 2.6- Main components of a complete health monitoring design and challenges [24]

From Figure 2.6 it is possible to observe that the first component of this chain consists in the identification of health and performance metric, which will dictate the system's technological needs and requirements.

From here, the technological needs consist in sensors and data acquisition networks able to collect real-time data and correctly select which one is important to the predetermined goals. Future trends will be discussed further down in this document.

From a socio-organizational perspective, the end-users would like to take advantage of SHM for efficient operation, timely maintenance, reduced costs and improved safety [24]. The correct use of an SHM system must find a correct balance with the expectations of the end-users, resulting in situations such as the implementation of maintenance actions occurring during periods where the structure is less solicited (e.g. maintenance actions in bridges during night shifts, when its traffic is lower).

When implementing an SHM system to a new structure, its main goal is to obtain data regarding the significant intrinsic forces and distortions typically related to production in order to manage safety risks during construction, as incomplete structural systems are typically vulnerable and exposed to accidents and hazards [24].

When implementing an SHM system to an existing structure, its main goal is the understanding of the root of several problems that can create problems such as geometry changes, vibrations, displacements and deterioration, among others. With such information obtained, it is possible to correctly determine how to react pertaining to the structure's continuous use, decommission or the level of maintenance and repair actions that need to be taken. To achieve these results, in most cases, monitoring over an extended period may be a necessity for identifying the root causes and mechanisms leading to symptoms of deterioration or damage [24].

2.5.3. Approach to SHM system implementation

An SHM system implementation to a structure can effectively aid data treatment to manage decision making for application of scenarios. The development and implementation of a SHM system to a bridge usually follows trends such as [32]:

- SHM system implementation usually covers about 0.5 to 2% of the bridge's total construction cost.
- Design concept of SHM system usually emphasizes the function to support maintenance and management of the bridge.
- The durability of the SHM system is regarded, resulting in a need of making it possible to replace its elements and not influence the data recording during its replacement.

In order to correctly manage the collected data, the most common question regarding a SHM system management is if the target of the study is the structure or if only a structural component should be analyzed. In this regard, the SHM system can be divided into global-based and local-based approaches [33].

A global-based approach aims at detecting and locating damage by comparing the dynamic properties (e.g. natural frequencies and mode shapes) of a structure obtained at different times with those corresponding to a normal condition. Meanwhile, a local-based approach can be implemented on a component with known deterioration/damage mechanisms occurring at pre-determined critical locations [33].

The most significant difference between these approaches is that, while a global-based approach aims to qualitatively locate damages through the structure's dynamic response, a local-based approach aims at quantitatively determine damage in components that are pre-determined as critical.

From a different point of view regarding the strategy of how a SHM system should be implemented, it is divided in model-based (also known as physical-based) and data-based (also known as model-free or nonphysical-based) approaches.

A model-based approach relies on an initial physical representation/model of a structure (e.g. FEM), the parameters of which can be updated using the data provided by sensors [33]. This approach has the advantage that the damage detected has a direct physical interpretation, although there is an inherent difficulty in the development of an accurate model, as well as it can be intricate to obtain and update the parameters that define the structure [34].

A data-based approach involves the development of a statistical model not directly related to the underlying physical properties of a structure [33]. Therefore, a data-based approach allows circumventing the problem of having to develop a precise structural model, mostly by the use of artificial intelligence (e.g. artificial neural networks), though it is harder to assign a physical meaning to the detected damage [34].

The adopted approach is dependent of the damage precision required. The most general goals are often directly related to structural testing and modeling issues, while more precise goals are characterized by the fields of fracture mechanics, fatigue life analysis or structural design assessment, among others [35].

2.5.4. Advantages of SHM systems

SHM has been the object of continuous developments in the last thirty years, mostly due to the need to address the management of civil infrastructures worldwide. Several studies have been developed in different countries in order to correctly assess their national civil infrastructures with every advantage an SHM system provides. These advantages include identifying global and local structural parameters, obtaining data for structural identification and effective maintenance and operation planning.

Regarding data collection, the most important advantage is the high capability to collect a great amount of data, enabling the improvement of future designs and the diagnosis of the conditions before and after a hazardous event. This allows the maintenance teams to compile a thorough database, as well as sharing these data between several experts and departments inside the management team.

Although it can be considered an enormous advantage, these databases need to be continuously screened and updated so they can effectively consider the data that truly is important to the goals in question.

2.5.5. Challenges regarding SHM systems

Although the SHM system framework has been continuously improved, several challenges still need to be addressed in order to mitigate some aspects that unable SHM to be a completely accurate monitoring method.

The current challenges for SHM implementation, particularly regarding to its implementation on a bridge structure, can be identified as distributed and embedded sensing, data management and storage, data mining and knowledge discovery, elaborating diagnostic methods and presenting useful and reliable information to the owners/managers for decision making on maintenance and management [36].

The most important challenge of a SHM system implementation regards its use towards a structural reliability approach, since these systems are not fully developed regarding correctly assess the structure's maintenance, management and operation actions. This makes SHM more suitable to assess the structural reliability of a structure and its components individually [27].

Considering the uncertainties in data analysis, it is important to consider the inclusion of system reliability analysis and prediction of future performance [27]. With the capability of efficiently collecting an enormous amount of data, there is a growing need to develop systems that can analyze the data and interpret the results efficiently and in a timely manner. This can be achieved through developing data analysis algorithms and methods that can provide the information required in a timely manner, in order to correctly determine the reliability of critical structural components, as well as of the structure as a whole, enabling to determine its probability of failure and expected remaining lifetime [27]. To take full advantage of an SHM system, these algorithms and methods also need to select which data needs to be ignored, in order to correctly offer useful and reliable information, making the SHM system more effectively both in terms of time and costs.

One of the most important challenges regarding an SHM system implementation, irrespective of the approach taken, are the changing environmental (e.g. temperature, humidity, wind) and operational conditions (e.g. vehicle loading) to which the structure is subjected to. These factors will have a significant impact on the measured signals (some authors found discrepancies as high as thirty percent when comparing raw and screened data), making data interpretation a complex task and translating in failures to quantify the input-output relationship between environmental and operational conditions, as well as structural responses (e.g. strains and displacements) or derived quantities (e.g. natural frequencies). These factors make it impossible to distinguish structural changes of interest due to damage/deterioration from changes induced by variability in operational and environmental conditions [37].

2.5.6. Implementing an SHM system to a bridge structure

2.5.6.1. Sample treatment in SHM bridge monitoring

When a global-based approach is adopted, it is difficult to analyze the structure, especially in long-span bridges. In this regard, several local-based analyses need to be addressed, with the entire structure being regarded as a series-parallel system. This approach is commonly used when trying to determine a bridge's reliability and can be very time-effective since the users can take advantage of several factors such as bridge symmetries, helping the maintenance teams to only analyze a portion of the bridge.

Knowing that these analyses need to be time and cost-efficient, each bridge configuration has a specific behavior when subjected to similar loads. Therefore, one can apply statistical sampling along with structural identification and an SHM system, allowing bridges to be grouped into populations whose critical loading and behaviors (e.g. mechanisms that control their serviceability, load capacity and failure modes) may be expressed in terms of only a small number of statistically independent parameters [24].

Knowing which parameters will influence the statistical sampling, the decision process of which measures should be applied can be generalized for every structure within a group, resulting in fewer data needed and simplifying the decision-making process.

2.5.6.2. Variables to be monitored in a bridge structure

Being a key element of a country's road network, the correct implementation of a SHM system in a bridge is an important tool to evaluate its solicitations and current structural health, as well as to predict its evolution through several mathematical models. Bridge SHM systems are generally envisaged to [25]:

- Validate design assumptions and parameters with the potential benefit of improving design specifications and guidelines for future similar structures.
- Detect anomalies in loading and response, and possible damage/deterioration at an early stage to ensure structural and operational safety.
- Provide real-time information for safety assessment immediately after disasters and extreme events.
- Provide evidence and instruction for planning and prioritizing bridge inspection, rehabilitation, maintenance and repair.
- Monitor repairs and reconstruction with the view of evaluating the effectiveness of maintenance, retrofit and repair works.
- Obtain massive amounts of in situ data for leading-edge research in bridge engineering, such as wind-and-earthquake-resistant designs, new structural types and smart material applications.

When implementing a SHM system to a bridge, several variables need to be monitored to assure that the structure in question is within the pre-established domain, ensuring its safe use. Table 2.4 mentions the most important variables to be monitored [38].

Table 2.4-Variables to be monitored when implementing SHM in a bridge [38]

Types	Variables
Load	Earthquake ground motion, wind speed and pressure, vehicles, impact loads, explosive loads and other accidental loads.

Table 2.4-Variables to be monitored when implementing SHM in a bridge (cont.) [38]

Load effects	Acceleration, velocity, static deformation, dynamic displacement, altitude, strain, cracks, tension forces.
Environment	Temperature, humidity, acid, salty solution, alkali, carbon dioxide.
Deterioration	Fatigue damage, corrosion, material ageing, carbonization, freeze-thaw, UV radiation.

When analyzing the variables previously mentioned while observing the individual components of a bridge, several physical phenomena must be studied in order to assess their structural integrity. Some of these phenomena are mentioned in Table 2.5.

Table 2.5- Phenomena to observe in bridges [31]

Structural elements	**Phenomena to observe**
Deck	• Efforts and Deformations • Displacements • Cracking • Reinforcement corrosion • Temperature
Columns	• Efforts and Deformations • Displacements • Cracking • Reinforcement corrosion • Temperature
Abutments	• Efforts and Deformations • Displacements • Cracking • Reinforcement corrosion • Temperature
Foundations	• Settlements • Efforts and Deformations • Cracking • Reinforcement corrosion • Temperature
Accessories (structural bearings, expansion joints, etc.)	• Operation of the various devices

The sensors installed will gather information about the strain suffered by the components and the maintenance team will need to convert this value in order to obtain the stresses to which the components are subjected. This challenge is easily overcome when FEM is used because the stress values are directly obtained from the simulations, as well as it avoids the resource allocation, whether human, material or economic, to the sight where the bridge is located, allowing its analysis in a remote fashion.

2. Research methods and techniques

This chapter presents a detailed review of the methods and techniques implemented in this Master dissertation, namely the Design of Computational Experiments methodology, the Structural Reliability approach to assess the structural response of a structure when subjected to external loading, the Monte Carlo Sampling methodology and a review of stress-life assessment for a bridge subjected to cyclic loadings.

3.1. Design of Computational Experiments

3.1.1. Context

When referring to mathematical optimization, an *experiment* can be defined as a series of tests which the input variables are changed according to a given rule with the goal of identifying the reasons for these changes in the output response. As referred by Montgomery and quoted by Cavazzuti [39]:

> *"Experiments are performed in almost any field of enquiry and are used to study the performance of processes and systems. […] The process is a combination of machines, methods, people and other resources that transforms some input into an output that has one or more observable responses. Some of the process variables are controllable, whereas other variables are uncontrollable, although they may be controllable for the purpose of a test. The objectives of the experiment include: determining which variables are most influential on the response, determining where to set the influential controllable variables so that the response is almost near the desired optimal value, so that the variability in the process is small, so that the effect of uncontrollable variables is minimized."*

Knowing that the data analyzed in an experiment is often corrupted by experimental errors, also referred to as 'noise', and that the output results will be also affected by these errors, it is important to adopt an adequate statistical model to the experiment and to determine the conditions in which an experiment will be performed [40].

This process is referred as 'Design of Experiments' (DoE), which can be defined as a procedure to plan and define the conditions for performing controlled experimental trials. First introduced by Fisher on designing agricultural experiments, this is widely considered as the first major step of this methodology and created the foundations for modern statistical experiments. Although DoE comprises a plethora of statistical analysis models, the second major step is considered to have been made with the work of Box and Wilson, who adapted the principles established by Fisher to industrial experiments and developed the ground basis of the Response Surface Methodology (RSM). Finally, and although considered controversial because of its goals, the work of Taguchi made an indirect significant impact in the increasing the areas in which DoE can be applied, as well as showing its importance in terms of quality improvement.

With the advent of computer technology, a branch of DoE referred to as Design of Computational Experiments (DoCE) was created and has been the focus of increasing publications and developments. This was achieved by developing statistical methodologies that, along with the increase of the computational power available, made it possible to obtain accurate experimental results with a high efficiency, translating in faster computational experiments with precise results. For the purpose of this study, the DoE technique that will be implemented is the Monte Carlo Simulation (MCS) method.

In order to correctly implement the DoE methodology and obtain accurate results, several steps need to be addressed [39]:

- Set the objectives.
- Select the process variables.
- Select an experimental design.
- Execute the design.
- Check that the data obtained are consistent with the experimental assumptions.
- Analyze and interpret the results.

If well addressed, DoE will obtain a precise estimation value with fewer systematic errors.

3.1.2. Development of DoE methodology

The goals of DoE methodologies can be classified as [40]:

- System approximation/prediction.
- System optimization.
- System visualization.
- Numerical integration.

In this regard, the historical developments of DoE can be summed up to improvements in two major fields, namely static and adaptive. Figure 3.1 shows a schematic representation of these fields.

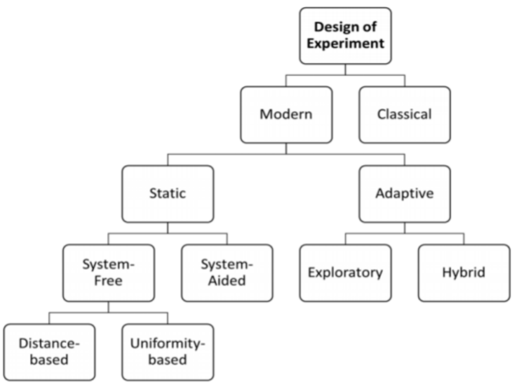

Figure 3.1- Flowchart describing the classification of DoE [40]

The first modern DoE techniques were of static nature, merely aiming to fill the domain as uniformly as possible through randomly placing sample points within the domain, allowing the user to obtain generic and fixed designs which could be repeatedly used for any system once generated [40]. This approach can be referred to as 'system-free' techniques.

Over time, the shortcomings of these techniques had to be addressed, and the 'system-aided' techniques were developed in order to incorporate system knowledge to adapt the design generated to the system's characteristics [40].

Still of static nature, several of the shortcomings of the system-free techniques were still inherent to these new methodologies, and a subject of study in recent years to adopt a more flexible approach in which system knowledge is integrated and used in an incremental or progressive manner during sample generation, in order to obtain efficient results with the smallest sample set as possible [40]. These techniques are defined as 'adaptive' techniques.

For the purpose of this study, it will be only considered static system-free techniques. Readers can refer to [40] for further study.

3.1.3. Experimental techniques

In order to correctly plan and establish the conditions in which an experiment will be implemented, several factors need to be taken into consideration in order to produce correct results. In this regard, the basic principles of statistical methods in DoE are [39]:

- Replication: Repeating the experiment in order to obtain a more precise result and to estimate the experimental error (sample mean value and standard deviation, respectively).
- Randomization: Random order in which the experimental runs will be performed, resulting that each run will not depend on the outcome of the previous runs and will not affect the outcome of subsequent runs.
- Blocking: Isolating a known systematic bias effect and preventing it from obscuring the main effects.

Experiments, whether physical or computational, involve the use of design/input variables, which can be defined as elements, features and attributes that vary to study the response of the system. In this regard, considering that [40]:

$$x = \{x_n | n = 1, 2, \dots, N\} \in \mathbb{R}^N \tag{3.1}$$

Denotes a N-dimensional vector of design variables.

These design variables are comprised in a specified limit defined by the user inside an upper and a lower bound. This space is referred as *domain*, and can be denoted as [40]:

$$D: x^L \leq x \leq x^U \tag{3.2}$$

Where *D* represents the domain, usually scaled as *[0,1]*^{N} to avoid numerical ill-conditioning, and x^L and x^U represent the domain's lower and upper bounds, respectively.

In this regard, a *sample point* can be defined as a specific instance of $x \epsilon D$ and, a set of these points of a size *K* can be defined as a *sample set* [40]:

$$X_N^{(K)} = \{x^{(k)} | k = 1, 2, \ldots, K\} \tag{3.3}$$

Let the *system response* at $x^{(k)}$ be described by *S* output variables[40]:

$$y = \{y_s | s = 1, 2, \ldots, S\} \epsilon \mathbb{R}^S \tag{3.4}$$

The collection of all responses obtained is a system *response set*, defined by[40]:

$$Y_S^{(K)} = \{y^{(k)} | k = 1, 2, \ldots, K\} \tag{3.5}$$

From the sample sets and the respective responses, it is possible to build an approximation for the system response surface, also known as a *surrogate model* or *meta model* [40]:

$$\tilde{f}^{(K)}(x), \tilde{f}^{(K)}: D \rightarrow \mathbb{R}^S \tag{3.6}$$

This approximation model is needed because most variables considered in experiments are of stochastic nature due to having a variety of unknown (hidden) and/or uncontrolled factors resulting in random errors [40]. In this regard, the measured response in an experiment can be modeled as [40]:

$$y(x) = y_t(x) + \varepsilon \tag{3.7}$$

Where $y(x)$ is the measured response, $y_t(x)$ is the true response and ε represents a random error.

In this regard, the primary goal of DoE is to define the best sample points where $y(x)$ is simulated, and to derive the best possible approximation $\hat{y}(x)$ for $y_t(x)$, despite the random error [40]. The way classical and modern DoE address this issue defines its impact in experimental design. In classical DoE, the system response typically assumes a linear/quadratic approximation, where the vertices or points on the boundaries of *D* are considered the best sample points, while in modern DoE this approximation is not considered and the best sample points are the ones distributed within *D* [40].

3.1.4. Generating sample points

Being increasingly used in the last few decades, simulation methods are based on producing a set of random variables (preferably independent), distributed according to a certain distribution *f*, not necessarily known [40]. Although these variables are termed 'random', this is an inaccurate term, such as Von Newmann stated [41]:

> *"Anyone who considers arithmetical methods of reproducing random digits is, of course, in a state of sin. As been pointed out several times, there is no such thing as a random number- there are only methods of producing random numbers, and a strict arithmetic procedure of course is not such a method."*

This is a typical issue regarding physical experiments, where it is possible to generate uniformly distributed random numbers from a largely enough sample size with small intervals between them. These generators tend to be slow and non-reproducible, so that the experiment cannot be checked [39].

In this regard, the most common practical approach regarding simulation methods consists in employing a 'Pseudo' Random Number Generator (PRNG), which a sequence of random numbers that repeats itself after a long cycle interval and based on a mathematical formulation [39]. Although not completely random, these numbers are considered as such for practical purposes. PRNG are among the features presented in several software, such as Microsoft Excel, the software considered in this study for the purpose on random number generation. When using PRNG's in MCS, these methods are dubbed as 'quasi Monte Carlo Sampling' and its use is justified by its uniform distribution property, ensuring a reasonable level of accuracy.

Most PRNG software generate uniform random variables on the interval [0,1], because the uniform distribution $u_{[0,1]}$ provides the basic probabilistic randomness representation, as well as because all other distributions require a sequence of uniform random variables [40]. The most used formulations used in PRNG software are the uniform simulation and inverse transformation, both based on the above domain. Readers can refer to [39], [40] for further reading on these formulations. For the purpose of this study, the PRNG used was a feature of the software Microsoft Excel, based on the Mersenne Twist algorithm.

3.2. Structural reliability

3.2.1. Context

One of the most important steps of a bridge maintenance scheme involves the safety assessment of the structure. A bridge is considered safe when its capacity to resist loadings exceeds these loadings. In other words, a bridge is considered safe when there is a low probability of loading, *S*, to exceed the structure's resistance, *R*, with structural reliability aiming to calculate and predict the possibility of this event happening.

In general, *R* can be described as a function of material properties and element or structure dimensions, while *S* is a function of the applied loads, material densities and, sometimes, element or structure dimensions [42].

In this regard, structural reliability can be described as the calculation of the probability of failure of a random system subjected to random conditions. A structure's response is considered satisfactory regarding if its requirements are satisfied, with each requirement corresponding to a limit state. In other words, structural reliability is concerned with the violation of a structure's performance metrics, in which limit states are included.

The most common limit states are presented in Table 3.1.

Table 3.1- Typical limit states for structures [42]

Limit state	**Description**	**Requirements**
Ultimate (safety)	Collapse of all or part of the structure	Tipping or sliding, rupture, progressive collapse, plastic mechanism, instability, corrosion, fatigue, deterioration, fire.
Damage (often included above)		Excessive or premature cracking, deformation or permanent inelastic deformation.
Serviceability	Disruption of normal use	Excessive deflections, vibrations, local damage.

From Table 3.1 it is possible to say that an undesirable condition results in the violation of the requirements of a certain limit state. Therefore, the study of structural reliability is concerned with the calculation and prediction of the probability of limit state violation for an engineered structural system at any stage during its lifespan [42].

A bridge's loading carrying capacity can be reduced by different forms of deterioration, depending on factors such as the structural material, the quality of workmanship during construction, the age of the structure, the environment and the loading history [43]. This can be carried out by BWIM systems, enabling loading monitoring during an adequate long period of time. In order to assess these negative situations, both NDE and DE can be carried out to get more detailed site-specific information on these deterioration mechanisms to reduce uncertainty and associated level of safety [43].

3.2.2. Structural reliability approaches

It is possible to access a bridge's structural reliability through both a deterministic and a probabilistic point of view. The adopted approach should take into consideration the level of accuracy and level of safety needed, as well as the computational power and knowledge available.

3.2.2.1. Deterministic approaches

Determining a structure's structural reliability through a deterministic point of view is considered a traditional approach and widely used and taught in universities worldwide. This type of approach is considered conservative and considers the factors that define a structure, such as the mechanical properties of the materials and the applied loads, do not have any kind of uncertainty related its values.

The traditional method to define structural safety through a deterministic point of view is through a factor of safety, usually associated with elastic stress analysis and which requires that [42]:

$$\sigma_i(a) \leq \sigma_{pi} \tag{3.8}$$

Where $\sigma_i(\varepsilon)$ is the i^{th} applied stress component calculated to act at the generic point a in the structure, and σ_{pi} is the permissible stress for the i^{th} stressed component, usually defined through structural design codes and derived from material strengths (i.e. ultimate moment, yield point moment).

σ_{pi} are usually expressed in stress terms reduced by a factor of safety, through [42]:

$$\sigma_{pi} = \frac{\sigma_{ui}}{n_s} \tag{3.9}$$

Where n_s is the factor of safety, normally defined from previous knowledge, law requirements translated in national standards, and several approaches such as the Pugsley method [42].

This allows the determination of the limit state violation condition through [42]:

$$\frac{\sigma_{ui}(a)}{n} \leq \sigma_i(a) \tag{3.10}$$

Through appropriate integration, it is possible to rewrite Equation 3.10 in terms of stress resultants [42]:

$$\frac{R_i(a)}{n_s} \leq S_i(a) \tag{3.11}$$

Where $R_i(a)$ is the i^{th} resistance at location a and $S_i(a)$ is the i^{th} stress resultant at location a (internal action). Generally, S_i is comprised by the effects of several applied loads, such as [42]:

$$S_i = S_{iD} + S_{iL} + S_{iW} \tag{3.12}$$

Where S_{iD} is the i^{th} dead load, S_{iL} is the i^{th} live load and S_{iW} is the i^{th} wind load.

Knowing that the failure of a structure occurs when there is a violation of the limit state function, it is possible to determine this event from a deterministic approach through [44]:

$$G(x) = \bar{r} - r(x) = 0 \tag{3.13}$$

Where $G(x)$ is the limit state function, $\bar{r}$ is the predetermined critical value of a variable and $r(x)$ the response to a stochastic input x (basic variable).

Although a simple way of determining structural reliability, this deterministic approach considers that stresses do not represent well the linear elastic structural behavior, not accounting the effects that result from factors such as stress redistribution and stress concentration, among others. This makes this approach highly conservative, which can result in errors in the decision-making process, namely in situations where maintenance, repair and replacement actions are not needed but the calculations show otherwise. These challenges are addressed through probabilistic approaches.

3.2.2.2. Probabilistic approaches

Determining a structure's structural reliability through a probabilistic approach is an important element to optimize resources through the mitigation of the excessive level of safety inherent to deterministic approaches. The inherent probabilistic nature of design parameters, material properties and loading conditions involved in structural analysis is an important factor that influences structural safety, with reliability analysis leading to safety measures that a design engineer must take into account due to the aforementioned uncertainties. In other words, in probabilistic assessments, any uncertainty about a variable (expressed in terms of its probability density function) is considered explicitly [42].

Although an important improvement when compared to deterministic approaches, it does not consider that the actual value of stress is uncertain in a given instant. This is a result of the impossibility to solve a problem considering every factor as a variable.

3.2.2.2.1. Basic structural reliability problem

The simplest structural reliability problem consists on one load effect resisted by one resistance effect from the structure. The inherent probabilistic nature of design parameters, material properties and loading conditions in structural analysis is an important factor that influences structural safety. These effects are obtained through structural analysis procedures from loads and material strength properties (i.e. steel's yield stress), respectively, and translate in a structure's limit state function:

$$G(x) = R(x) - S(x) = 0 \tag{3.14}$$

Where $R(x)$ is the resistance effect, $S(x)$ is the load effect and x represents a stochastic variable of the structural reliability problem. The structure's limit state function will be violated when [42]:

$$G(x) = R(x) - S(x) = 0 \tag{3.15}$$

Similarly, the load and resistance effects can be described in probabilistic terms, translated through probability density functions of both effects. Figure 3.2 shows both functions, as well as the joint density function $f_{RS}(x)$.

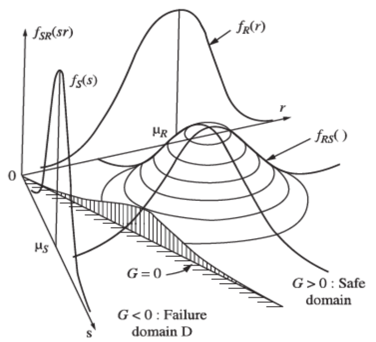

Figure 3.2- R-S problem in the space of two of the random variables *(r,s)*[42]

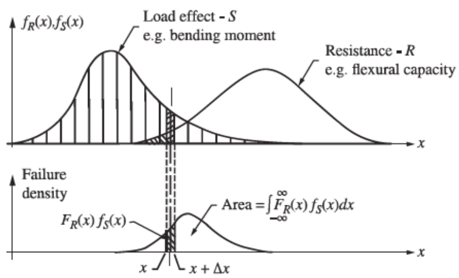

Figure 3.3-Basic R-S problem: $f_{RS}(x)$ representation [42]

In this regard, for any given point, the probability of failure can be formulated as [45]:

$$p_f = P[R(r,s) - S(r,s) \leq 0] = P[G(r,s) \leq 0] \qquad (3.16a)$$

$$p_f = \int_{G(X)\leq 0} \int f_{RS}(r,s)\; dr\; ds \qquad (3.17a)$$

Or:

$$p_f = P[R(x) - S(x) \leq 0] = P[G(x) \leq 0] \qquad (3.16b)$$

$$p_f = \int_{G(X)\leq 0} f_{RS}(x)\; dx \qquad (3.17b)$$

The integrals from Equations 3.17 b) and 3.17 b) represents the volume of the joint probability density function in the failure domain [45]. From Equation 3.19 a), it is possible to observe that the probability of failure is equal to the probability regarding a violation of the limit state function. When $R(x)$ and $S(x)$ are independent, the joint probability function factorizes into the marginal probability density functions. Therefore:

$$f_{RS}(r,s) = f_R(r).\, f_S(s) \qquad (3.18a)$$

Or:

$$f_{RS}(x) = f_R(x).f_S(x) \tag{3.18b}$$

From Equations 3.18a and 3.18b, the probability of failure can be determined through:

$$p_f = \int_{-\infty}^{\infty} \int_{-\infty}^{S \geq r} f_R(r).f_S(s)\ dr\ ds \tag{3.19a}$$

Or:

$$p_f = \int_{G(X) \leq 0} f_R(x).f_S(x)\ dx \tag{3.19b}$$

$R(x)$ and $S(x)$ are usually considered independent but may not be in cases such as when some loads act to oppose failure (e.g. overturning) or when the same dimensions affect both R and S [42].

In terms of the cumulative distribution function, for any random variable x [42]:

$$F_X(x) = P(X \leq x) = \int_{-\infty}^{x} f_X(y)\ dy \tag{3.20}$$

When $x \leq y$, and $R(x)$ and $S(x)$ are independent, Equation 3.19b takes the following form [42]:

$$p_f = \int_{-\infty}^{\infty} F_R(x).f_s(x)\ dx \tag{3.21}$$

Equation 3.25 is also known as the convolution integral, and its behavior is illustrated in Figure 3.4.

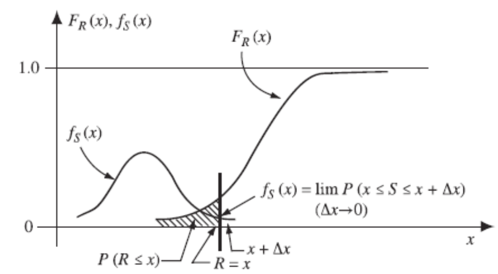

Figure 3.4- Basic R-S problem: $F_R(\,).f_S(\,)$ representation [42]

From Figure 3.4, it is possible to observe that $F_R(x)$ is the probability that $R \leq x$ or the probability that the actual resistance R of a member is less than some value x, resulting in failure (violation of the limit state) [42]. The probability that this is the case is given by $f_S(x)$ that represents the probability that the load effect S acting in the member has a value between x and $x + \Delta x$ in the limit as $\Delta x \rightarrow 0$ [42]. When considering all possible values of x, it is possible to determine the total probability of failure.

Equation 3.21 may also take the following form [42]:

$$p_f = \int_{-\infty}^{\infty} [1 - F_S(x)].f_R(x)\,dx \tag{3.22}$$

Equation 3.26 can be simply interpreted as the sum of all situations where the violation of the limit state occurs.

3.2.2.2.2. General structural reliability problem

Although with a wide range of applications, Equation 3.15 is not entirely adequate to certain situations in which is impossible to reduce the structural reliability problem of a simple formulation for $R - S$, with each being independent random variables [42].

In such kind of problems, the first step is the definition of the variables involved. The most common ones are named basic variables, which are fundamental variables that define and characterize the behavior and safety of a structure, usually employed in conventional structural analysis and design (e.g. dimensions, densities or unit weights, materials, load, material strengths) [42]. These variables are usually independent; however, it is not always the case. When it is not the case, these variables can add levels of complexity to the problem, which makes it necessary to determine the level of dependence between them and the structure to be in some way expressible (normally, through a correlation matrix) [42].

In order to correctly select independent random variables, it is necessary to determine the way the affect one another. This is achieved by determining its correlation, which represents the likelihood that a change in a certain variable will influence the other ones. A well-established method of determining the correlation of variables is the Pearson's R linear correlation method.

$$\rho = \frac{\sum_{i=1}^{n}(x_{ai} - \bar{x}_a).(x_{bi} - \bar{x}_b)}{\sqrt{\sum_{i=1}^{n}(x_{ai} - \bar{x}_a)^2}.\sqrt{\sum_{i=1}^{n}(x_{bi} - \bar{x}_b)^2}} = \frac{cov(\boldsymbol{X}_a, \boldsymbol{X}_b)}{\sqrt{var(\boldsymbol{X}_a).var(\boldsymbol{X}_b)}} \tag{3.23}$$

Where x_{ai} and x_{bi} represent the measured values for each variable, $\bar{x}_a$ and $\bar{x}_b$ represent its mean values and $\boldsymbol{X}_a$ and $\boldsymbol{X}_b$ represent the correspondent vectors for each group of variables.

This method is easily implemented in a software such as Microsoft Excel, and the ways the correlation value can be represented are presented in Table 3.2.

Table 3.2-Pearson's R method values and meanings

Pearson's R method value	Meaning
$\rho \rightarrow 1$	There is a strong positive correlation
$\rho \rightarrow 0$	There is a weak correlation
$\rho \rightarrow -1$	There is a strong negative correlation

From Table 3.2, it is possible to observe that a basic variable will have a correlation value (with other variables) close to zero, meaning that a change in it will not affect the contribution of the other variables.

After selecting these variables, the probability distribution to be assigned depends on the available knowledge, if previous studies and experiences of similar structures are available or, more generally, if the information available is greatly subjective. The probability of distribution may be directly inferred from such data in the first case, whereas in the second case it is necessary to employ subjective information or some combination of techniques. Regarding the parameters of the distribution, they can be estimated through methods such as methods of moments, maximum likelihood or order statistics, which must always be used after data screening, in order to detect trends and outliers, as well as reasons to explain these phenomena [42].

Finally, the model should be compared with the data available whenever possible. This can be done through an appropriate probability paper or, in some cases, through analytical "goodness of fit" tests (e.g. Kolmogorov-Smirnov test) [42].

After establishing all the basic variables and their probability distributions, the first step is to replace the formulation Equation 3.17 for a generalized version, directly expressed in terms of basic variables. Considering a vector $\boldsymbol{X}$ representing all basic variables, the resistance and load can be expressed as, respectively [42]:

$$R = G_R(\boldsymbol{X}) \tag{3.24}$$

$$S = G_S(\boldsymbol{X}) \tag{3.25}$$

When $G_R(\boldsymbol{X})$ and $G_S(\boldsymbol{X})$ are used in $G(R,S)$, the resulting may be expressed as $G(\boldsymbol{X})$, expressing the relationship between the limit state and the basic variables. Since $G_R(\boldsymbol{X})$ and $G_S(\boldsymbol{X})$ may be non-linear, the cumulative distribution functions must be obtained by multiple integration over the relevant basic variables [42]:

$$F_R(r) = \int_r \dots \int f_{\boldsymbol{X}}(x)dx \tag{3.26}$$

$$F_S(r) = \int_s \dots \int f_{\boldsymbol{X}}(x)dx \tag{3.27}$$

Where $f_X(\boldsymbol{X})$ represents the joint probability density function and the integral represents the volume of the joint probability density function in the failure domain.

3.2.2.2.3. Multi-dimensional structural reliability problem

If the problem is of multi-dimensional nature, the probability of failure can be obtained through [42]:

$$p_f = \int \dots \int_{G(\boldsymbol{X})\leq 0} f_{\boldsymbol{X}}(\boldsymbol{x})\, d\boldsymbol{X} \tag{3.28}$$

Where X represents a vector of stochastic variables of the reliability problem, $f_{\boldsymbol{X}}(\boldsymbol{x})$ represents a joint probability density function in X-space and the integral represent the volume of the joint probability density function in the failure domain [45]. R and S are not explicit in Equation 3.28, since they are implicit in $\boldsymbol{X}$.

If all variables are independent [42]:

$$f_{\boldsymbol{X}}(\boldsymbol{x}) = \prod_{i=1}^{n} f_{X_i}(x_i) = f_{X_1}(x_1).f_{X_2}(x_2). \dots .f_{X_n}(x_n) \tag{3.29}$$

Where $f_{X_i}(x_i)$ is the marginal probability density function for the basic variable X_i.

3.2.2.3. Structural reliability assessment methods

It is possible to observe that it is almost impossible to analytically determine $R(x)$ when the subject is a complex or large structure. In order to correctly evaluate the probability of failure with a nice approximation, it is possible to use both simulation and approximation methods.

Many methods have been presented in the last few decades to solve these integral equations. The most popular ones are the First-Order Reliability Method (FORM) and Second-Order Reliability Method (SORM), heuristic methods and simulation methods. The description of these methods is presented in Table 3.3 [45].

Table 3.3- Probabilistic methods to determine probability of failure [45]

Method	Description
FORM/SORM	• Based on first and second-order approximations of the limit state function at the design point, respectively. • Based on different moments of basic random variables such as mean and variance. • Methods that use gradient methods to obtain the shortest difference of the limit state function from the origin of the standard normal coordinate system (reliability index). • Probability of failure determined by $p_f \approx \Phi(-\beta)$, with Φ being the standard normal cumulative function and β the reliability index.
Heuristic	• The shortest distance of the limit state function from the origin of the standard normal coordinate system is considered a fitness function. • Includes methods such as the Genetic Algorithm and Particle Swarm Optimization, among others.
Simulation	• Random samples based on sampling probability density functions are generated from random variables. • Limit state function is calculated for each sampling. • Probability of failure is calculated by dividing the times in which the limit state function is negative to total sampling numbers. • Includes methods such as Monte Carlo Sampling (MCS), Importance Sampling (IS) and Response Surface Method (RSM), among others.

For the purpose of this study, the methods considered will be of simulation nature, namely the MCS methodology.

3.3. Monte Carlo Simulation

3.3.1. Context

As seen in section 3.2, when dealing with time invariant structural reliability problems, several basic variables need to be taken into consideration in order to correctly create a model able to predict the probability of failure. Whereas FORM and SORM approximation methods lead to formulations txxhat require prior knowledge of the means and variances of these variables and the definition of a differentiable failure function, the Monte Carlo Simulation (MCS) techniques only require that the probability density functions of all random variables must be known prior to the reliability analysis [46].

In this regard, the FORM and SORM approximation methods are more suitable for small-scale problems, whereas MCS is more reliable when dealing with high complex problems characterized by a high number of random variables. The available MCS techniques for estimating the probability of failure can be divided either if the application depends or does not depends on the limit state function. For the purpose of this study, it will be considered that the limit state function needs to be taken into consideration by the Monte Carlo Simulation.

MCS is worth exploring when the number of trials or simulations is less than the number of integration points required in numerical integration. This can be achieved for higher dimensions through the replacement of systematic point selection by a 'random' selection, assuming that these points will have an unbiased effect when representing the function to be integrated [42].

3.3.2. Monte Carlo Simulation in bridge structural reliability assessment

When implementing simulation techniques to a structural reliability problem, these methods aim to determine the number of sample points needed to extract the most information possible from these points and to improve the sampling technique in order to increase its accuracy for the same sample size or, if possible, to reduce its size.

In order to apply MCS in a structural reliability problem, the following steps need to be taken into consideration [42]:

- Developing systematic methods for numerical sampling of the variables X_i.
- Selecting an appropriate and reliable simulation technique or sampling strategy.
- Considering the complexity of calculating the limit state function and the number of basic variables on the simulation technique used.
- Selecting an appropriate simulation technique in order to determine the amount of 'sampling' required to obtain a reasonable estimation of the probability of failure.
- Dealing will possible dependence between basic variables.

In order to correctly implementing MCS methodology to a structural reliability problem, it is necessary to estimate as accurately as possible the probable maximum bridge load effects (e.g. bending moments, shears) over a selected period of time [47]. This is made through deriving the statistical distributions of the vehicle weights, inter-vehicle gaps (when there is more than one vehicle over a bridge in the same lane) and other characteristics obtained from previous measurements and used as the basis for traffic simulations for periods up to several years [47]. From here, the maximum load effects over the lifetime of the bridge can be extrapolated from the simulation results [47].

3.3.3. Mathematical formulation

In structural reliability analysis, MCS can be used when an analytical solution cannot be achieved and the failure domain cannot be expressed or approximated by an analytical form (e.g. problems with an inherent level of complexity, where R and S are not independent or with a large number of basic variables where analytical reliability assessment methods are not applicable) [46].

Suppose that the following integral needs to be evaluated:

$$p_f = \int I(\boldsymbol{X}).f(\boldsymbol{X})\,d\boldsymbol{X} \qquad (3.30a)$$

$$p_f = \int_{-\infty}^{\infty} \dots \int_{-\infty}^{\infty} I(X_1, \dots, X_n). f(X_1, \dots, X_n)\, dX_1 \, \dots dX_n \tag{3.30b}$$

From Equation 3.30a, following the law of large numbers, MCS allows to determine the probability of failure through an unbiased estimator [42]:

$$p_f \approx \bar{p}_f = \frac{1}{N}\sum_{i=1}^{N} I(X_i) \tag{3.31}$$

Where X_i is each computational sample with density $f(X)$ and $I(X_i)$ is a failure indicator, defined as:

$$I(X_i) = \begin{cases} 1, & G(X_i) \leq 0 \\ 0, & G(X_i) > 0 \end{cases} \tag{3.32}$$

From Equation 3.37, it can be seen that N independent random samples of a specific probability density function of the vector $\boldsymbol{X}$ are prepared and the failure function is computed for every value of X_i. This makes it possible for MCS to determine the probability of failure in terms of sample mean as [46]:

$$\bar{p}_f \cong \frac{N_H}{N_S} \tag{3.33}$$

Where N_H is the number of successful simulations where failure occur and N_S the number of total simulations made.

3.3.4. Sample size

When implementing simulation techniques to a structural reliability problem, these methods aim to determine the number of sample points needed, to extract the most information possible from these points and to improve the sampling technique in order to increase its accuracy for the same sample size or, if possible, to reduce its size. Figure 3.5 shows a schematic representation of the way a fitted cumulative distribution function to estimate the probability of failure is implemented.

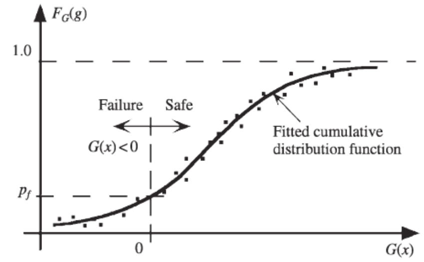

Figure 3.5- Use of fitted cumulative distribution function to estimate p_f [42]

Through Figure 3.5 it is possible to understand that the region of interest is only the one where failure occurs and selecting an appropriate distribution function, since its parameters are extremely sensitive if the sample size is not large enough. In order to overcome this problem, it is possible to fit a sequence of distribution and to optimize the parameters in order to obtain a 'best fit' to a sample problem [42].

In this regard, as shown in Equation 3.33, the probability of failure obtained by MCS approaches the exact value of the actual probability of failure when $N \rightarrow \infty$. Figure 3.6 shows a schematic representation of the convergence of the probability of failure when the sample size increases.

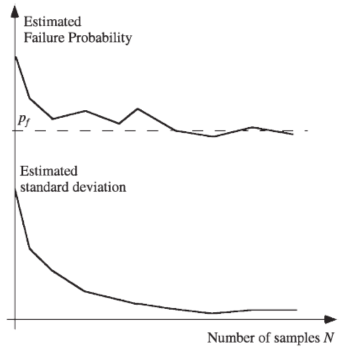

Figure 3.6 -Probability of failure estimation convergence with increasing sample size [42]

Several approaches have been suggested in order to determine the sample size needed to obtain thorough results. A good indicator was presented by [42], which suggests that the number of sample points required for a given confidence level and precision of the probability of failure can be obtained by:

$$N_S > \frac{-\ln(1-C)}{p_f} \tag{3.34}$$

Where C is the desired confidence level.

The variance of the MCS problem is determined through [42]:

$$\sigma^2[p_f] = \frac{1}{{N_S}^2}\sum_{i=1}^{N}\sigma^2[I(G \leq 0)] = \frac{\sigma^2_{I(G\leq 0)}}{N_S} \tag{3.35}$$

From Equation 3.35, it is possible to observe that the standard deviation of the MCS estimation varies directly with the standard deviation of I and inversely with $N^{0,5}$. In other words, MCS estimations decrease proportionally by $N^{-0,5}$. An alternate way of improving the convergence of the probability of failure estimations is through variance reduction.

3.4. Fatigue assessment in steel components

3.4.1. Context

A structure or component may fracture and fail if a certain load is cyclically applied many times. This is defined as a *fatigue failure*, resulting from a progressive propagation of flaws when the material is subjected to static loading [42]. In this regard, fatigue failure can be defined as the number of cycles taken to reach a predefined threshold failure criterion [42].

A fatigue failure occurs after the development of four different stages: crack initiation, crack propagation (when the crack reaches a length in which is able to expand), crack growth (continuation of the crack propagation phase) and final rupture [42]. When dealing with fatigue-life methods, the three major methods are the stress-life method, strain-life method and linear-elastic fracture mechanics method. For the purpose of this study, the fatigue assessment that is made is based on the stress-life method.

When assessing a fatigue failure, three regions to be addressed [48]:

- Plastic region: Region where the material experiences high stress levels with a low number of cycles to failure, resulting in changes in the shape and/or geometry of the component due to repeated stress cycles. This can be defined as the *low-cycle fatigue* (LCF) region of the S-N curve, where the material's plasticity and geometry have a high influence in the number of cycles to failure. This region can be considered when the number of applied cycles is lower than 10^6, and usually it is preferred to implement the strain-life method on this region.
- Elastic region: Region where the relationship between stress and strain is linear. The material returns to its original shape and/or length when the cyclic load is removed. This can be defined as the *high-cycle fatigue* (HCF) region of the S-N curve, where the number of applied cycles is in the interval between 10^4 and 10^6.
- Infinite life: Region where the value of stress is below the value defined as the *endurance limit*, which can be characterized by the possibility of applying an infinite number of cycles without resulting in failure. Widely accepted as being the region there the number of cycles applied it higher than 10^6.

3.4.2. Fatigue curve of a steel

The common form of presenting fatigue data from a stress-life analysis it through a fatigue curve, also known as a *S-N curve*. Originally developed by Wöhler in the 19th century, the fatigue curve can be obtained through experimental testing with test specimens of a certain material in a laboratory and can be defined as a plot of the magnitude of an alternating stress versus the number of cycles to failure of a given material subjected to the loads [48]. Figure 3.7 presents a schematic representation of a fatigue curve.

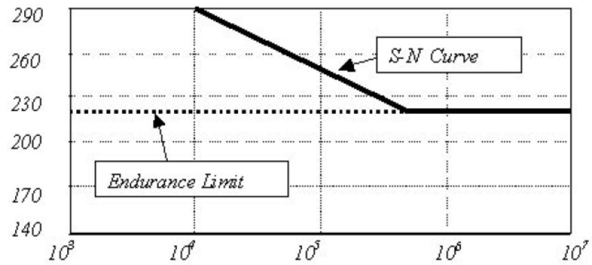

Figure 3.7- Schematic representation of a fatigue curve [48]

From Figure 3.7, it is possible to observe that the fatigue life of a component reduces when the stress range applied increases. When this stress range is lower than a value deemed as *endurance limit* (usually defined at a number of 10^6 applied cycles), the fatigue curve "flattens", and the component is considered safe for infinite life. In this regard:

$$\begin{cases} \Delta\sigma \geq \sigma_e, & finite\ life \\ \Delta\sigma < \sigma_e, & infinite\ life \end{cases} \tag{3.36}$$

Where $\Delta\sigma$ is the applied nominal stress range and σ_e is the endurance limit of the material.

In terms of $\Delta\sigma$, this parameter is determined by:

$$\Delta\sigma = \sigma_{max} - \sigma_{min} \tag{3.37}$$

Where σ_{max} and σ_{min} represent the maximum and minimum stress in a cycle, respectively.

. For the purpose of this study, the cyclic loading is considered as a pulsating cycle, characterized by a minimum stress of zero and a maximum stress referring to the load applied, so that:

$$R_\sigma = \frac{\sigma_{min}}{\sigma_{max}} = 0 \tag{3.38}$$

Where R_σ is the stress ratio, σ_{min} the minimum cyclic stress and σ_{max} the maximum cyclic stress.

To carry out fatigue life predictions in finite life, the relation between the stress range and the number of cycles to failure can be written as:

$$\log(N) = \log(A) - m * \log(\Delta\sigma) \tag{3.39}$$

Where A is a constant dependent on the detailing category, N is the number of cycles to failure and m the slope of the fatigue curve.

It is important to refer that, when a fatigue test is made in a laboratory, the conditions in which the test is made (e.g. environmental conditions) will influence the results obtained. Also, the frequency at which the cycles are applied is not considered to be a factor in the number of cycles to failure [48].

3.4.3. Damage accumulation

When dealing with a structure such as a bridge, it is imperative to determine the cumulative effects of cyclic loading in order to correctly assess its structural condition. A widely accepted approach to assess damage accumulation is the *Miner's rule* [48]:

$$Damage = \sum \frac{n_i}{N_i} \tag{3.40}$$

Where n_i is the number of cycles at $\Delta\sigma$ and N_i the number of cycles to failure at $\Delta\sigma$.

In this regard:

$$\begin{cases} Damage < 1, & safe \\ Damage \geq 1, & failure \end{cases} \tag{3.41}$$

From Equation 3.41, it is possible to understand the importance of the cumulative effects of cyclic loading on the life of a component.

3. Case study

In the present chapter, a detailed description of the case study of this Master dissertation is presented. This description is comprised with information of the configuration and location of the Várzeas bridge, the physical and mechanical information of its components and the information about the sensors installed in the structure. For the purpose of calculations of this Master dissertation, a detailed review of the traffic loading and fatigue considerations for this case study are presented.

4.1. Case study description

The subject of this study will be the Várzeas bridge, located in the Mealhada municipality in the central region of Portugal. This is a short-span girder bridge built in 2009 by VESAM Group, located over an embankment with a small creek flowing by. This structure was originally design in compliance with the Eurocode standards referring to loading and steel behavior to fatigue Figure 4.1 shows an aerial picture of the structure.

Figure 4.1- Aerial picture of the Várzeas Bridge

Through a bug on the Google Maps algorithm, it was possible to see how this portion of the road network was before and after the Várzeas bridge construction, presented in Figure 4.2.

Figure 4.2- Road network on the site of the Várzeas Bridge a) before construction; b) after construction

Figure 4.2 shows that the Várzeas bridge was an improvement on this portion of the road network in several ways. The one that stands out the most is the adding of another lane, existing now one in each direction. This resulted in eliminating traffic congestions that resulted from the existence of a single lane, mitigating the environmental impact caused by this situation. In terms of utility, the bridge was designed to have a pathway in each side, making it possible for pedestrians to cross it safely and without disrupting the traffic. The increase of the bridge's total width was established by the Mealhada municipality in 9 meters, with each lane being 3.5 meters width and each pathway 1-meter width.

In terms of safety, the pathway to be used by pedestrians resulted in a larger distance between the edge of each land and the guarding rails, resulting in a decrease of accidents with vehicles colliding against the guarding rail and, because of this, the bridge's failure probability due to vehicle impact decreased considerably.

In terms of construction issues, the bridge construction involved actions in about 134 meters of the path and VESAM tried to modify the bridge access only 15 to 20 meters on each side, so the landscape would not be significantly altered. Knowing that the soil is of schist nature and has bad geotechnical characteristics, the actions taken were from a land equilibrium point of view, with more excavations than landfills. Finally, the bridge was designed in order to keep the number of expropriations to a bare minimum, trying to use existing tracks when possible.

4.2. Structural information of the Várzeas bridge components

The Várzeas bridge is a short-span, steel beam (girder) bridge composed by four simply supported longitudinal beams, each with a span of 19 meters [49]. In order to increase the bridge's stiffness to torsion when assembling and stabilizing the compressed flanges, a cross girder system was installed through the width of the bridge [49]. To increase the bridge stiffness when subjected to dynamic loads, a bracing system with angles was adopted [49].

This bridge belongs to a secondary path, making it a good test subject because:

- It is located in a remote area, resulting in a small possibility of vandalism.
- The embankment under the bridge means that sensors installation and maintenance and repair actions can be done easily.
- The pathway allows that maintenance, repair and monitoring actions to take place safely and with relative easiness.
- The tight curves that vehicles need to make both entering and exiting the bridge translates in vehicles crossing the bridge slower than usual, which makes the static assessment more thorough.

In terms of constraints, the deck of the Várzeas Bridge is supported by eight supports (four on each side), located on the ends of each longitudinal beam. Figure 4.3 shows a schematic representation of these supports.

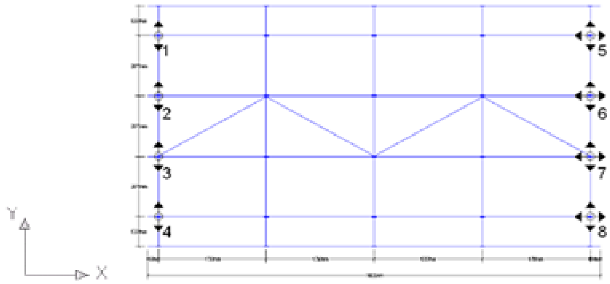

Figure 4.3- Schematic representation of the supports of the Várzeas Bridge [49]

From Figure 4.3, it is possible to observe that the supports 1 to 4 are able to restrict the structure's movement transversally (*y* direction), while the supports 5 to 8 restrict the movement both longitudinally and transversally (*x* and *y* directions, respectively).

Tables 4.1 and 4.2 show the physical and mechanical properties of the bridge's main components, respectively.

Table 4.1- Physical properties of the main components of Várzeas bridge [49]

Component	**Quantity [N]**	**A [m^2]**	**h [mm]**	**L [m]**	**V [m^3]**	**m [kg]**
Slab + pavement	.	.	.	19	42	.
Longitudinal beam	4	0.038	.	19	.	.
Cross beam	20	.	.	2.35	.	134
Profiled sheet	.	213	1.2	.	.	.
Reinforcements	.	.	.	.	.	8800

Table 4.2- Mechanical properties of the main components of Várzeas bridge [49]

Component	**Material**	**Density [kg/m^3]**	**Poisson ratio**
Slab + pavement	C30/37	2400	0.15
Longitudinal beam	S355	7850	0.3
Cross beam	IPE360	7850	0.3
Profiled sheet	S320GD	7850	0.3
Reinforcements	S500	7850	0.3

For the purpose of this study, as the longitudinal strain will occur in the longitudinal direction, it will be considered that the critical components of this bridge will be the longitudinal beams, namely the center longitudinal beams. These components are made of S355 steel, a widely used structural steel characterized by a yield stress of 280 MPa for these range of dimensions.

Through past experiments, it was possible to observe that the longitudinal beams near the bridge's pathway will offer a greater resistance to loading. This can be explained by the effects between the safety rails and the concrete, which can result in an analysis that is more conservative in the longitudinal beams near the edge and less conservative in the longitudinal beams near the center of the bridge, when analyzing the structure.

4.3. Várzeas bridge sensors

In order to continuously monitor the structure, several sensors are installed in the Várzeas bridge to correctly assess the loadings from both vehicles crossing by and environmental conditions. These sensors are a part of the MIRA system, developed by VESAM and tested in several bridges.

In terms of the sensors used to measure the bridge effects when subjected to loadings, Figure 4.4 shows the schematic representation of the monitoring system used in the Várzeas Bridge.

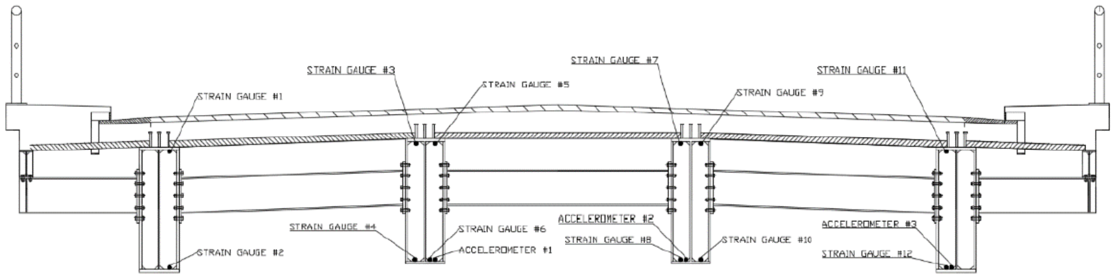

Figure 4.4- Schematic representation of the sensors installed in the Várzeas Bridge [49]

From Figure 4.4, it is possible to observe that several strain gauges are installed under the structure, in order to measure the longitudinal loads in several sections. These sensors are installed both in the top and bottom of the cross beams, with the top sensors measuring these effects close to the bridge deck and the bottom sensors serving as a validation component. It is also possible to observe that several accelerometers are installed in the bottom of the cross beams in order to measure the dynamic effects from vehicles in motion.

These sensors are activated through a trigger installed in the pavement. Figure 4.5 shows the trigger of the sensors installed on the Várzeas bridge.

Figure 4.5- Sensors' trigger on the deck of the Várzeas bridge

The trigger in Figure 4.5 activates the strain gauges and accelerometers when a vehicle crosses the bridge, with these sensors recording the vehicle loadings during a 5 seconds period and working with a 50 Hz frequency.

In order to measure the weather effects to which the bridge is subjected to, a weather station is installed in the bridge in order to measure temperature, wind, precipitation and relative humidity at each moment. Unlike both strain gauges and accelerometers, these sensors are activated periodically in order to measure the environmental effects in a certain time. For the purpose of this study, the environmental effects will not be considered, since only the static effects of vehicle loading, and the bridge's dead weight will be considered.

4.4. Traffic load considerations

Although well instrumented, the MIRA system has the disadvantage of not recording the traffic loadings and rating them by vehicle types. This is a result of the poor accuracy of the sensors used, which are able to correctly assess the effects of vehicle loading but not to correctly assess the weight per axle and distance between axles. To overcome this constraint, it will be considered vehicle loadings made in the study conducted by Guo et al. [20].

These authors considered that the vehicles analyzed can be divided into six configurations based on its load per axle and distance between axles, and both mean value and standard deviation for each variable. These values were obtained by the authors through a BWIM system integrated in the Throgs Neck Bridge in the USA, a long-span girder bridge with six lanes (three lanes for each direction), with all the measurements referring to September 27th, 2005. To ensure that the measurements were accurate, all vehicles that registered a front axle weight higher than 89 kN were removed from the sample and any anomaly registered by the BWIM system was compared to video footage of the day in question. The dimensions for axle weights and axle spacing are in kN and meter, respectively.

To analyze the loads to which the bridge is subjected to, sets of 3000 random numbers were generated for each variable distribution in Microsoft Excel, which were then inserted to the associated cumulative distributions to generate a single random vehicle. This procedure will ensure that all variables for each type of vehicle will be independent from each other.

From simulation purposes, two vehicles will be considered at mid-span and, through Autodesk ROBOT Structural Analysis, the loads to which the longitudinal beams will be subjected to will be simulated and the resulting value of stress will be compared to the limit state value. A failure occurrence will be defined by a stress value obtained from a simulation higher than the limit state stress value.

4.4.1. Vehicle variables to be considered

In order to implement a DoE methodology, the first step is the definition of which variables will be of interest to the study. For the present study, six types of vehicles will be considered, with each type being characterized by the weights per axle and distance between axles. These variables can be considered as basic/independent variables, since the authors of [20] collected the results for which its distributions were determined individually. The model used to implement the vehicle loads has been previously developed and optimized by VESAM, and it was possible to use with the software Autodesk ROBOT. This model is comprised by isoparametric finite elements and is widely used by VESAM in its structural analysis of the Várzeas bridge.

4.4.1.1. Vehicle 1 characteristics

The vehicle type 1 considered is a basic automobile, comprised of two axles. Figure 4.6 presents a schematic representation of the vehicle type and Table 4.3 includes the characteristics of these vehicles.

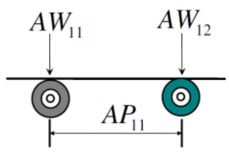

Figure 4.6- Schematic representation of vehicle type 1 [20]

Table 4.3- Random variables for axle weights and axle spacing regarding vehicle type 1 [20]

Variable designation	**Distribution type**	**Mean value**	**Standard deviation**
$AW_{11}[kN]$	Normal	38.22	9.74
$AW_{12}[kN]$	Normal	62.08	20.92
$AP_{11}[m]$	Lognormal	6.036	1.044

In terms of the distance between wheels, it will be considered that it has a constant value of 1.50 meters for all axles.

4.4.1.2. Vehicle 2 characteristics

The vehicle type 2 considered is a basic automobile, comprised of three axles, with the middle axle closer from the rear axle. Figure 4.7 presents a schematic representation of the vehicle type and Table 4.4 includes the characteristics of these vehicles.

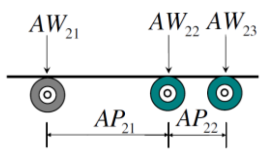

Figure 4.7- Schematic representation of vehicle type 2 [20]

Table 4.4- Random variables for axle weights and axle spacing regarding vehicle type 2 [20]

Variable designation	**Distribution type**	**Mean value**	**Standard deviation**
$AW_{21}[kN]$	Lognormal	48.06	8.51
□□$_{22}[kN]$	Lognormal	62.02	24.43
□□$_{23}[kN]$	Lognormal	56.92	21.99
□□$_{21}[m]$	Lognormal	4.864	1.388
□□$_{22}[m]$	Lognormal	1.310	0.052

In terms of the distance between wheels, it will be considered that it has a constant value of 1.60 meters for all axles.

4.4.1.3. Vehicle 3 characteristics

The vehicle type 3 considered is a basic automobile, comprised by four axles, with the second axle closer to the front one and the third axle closer to the rear one. Figure 4.8 presents a schematic representation of the vehicle type and Table 4.5 includes the characteristics of these vehicles.

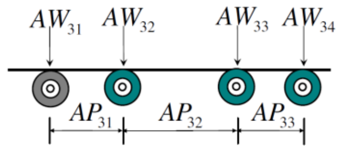

Figure 4.8- Schematic representation of vehicle type 3 [20]

Table 4.5- Random variables for axle weights and axle spacing regarding vehicle type 3 [20]

Variable designation	Distribution type	Mean value	Standard deviation
$AW_{31}[kN]$	Normal	44.46	8.73
$AW_{32}[kN]$	Normal	68.06	23.56
$AW_{33}[kN]$	Lognormal	63.96	25.04
$AW_{34}[kN]$	Lognormal	61.44	28.22
$AP_{31}[m]$	Lognormal	4.497	1.044
$AP_{32}[m]$	Normal	1.325	0.058
$AP_{33}[m]$	Normal	7.724	2.200

In terms of the distance between wheels, it will be considered that it has a constant value of 1.70 meters for all axles.

4.4.1.4. Vehicle 4 characteristics

The vehicle type 4 considered is a basic automobile, comprised by four axles, with axle weights' distributions similar to the ones presented for type-3 vehicles, with the difference being in the distances between axles. Figure 4.9 presents a schematic representation of the vehicle type and Table 4.6 includes the characteristics of these vehicles.

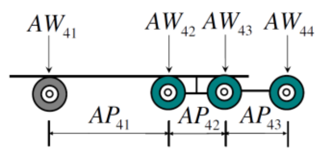

Figure 4.9- Schematic representation of vehicle type 4 [20]

Table 4.6- Random variables for axle weights and axle spacing regarding vehicle type 4 [20]

Variable designation	Distribution type	Mean value	Standard deviation
$AW_{41}[kN]$	Normal	44.46	8.73
$AW_{42}[kN]$	Normal	68.06	23.56
$AW_{43}[kN]$	Lognormal	63.96	25.04
$AW_{44}[kN]$	Lognormal	61.44	28.22
$AP_{41}[m]$	Lognormal	3.929	0.298
$AP_{42}[m]$	Normal	10.114	1.317
$AP_{43}[m]$	Normal	1.230	0.034

In terms of the distance between wheels, it will be considered that it has a constant value of 1.70 meters for all axles.

4.4.1.5. Vehicle 5 characteristics

The vehicle type 5 considered is a basic automobile, comprised by five axles. Figure 4.10 presents a schematic representation of the vehicle type and Table 4.7 includes the characteristics of these vehicles.

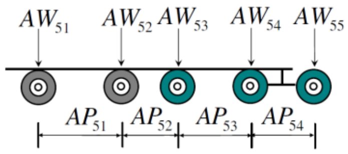

Figure 4.10- Schematic representation of vehicle type 5 [20]

Table 4.7- Random variables for axle weights and axle spacing regarding vehicle type 5 [20]

Variable designation	Distribution type	Mean value	Standard deviation
$AW_{51}[kN]$	Normal	47.31	6.68
$AW_{52}[kN]$	Lognormal	63.19	21.05
$AW_{53}[kN]$	Lognormal	59.70	20.44
$AW_{54}[kN]$	Lognormal	61.04	23.13
$AW_{55}[kN]$	Lognormal	59.37	27.24
$AP_{51}[m]$	Normal	4.609	1.073
$AP_{52}[m]$	Normal	1.323	0.033
$AP_{53}[m]$	Normal	9.839	1.136
$AP_{54}[m]$	Normal	1.258	0.049

In terms of the distance between wheels, it will be considered that it has a constant value of 1.90 meters for all axles.

4.4.1.6. Vehicle 6 characteristics

The vehicle type 6 considered is a basic automobile, comprised by six axles. Figure 4.11 presents a schematic representation of the vehicle type and Table 4.8 includes the characteristics of these vehicles.

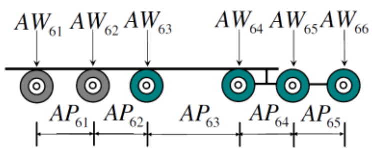

Figure 4.11- Schematic representation of vehicle type 6 [20]

Table 4.8- Random variables for axle weights and axle spacing regarding vehicle type 6 [20]

Variable designation	Distribution type	Mean value	Standard deviation
$AW_{61}[kN]$	Normal	48.48	8.06
$AW_{62}[kN]$	Normal	88.94	25.72
$AW_{63}[kN]$	Normal	82.61	25.49
$AW_{64}[kN]$	Normal	80.89	32.91

Table 4.8- Random variables for axle weights and axle spacing regarding vehicle 6 [20]

$AW_{65}[kN]$	Normal	84.88	30.65
$AW_{66}[kN]$	Normal	80.65	30.84
$AP_{61}[m]$	Normal	4.996	0.421
$AP_{62}[m]$	Normal	1.342	0.051
$AP_{63}[m]$	Lognormal	4.487	1.418
$AP_{64}[m]$	Lognormal	1.288	0.105
$AP_{65}[m]$	Lognormal	1.283	0.088

In terms of the distance between wheels, it will be considered that it has a constant value of 2.00 meters for all axles.

4.5. Fatigue considerations

In order to consider the traffic effects on fatigue, the number of vehicles considered will be based on the bridge's specifications. According to [20], the Várzeas Bridge is a type T6 bridge according to the ex-JAE specifications [50]. This means that the daily average yearly heavy vehicles traffic (vehicles characterized by a GWV higher that 3500 kg) in each lane of the bridge will be a maximum of 150 vehicles. Considering the daily vehicle's load spectrum as a constant, it is possible to determine the Várzeas Bridge fatigue spectrum for the first year by multiplying the results obtained from the daily fatigue spectrum by 365 days.

In order to assess the structure's response to loadings in further years, it is necessary to take into consideration the yearly traffic increases. For type T6 bridges, according to the former JAE specifications [50], the mean yearly traffic increase rate considered was 3%. This value is established taken into consideration several factors, such as previsions for the variation of the price of oil.

4. Results and discussion

This chapter presents the approach taken in order to assess the objectives of the study. It is first presented the mid span considerations, the number of sample points and the results obtained from the computational analysis. Further, it is presented the fatigue assessment of the longitudinal beams of the Várzeas bridge for these conditions. Finally, the obtained results are discussed.

5.1. Mid span section considerations

For various loadings over a simply supported bridge, it is common to consider that the mid span section is critical. This can be assumed since it is the section where the maximum (longitudinal) strain occurs, as well as it is the section where it is commonly assumed that the maximum bending moment occurs. Although it is a widely accepted assumption, it is not true for the maximum bending moment, which only occurs when the centerline of the span is located midway between the load's gravity center and the nearest concentrated load. This effect greatly influences the results in dynamic studies, but in static studies can be assumed without negatively influencing the obtained results in a considerable way.

For simulation purposes, it is considered that the geometric center of the vehicle obtained by dividing in half the distance between extreme axles is coincident with the midspan section. Furthermore, the vehicles will be considered moving forward in each direction according to the Portuguese traffic rules, which establishes that the vehicles need to be driven on the right side. Finally, it is considered that the vehicles at the midspan section are centered in terms of the lane occupied (transversally centered).

These assumptions, along with the geometrical characteristics of the bridge, can be a useful tool in order to simplify future calculations, since it can be considered that the effects that happen in one lane are symmetric to the effects on the other lane.

5.2. Sample size

In order to determine the number of samples required to achieve a certain precision for the value of the probability of failure and for a given confidence level, Equation 3.32 provides a good estimation of this value.

For a confidence level of 95%, in order to obtain a precision level of 10^{-3}, the number of samples required are:

$$N > \frac{-\ln(1-0{,}95)}{10^{-3}} \approx 2995{,}73$$

For simplification purposes, each situation will be analyzed for 3000 random sample points.

In terms of data treatment, it is expected that several sample points por each vehicle type needs to be eliminated as a result of the random numbers generated being too high or low, resulting in a value obtained from the inverse cumulative function of said variable that will negatively influence the output results.

The correct way to treat the data obtained would be by defining a range of values obtained from the inverse cumulative function of each variable of each vehicle type, eliminating every sample point that has at least one of these values off the predetermined ranges. Although, for the purpose of this study, there is not any criteria applied in the literature that can deem a certain range as adequate. This makes the efforts to determine these ranges susceptible to the assumptions made by the author and leading to the possibility of eliminating adequate data and/or not eliminating inadequate data, along with the possibility of reducing the number of sample points to a size that will not allow to obtain the precision required.

Taken into consideration the above considerations, for the purpose of this study, it was considered that the criteria for data treatment are:

- The values obtained by the inverse cumulative function of each variable must be positive;
- The resulting TLV must not be higher than the total length of the Várzeas bridge.

In order to assess the correlation between variables, Equation 3.32 was implemented. The correlation matrixes for each type of vehicle are presented in Appendix C.

5.3. Failure occurrences from overloading

In order to analyze the structural integrity of the computational model for the Várzeas bridge, the first step is to create the sample points from which the vehicle loading will be generated. This was achieved by generating 3000 random numbers between 0 and 1 (using Microsoft Excel random number generator) and applying the inverse cumulative distribution for each variable of a given vehicle type. In other words, each vehicle type associated random variables (AW_{ai} and AP_{ai}, representing the value of the i^{th} axle weight and distance between axles for vehicle type a, respectively) will represent a single vehicle described by its axle weights and distances between axles.

In terms of analyzing the vehicle loadings on the computational model of the Várzeas bridge, each vehicle generated randomly were inserted into Autodesk ROBOT Structural Analysis and placed on the model, specifically where the geometric center of the vehicles is coincident with the midspan section, and centered in the lane they were in. Each simulation occurrence is comprised of two vehicles (one in each lane) of the same type and generated by the same random number, with each situation being simulated 3000 times for each type of vehicle.

5.3.1. Limit state consideration

When determining if a certain situation will result in failure of the critical components (longitudinal beams), with the data from [49], it will be considered that the limit state is defined by the limit state stress, σ_{LS}, and that the Várzeas bridge will not fail when:

$$\sigma_{sim} \leq \sigma_{LS} \tag{5.1}$$

$$\sigma_{sim} \leq 280\ MPa$$

Where σ_{sim} is the stress obtained from each simulation. In this regard, a failure occurrence will result from a value of σ_{sim} higher than 280 MPa.

5.3.2. Simulation results from loading of vehicles type 1

The first situation analyzed is two vehicles of type 1, one on each lane and located at midspan, simulated for 3000 sample points. For 3000 sample points generated, 1 sample point was eliminated due to a low random number generated, resulting in a negative value obtained from the correspondent inverse cumulative distribution. For the remaining 2999 sample points generated, it is possible to observe from Table C1 that the variables that will be considered in the simulation process for vehicle type 1 are independent from one another, since the correlation values obtained by Equation 3.23 tend to zero. Therefore, the variables that characterize vehicle type 1 can be considered basic variables.

After obtaining the σ_{sim} values for each sample point, it was possible to observe that there were no failure occurrences.

$$\left(\sum_{i=1}^{3000} I(\boldsymbol{X}_i)\right)_{1-1} = 0$$

$$\left(p_f\right)_{1-1} = 0$$

Analyzing the values of σ_{sim} obtained from each sample point, it is possible to observe that the maximum value obtained is about 56.52 times lower than the value of σ_{LS} for the longitudinal beams. From these values, it is possible to conclude that the Várzeas bridge is safe when subjected to static loads from vehicles of type 1 and to the loads that result from the structure's components' weights.

Figure 5.1 shows the histogram for the σ_{sim} obtained from all the simulations made for this situation, as well as the number of events for each range.

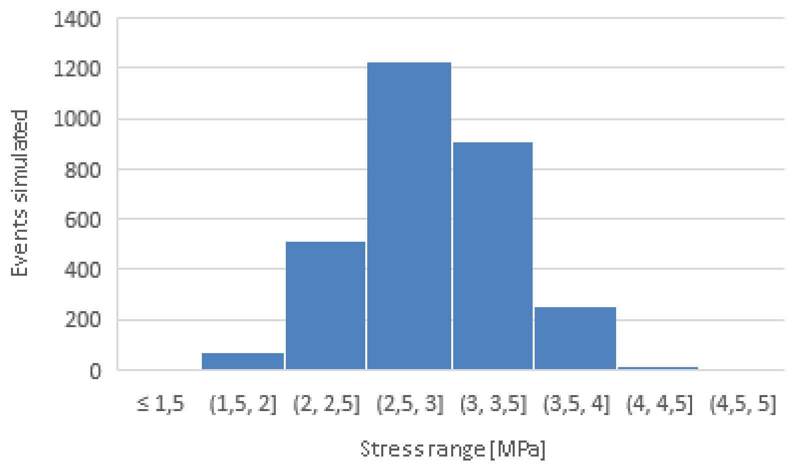

Figure 5.1- Histogram of the stresses obtained from the simulations made for vehicle type 1

From the simulations made, the minimum value obtained was 1.325 MPa and the maximum stress value obtained was 4.954 MPa. From Figure 5.1, it is possible to observe that the values of σ_{sim} obtained are defined by a normal distribution with a mean value of 2.885 MPa and a standard deviation of 0.460 MPa, as well as that the stress range with a higher number of events simulated is]2.5;3.0] MPa, with 1223 occurrences.

5.3.3. Simulation results from loading of vehicles type 2

The second situation analyzed is two vehicles of type 2, one on each lane and located at midspan, simulated for 3000 sample points. For 3000 sample points generated, 37 sample points were eliminated due to a low random number generated, resulting in a negative value obtained from the correspondent inverse cumulative distribution. For the remaining 2963 sample points generated, it is possible to observe from Table C2 that the variables that will be considered in the simulation process for vehicle type 2 are independent from one another, since the correlation values obtained by Equation 3.23 tend to zero. Therefore, the variables that characterize vehicle type 2 can be considered basic variables.

After obtaining the σ_{sim} values for each sample point, it was possible to observe that there were no failure occurrences.

$$\left(\sum_{i=1}^{3000} I(\boldsymbol{X}_i)\right)_{2-2} = 0$$

$$\left(p_f\right)_{2-2} = 0$$

Analyzing the values of σ_{sim} obtained from each sample point, it is possible to observe that the maximum value obtained is about 13.86 times lower than the value of σ_{LS} for the longitudinal beams. From these values, it is possible to conclude that the Várzeas bridge is safe when subjected to static loads from vehicles of type 2 and to the load that result from the structure's components' weights.

Figure 5.2 shows the histogram for the σ_{sim} obtained from all the simulations made for this situation, as well as the number of events for each range.

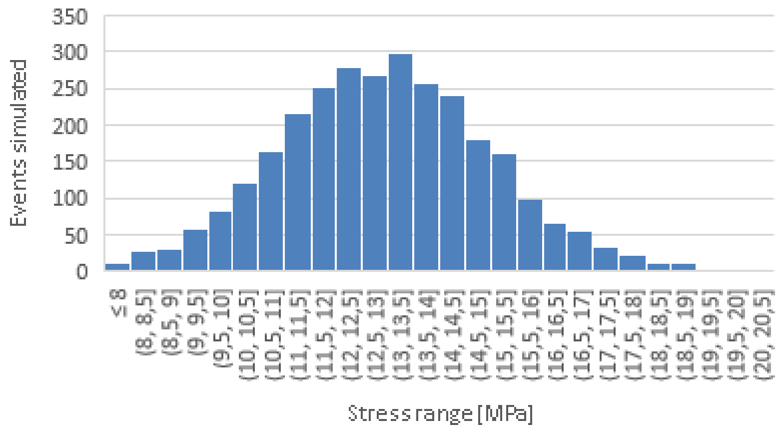

Figure 5.2- Histogram of the stresses obtained from the simulations made for vehicle type 2

From the simulations made, the minimum value obtained was 5.597 MPa and the maximum stress value obtained was 20.208 MPa. From Figure 5.2, it is possible to observe that the values of σ_{sim} obtained are defined by a normal distribution with a mean value of 12.956 MPa and a standard deviation of 2.058 MPa, as well as that the stress range with a higher number of events simulated is]13.0;13.5] MPa, with 299 occurrences.

5.3.4. Simulation results from loading of vehicles type 3

The third situation analyzed is two type-3 vehicles, one on each lane and located at midspan, simulated for 3000 sample points. For 3000 sample points generated, 61 sample points were eliminated due to a low random number generated, resulting in a negative value obtained from the correspondent inverse cumulative distribution. For the remaining 2939 sample points generated, it is possible to observe from Table C3 that the variables that will be considered in the simulation process for vehicle type 3 are independent from one another, since the correlation values obtained by Equation 3.23 tend to zero. Therefore, the variables that characterize vehicle type 3 can be considered basic variables.

After obtaining the σ_{sim} values for each sample point, it was possible to observe that there were no failure occurrences.

$$\left(\sum_{i=1}^{3000} I(\boldsymbol{X}_i)\right)_{3-3} = 0$$

$$\left(p_f\right)_{3-3} = 0$$

Analyzing the values of σ_{sim} obtained from each sample point, it is possible to observe that the maximum value obtained is about 12.90 times lower than the value of σ_{LS} for the longitudinal beams. From these values, it is possible to conclude that the Várzeas bridge is safe when subjected to static loading from vehicles of type 3 and to the loads that result from the structure's components' weights.

Figure 5.3 shows the histogram for the σ_{sim} obtained from all the simulations made for this situation, as well as the number of events for each range.

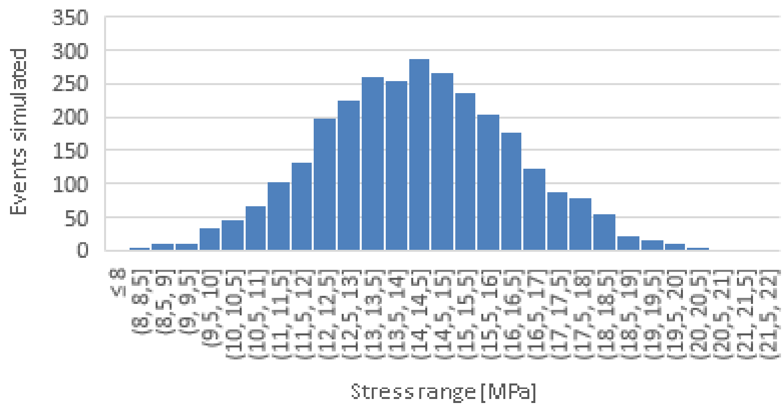

Figure 5.3- Histogram of the stresses obtained from the simulations made for vehicle type 3

From the simulations made, the minimum value obtained was 7.903 MPa and the maximum stress value obtained was 21.710 MPa. From Figure 5.3, it is possible to observe that the values of σ_{sim} obtained are defined by a normal distribution with a mean value of 14.211 MPa and a standard deviation of 2.111 MPa, as well as that the stress range with a higher number of events simulated is]14.0;14.5] MPa, with 289 occurrences.

5.3.5. Simulation results from loadings of vehicles type 4

The fourth situation analyzed is two type-4 vehicles, one on each lane and located at midspan, simulated for 3000 sample points. For 3000 sample points generated, 68 sample points were eliminated due to a low random number generated, resulting in a negative value obtained from the correspondent inverse cumulative distribution. For the remaining 2932 sample points generated, it is possible to observe from Table C4 that the variables that will be considered in the simulation process for vehicle type 4 are independent from one another, since the correlation values obtained by Equation 3.23 tend to zero. Therefore, the variables that characterize vehicle type 4 can be considered basic variables.

After obtaining the σ_{sim} values for each sample point, it was possible to observe that there were no failure occurrences.

$$\left(\sum_{i=1}^{3000} I(\boldsymbol{X}_i)\right)_{4-4} = 0$$

$$\left(p_f\right)_{4-4} = 0$$

Analyzing the values of σ_{sim} obtained from each sample point, it is possible to observe that the maximum value obtained is 11.48 times lower than the value of σ_{LS} for the longitudinal beams.

From these values, it is possible to conclude that the Várzeas bridge is safe when subjected to static load loadings from vehicles of type 4 and to the loads that result from the structure's components' weights.

Figure 5.4 shows the histogram for the σ_{sim} obtained from all the simulations made for this situation, as well as the number of events for each range.

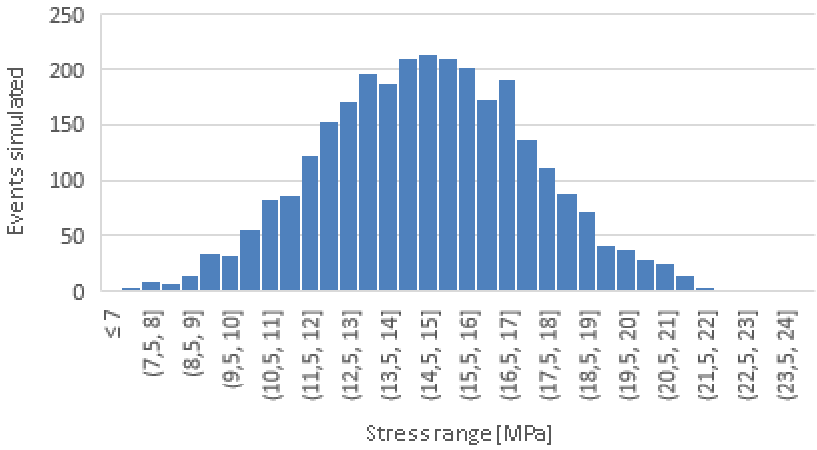

Figure 5.4- Histogram of the stresses obtained from the simulations made for vehicle type 4

From the simulations made, the minimum value obtained was 6.571 MPa and the maximum stress value obtained was 24.383 MPa. From Figure 5.4, it is possible to observe that the values of σ_{sim} obtained are defined by a normal distribution with a mean value of 14.721 MPa and a standard deviation of 2.655 MPa, as well as that the stress range with a higher number of events simulated is]14.5;15.0] MPa, with 214 occurrences.

5.3.6. Simulation results from loadings of vehicles 5

The fifth situation analyzed is two type-5 vehicles, one on each lane and located at midspan, simulated for 3000 sample points. For 3000 sample points generated, 173 sample points were eliminated, 71 of them due to a low random number generated, resulting in a negative value obtained from the correspondent inverse cumulative distribution, as well as 102 of them due to a high random number generated, resulting in a TLV higher than 19.900 meters. For the remaining 2827 sample points generated, it is possible to observe from Table C5 that the variables that will be considered in the simulation process for vehicle type 5 are independent from one another, since the correlation values obtained by Equation 3.23 tend to zero. Therefore, the variables that characterize vehicle type 5 can be considered basic variables.

After obtaining the σ_{sim} values for each sample point, it was possible to observe that there were no failure occurrences.

$$\left(\sum_{i=1}^{3000} I(\boldsymbol{X}_i)\right)_{5-5} = 0$$

$$\left(p_f\right)_{5-5} = 0$$

Analyzing the values of σ_{sim} obtained from each sample point, it is possible to observe that the maximum value obtained is 12.11 times lower than the value of σ_{LS} for the longitudinal beams. From these values, it is possible to conclude that the Várzeas bridge is safe when subjected to static loadings from vehicles of type 5 and to the loads that result from the structure's components' weights.

Figure 5.5 shows the histogram for the σ_{sim} obtained from all the simulations made for this situation, as well as the number of events for each range.

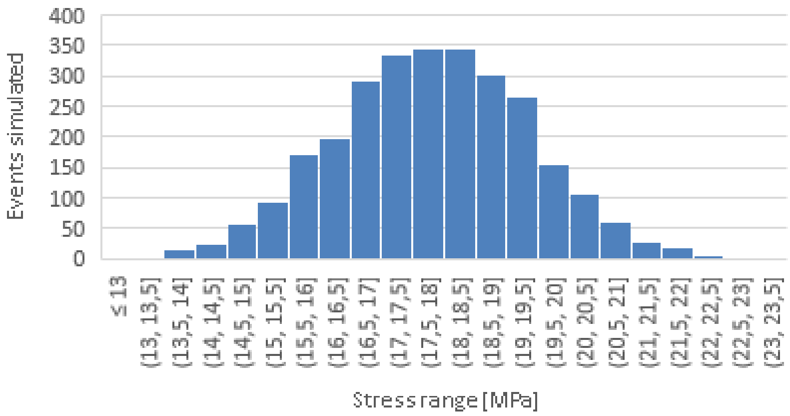

Figure 5.5- Histogram of the stresses obtained from the simulations made for vehicle type 5

From the simulations made, the minimum value obtained was 11.969 MPa and the maximum stress value obtained was 23.127 MPa. From Figure 5.5, it is possible to observe that the values of σ_{sim} obtained are defined by a normal distribution with a mean value of 17.815 MPa and a standard deviation of 1.555 MPa, as well as that the stress range with a higher number of events simulated are]17.5;18.0] MPa and]18.0;18.5], with 345 occurrences each.

5.3.7. Simulation results from loadings of vehicles type 6

The sixth and final situation analyzed is two type-6 vehicles, one on each lane and located at midspan, simulated for 3000 sample points. For 3000 sample points generated, 40 sample points were eliminated due to a low random number generated, resulting in a negative value obtained from the correspondent inverse cumulative distribution. For the remaining 2960 sample points generated, it is possible to observe from Table C6 that the variables that will be considered in the simulation process for vehicle type 6 are independent from one another, since the correlation values obtained by Equation 3.23 tend to zero. Therefore, the variables that characterize vehicle type 6 can be considered basic variables.

After obtaining the σ_{sim} values for each sample point, it was possible to observe that there were no failure occurrences.

$$\left(\sum_{i=1}^{3000} I(\boldsymbol{X}_i)\right)_{6-6} = 0$$

$$\left(p_f\right)_{6-6} = 0$$

Analyzing the values of σ_{sim} obtained from each sample point, it is possible to observe that the maximum value obtained is 6.40 times lower than the value of σ_{LS} for the longitudinal beams. From these values, it is possible to conclude that the Várzeas bridge is safe when subjected to static loadings from vehicles of type 6 and to the loads that result from the structure's components' weights.

Figure 5.6 shows the histogram for the σ_{sim} obtained from all the simulations made for this situation, as well as the number of events for each range.

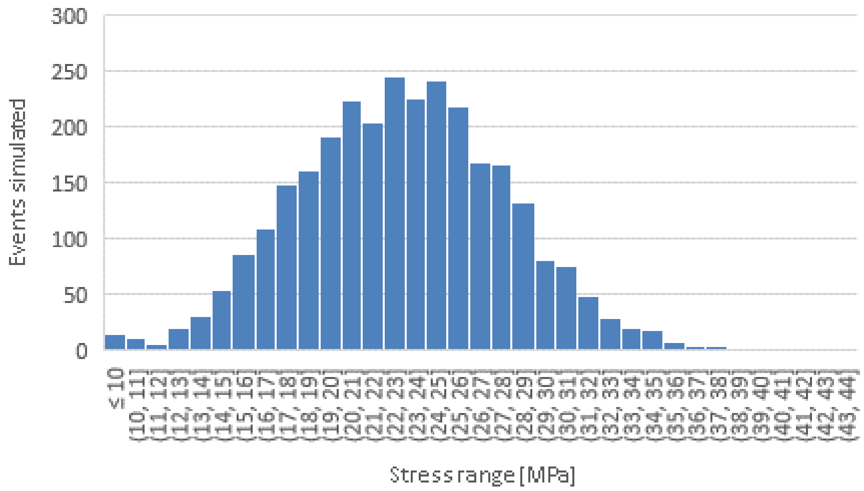

Figure 5.6- Histogram of the stresses obtained from the simulations made for vehicle type 6

From the simulations made, the minimum value obtained was 4.261 MPa and the maximum stress value obtained was 43.748 MPa. From Figure 5.6, it is possible to observe that the values of σ_{sim} obtained are defined by a normal distribution with a mean value of 22.873 MPa and a standard deviation of 4.892 MPa, as well as that the stress range with a higher number of events simulated are]22.0;23.0] MPa, with 245 occurrences each.

5.3.8. Frequency density of results obtained for each type of vehicle

For the results obtained, it is possible to observe that the dispersion of the σ_{sim} results increases with the number of variables that characterize a vehicle. Figure 5.7 shows a graphic representation of the probability density function for each vehicle type.

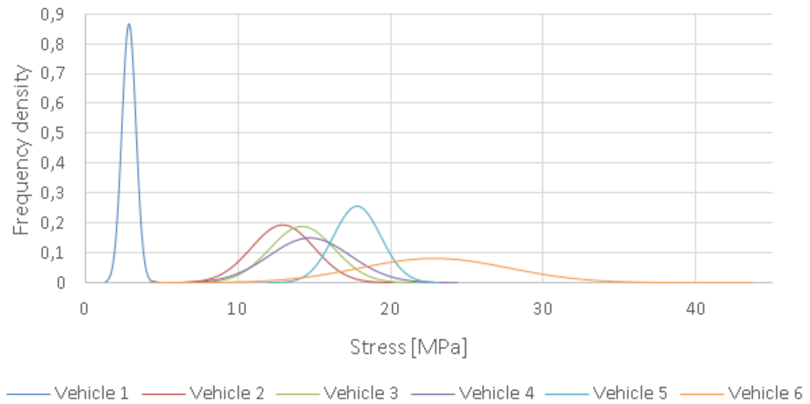

Figure 5.7- Probability density function of the σ_{sim} results obtained for each vehicle type

Through Figure 5.7, it is possible to observe that the σ_{sim} results obtained for each type of vehicle have a decreasing frequency density with the increase of basic variables needed to characterize a vehicle type. This can be emphasized when comparing the probability density functions of vehicle types 1 and 6, where it is possible to observe that the σ_{sim} results obtained have a higher dispersion value with the increase of the number of basic variables that characterize a vehicle type.

When trying to understand the effect of the basic variables AP_{ai} on the σ_{sim} results obtained, this can be achieved by comparing the probability density function of vehicle types 3 and 4. These vehicle types are characterized by the same number of basic variables, where the distribution types and parameters of AW_{ai} are identical, and the distribution types and parameters of AP_{ai} will result in a higher TVL for the vehicle type 4 sample points.

Through Figure 5.7, it is possible to observe that the dispersion of the σ_{sim} results obtained is higher for the vehicle type 4, showing the influence of TVL on the results obtained through computational simulation.

5.4. Traffic spectrum for the first year

For the purpose of this study, when establishing the yearly stress spectrum for the Várzeas bridge case study, it is considered that the daily stress spectrum will be considered as a constant, with each cycle consisting of two vehicles of the same type crossing the midspan section of the Várzeas bridge simultaneously.

Considering that there will be a total of 300 heavy vehicles crossing the bridge in a single day, and according to the traffic volume percentages obtained from [20], the first approach discussed consisted of dividing the vehicle types into heavy and non-heavy vehicles and the number of vehicles of each type crossing the bridge in a single day was extrapolated from the percentage in the cases, and where the number obtained is not integer, its value has been rounded up.

From the results of σ_{sim} obtained through computational simulations, it is possible to observe that only vehicles of type 1 can be considered as non-heavy vehicles, but only when the sum of the axle weights does not exceed the value corresponding to a GWV of 3500 kg. Therefore:

$$\sum AW_{i1} < m.g \tag{5.2}$$

$$\sum AW_{i1} < 3500 * 9{,}81 = 34335N$$

Knowing the number of vehicles of each type crossing the Várzeas bridge in a single day, and that the simulations made consider two vehicles simultaneously over the midspan section of the bridge, the number of samples needed to develop the daily stress spectrum is determined by dividing in half the number of daily vehicles of each type.

The number of vehicles of each type that will be considered to cross the Várzeas Bridge in a day and the number of sample points needed for the daily stress spectrum are presented in Table 5.1.

Table 5.1- Vehicles crossing the Várzeas Bridge in one day

Vehicle type	**Traffic volume [%]** [20]	**Daily vehicles** [50]	**Sample points required**
1	22.59	88	44
2	39.56	154	77
3	4.62	18	9
4	3.08	12	6
5	28.14	108	54
6	2.01	8	4
Total	**100**	**388**	**194**

To determine which simulated events would be considered in the daily stress spectrum, the adopted approach consists on randomly selecting which sample points of each vehicle type would be considered. In the specific case of the simulations regarding the vehicle type 1, points to be selected will only be the ones where the consideration for defining non-heavy vehicles is verified. This approach is widely accepted but has the disadvantage of having a high probability of not including the higher values obtained from the computational simulations made, especially in the cases where there is a high dispersion of the obtained results.

For the purpose of fatigue assessment of the longitudinal beams of the Várzeas bridge, it was considered that these components are subjected to a pulsating cycle where the maximum stress is the σ_{sim} value obtained through computational simulation and the minimum stress is zero. Therefore, through Equation 3.37:

$$\Delta\sigma = \sigma_{sim}$$

After selecting which values will be considered in the daily stress spectrum, in order to determine the number of cycles when a certain σ_{sim} value was achieved or surpassed throughout the first day of analysis, the values of the daily stress spectrum were analyzed with the *COUNT.IF* function of Microsoft Excel. This allowed to obtain the number of cycles when a certain σ_{sim} value was achieved or surpassed throughout the first year by multiplying the number of occurrences of the daily stress spectrum by 365 days.

Through this approach, it was possible to create the fatigue curve for the cyclic loading on the longitudinal beams for the first year, presented in Figure 5.8.

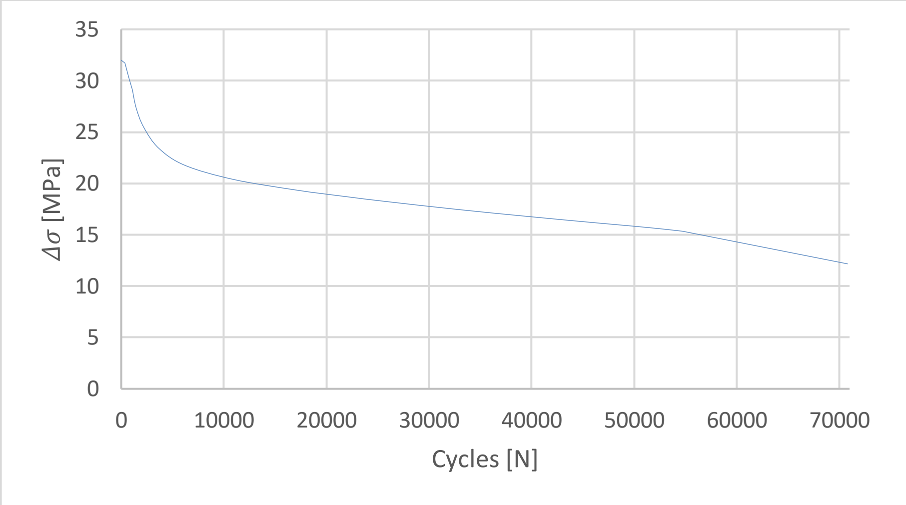

Figure 5.8- Fatigue curve for the first year of the Várzeas bridge

5.5. Fatigue assessment considering yearly traffic increases

With the economic development of a country, one of the most relevant anthropological factors has been the increase of automobile sales. This factor has been proven worldwide and is an important consideration when designing an infrastructure to be included in the road network.

Taking into consideration yearly increases of vehicles crossing the Várzeas bridge, VESAM adopted into the original Várzeas bridge's project the requirements from [50]. For the type T6 bridges, this document sets the yearly increase in heavy vehicle's traffic at a constant value of 3% per year. The former JAE established this value taken into considerations several factors, such as the variation of the Gross Domestic Product in Portugal and of the price of Brent, among others.

In this regard, it will be analyzed the increase of traffic for years 1, 2, 5, 10, 25, 50, 75 and 100 from both the loads in that year and the cumulative loads of the Várzeas bridge since the first year until the year to be analyzed. The values of vehicles that cross the mid span section of the Várzeas bridge for each year, both referring to only the traffic for that year and the cumulative traffic up until that year are presented in Appendix D. Figure 5.9 and Figure 5.10 illustrate the fatigue curves for the yearly cyclic loading and the cumulative cyclic loading up to the previous mentioned years, respectively.

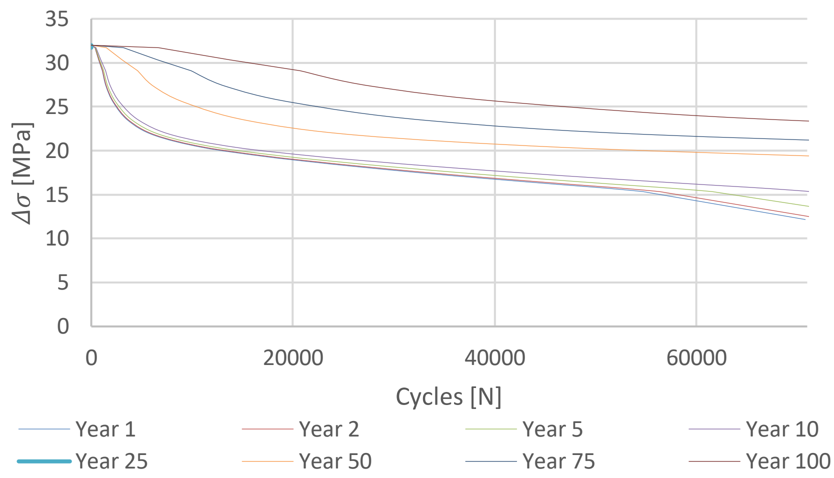

Figure 5.9- Yearly fatigue curves, considering a traffic increase of 3% per year

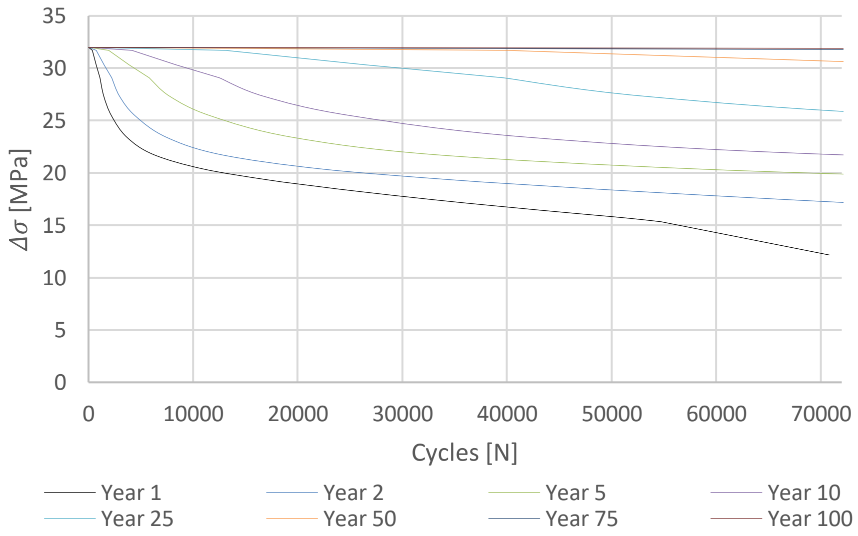

Figure 5.10- Yearly cumulative fatigue curves, considering a traffic increase of 3% per year

Comparing the results from Figures 5.9 and 5.10, it is possible to observe that the cumulative effect of cyclic loading has. For the purpose of assessing the fatigue behavior of the longitudinal beams to cyclic loads, the fatigue curve representing the cumulative effect of cyclic loading has on the components on the 100th year will be compared to a theoretical fatigue curve for the material S355 at $R_{\sigma} = 0$. This fatigue curve was developed by Stranghoner & Jungbluth [51], which analyzed ten specimens with a thickness of 40 mm, tested with an applied force of 1.4 MN at a frequency of 9 Hz and the experiments were stopped at 2*10^6 cycles or at the moment where failure occurred. These authors then established the following equation to characterize the obtained fatigue curve.

$$\log(N) = 15.133 - 4 * \log(\Delta\sigma)$$

This curve is characterized by a σ_e of 192 MPa. Therefore, if a fatigue failure were to occur, this value had to be surpassed, otherwise the longitudinal beams of the Várzeas bridge can be considered safe. In this regard, Figure 5.11 shows a representation comparing the values obtained from the theoretical fatigue curve for the material S355 and the fatigue curve for a cumulative effect of cyclic loading for the 100th year.

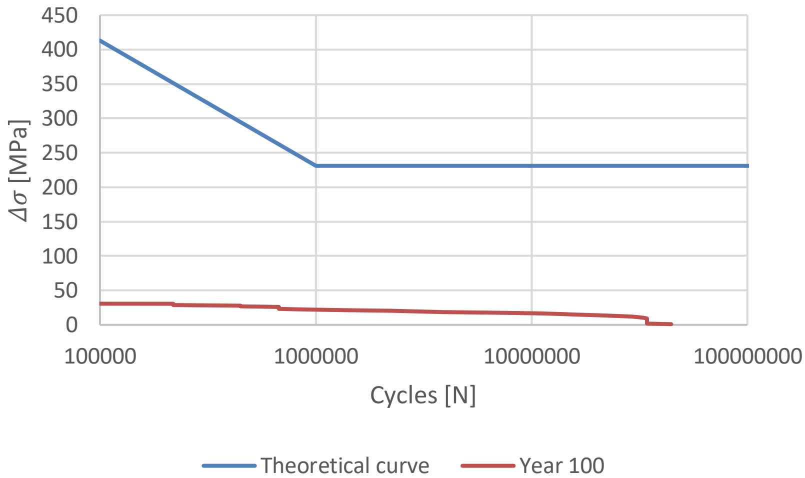

Figure 5.11- Theoretical S355 fatigue curve vs. cumulative cyclic loading curve for year 100

From Figure 5.11, it is possible to see that the value of σ_e from [51] is not surpassed by the cumulative cyclic loading up until the 100th year. Therefore, the longitudinal beams of the Várzeas bridge are considered safe to cyclic loading of this sort for an infinite life span. From Equation 3.41:

$$N_f = \infty$$

$$Damage = 0$$

Although an important assessment parameter, it is important to refer that this consideration is only valid when dealing with this type of loads. Phenomena such as corrosion will inevitably influence this assumption and need to be addressed by designers in future works.

5.6. Results discussion

From the present study, it was possible to understand that the longitudinal beams of the Várzeas bridge, considered as the components of the structure, is safe when subjected to static loading, both in the first year of its use and throughout the years. Although it was a thorough study, several situations need to be addressed in order to put the results obtained into context.

In terms of the static loading to which the computational model of the Várzeas bridge was subjected to, it is important to refer that the basic variables that defined the different types of vehicles were adapted from the previous work made in [20]. These loads were obtained by assessing the traffic from a long-span girder bridge with six lanes, and that every vehicle that registered a front axle weight higher than 89 kN eliminated from the database, being considered a measurement error. Although accepted due to the low probability of such an event occurs, it limits the range to which the vehicle loads created for the purpose of this study. Furthermore, it is important to refer that the design specifications for vehicles in the USA are different in terms of distance between axles than the ones used in the EU, in which these values are usually higher for vehicles made in the USA. This translates in values of the several AP_{ai} higher than the ones that would be expected when analyzing the actual loads. This translates in a higher value of TVL, which in turn will influence the value of σ_{sim} obtained, as seen when comparing the results from the simulations made for vehicle types 3 and 4.

In terms of data treatment, due to the lack of other criteria that establishes a range in which the values obtained from the several inverse cumulative functions of AW_{ai} and AP_{ai} are considered deemed, it were only eliminated the sample points where a negative value was obtained from these inverse cumulative functions, as well as the sample points where the value of TVL were higher than the total span of the bridge (19.9 meters). In this regard, it was possible to observe that all variables considered for the purpose of this study were deemed as independent (basic) and that the values of σ_{sim} obtained through computational simulation had a higher dispersion with the increase of variables needed to define a vehicle type. This was also possible to observe through the graphical representation of the values of σ_{sim} obtained from each type of vehicle.

In terms of failure occurrence, it was possible to observe that there were no occurrences registered for the sample points simulated. This can be translated by not existing any situation where the value of σ_{sim} is higher than the value σ_{LS} established as the limit state for the purpose of this study. Therefore, it is possible to conclude that the longitudinal beam of the Várzeas bridge are safe when subjected to static loads.

In terms of fatigue behavior for the first year, when establishing the daily stress spectrum for the first day as a constant and extrapolating the results for periods up to 100 years, it is possible to observe that the longitudinal beams' S-N curve never violated the considerations adopted for the fatigue behavior of the steel S355. As time goes by, the yearly loads for the years analyzed, both individual and cumulative, never violated the fatigue limits for the longitudinal beam's material. It is important to refer that, for the considerations made for the purpose of this study, it was obtained a total number of vehicles crossing the midspan section of the Várzeas bridge of 88707376 vehicles (presented in Table D2), with no fatigue failure registered for the longitudinal beams. This can be translated in a high degree of safety by the designers of the original Várzeas bridge's design.

When confronting VESAM Group with the results obtained from this study and comparing with the methodology used in the original design, it was referred that it was indeed adopted with a high degree of safety when designing the Várzeas bridge, but several other situations need to be taken into consideration in order to put these results into perspective.

The first consideration refers to the legal requirements in order to build a bridge in an EU member-state. These structures need to be designed in a way that accomplishes the requisites established in the Eurocodes documentation, namely Eurocodes 1 (loading) and 3 (steel requirements). When implementing these standards, the response of a structure must analyze not only its response to static loads, but other factors such as lateral buckling, buckling due to bending and torsion, environmental effects resulting from temperature, wind velocity, earthquakes and flooding, exposure to elements that enable corrosion mechanism, among others. These factors were not the subject of this Master dissertation but can be considered an explanation for such a high discrepancy between the values of σ_{sim} obtained through computational simulations and the σ_{LS} value, defined as a limit state.

A final consideration regards the financial aspects of building such a structure. Before building a structure, VESAM Group enters a bid in which the design of the structure must be both up to par with the buyer's expectations and financially appealing. Regarding the latter point, the total cost of the structure's construction, in which the acquisition of materials is one of the most important to cost estimation, must be both financially appealing to the buyer and able to maximize the profit for the company.

5. Conclusions

This chapter presents the conclusions of the study, referring its importance to understand the behavior of the longitudinal beams of the Várzeas bridge when subjected to static loads, as well as to understand fatigue behavior throughout the years analyzed. Following these conclusions, the limitations of this study are presented, making it possible for the reader to understand all the factors that can influence the obtained results and that were out of the purpose of this study. Finally, several suggestions are presented for further work on the matter of assessing the structural behavior of the Várzeas bridge.

6.1. Conclusions

This Master dissertation explored several research methods and techniques widely accepted academically, in order to assess the behavior of the longitudinal beams of the Várzeas bridge when subjected to static loading. In this regard, it was proposed a method to estimate the daily stress spectrum based on the traffic distribution per vehicle type and the effects of vehicle parameters in Autodesk ROBOT Structural Analysis.

When researching about past bridge failures and collapses, as well as both possible causes and consequences of these events, it was important to realize the level of importance such as asset has in a country's road network and the impact of such an event on both the responsible entities and the society as a whole. This translates in a level of safety in the design phase of a bridge, aiming to develop a safe structure to all kinds of analyzed loads. Although structural safety, maintenance and reliability are important fields, the effects of the economic crisis of 2008 still affect the way several countries look at the current structural condition of its bridges, making it imperative to address these issues as soon as possible in order to avoid another collapse, such as the Morandi bridge collapse in 2018.

The main goals of a company when making a bid to build a bridge are to meet the safety and structural requirements from both national and international standards, aiming to provide the end-user a safe structure and able to mitigate factors such as traffic congestions, environmental impacts and large changes in the landscape, as well as being able to maximize the company's profit while keeping it competitive. One of the main factors in terms of maximizing a company's profit when building such a structure is through the total weight of the components, since the materials are usually priced in terms of mass (e.g. euros per kilogram of material). When respecting the design requirements, this approach can result in over dimensioning the structure, which in turn will add an extra layer of safety to the structure but is in some way unnecessary from a design perspective.

The main goal of this Master dissertation was to create a design of computational experiments for fatigue assessment of the four longitudinal beams of the Várzeas bridge, made from a S355 steel, when subjected to static loading from two vehicles of the same type, both placed with its geometric center coincident with the midspan section of the bridge, as well as to the weight of every component of the Várzeas bridge. Due to the lack of a database to represent the actual vehicle load conditions the Várzeas bridge is subjected to, the information of the independent (basic) variables that characterize a vehicle by its type and the average number of daily vehicles a bridge such as this one is subjected to was adapted from the literature. This made it possible to generate a large amount of sample points, which in turn made it possible to assess the probability of failure of the longitudinal beams of the Várzeas bridge with a high level of precision. The stress values for each sample case were obtained through computational simulation using a computational model of the Várzeas bridge and then compared with a stress value deemed as *limit state*. This computational model was previously optimized by VESAM Group and is widely used by the company, enabling it to reproduce static load occurrences in a timely manner and without allocating resources to the location of the actual structure.

As initially expected, no failure occurrences were detected, which could be explained by the level of safety adopted in the original design of the Várzeas bridge, as well as due to several other structural assessment analyzes made in the original design of the structure in order to meet the requirements established by the Eurocode standards.

With the stress values obtained from the simulations, along with several criteria obtained from the literature, it was possible to determine a daily traffic spectrum, which was set as a constant and enabled to extrapolate the yearly traffic spectrums for several years, both in terms of cumulative effects and only for the yearly traffic. This made it possible to determine the fatigue curve for the loading and, when compared to the fatigue curve of the material S355 obtained from the literature and created for a pulsating cyclic loading, it was possible to conclude that the longitudinal beams are designed for infinite life when subjected to the static loadings of the several traffic spectrums analyzed.

In conclusion, the design of a structure with the importance of a bridge must take into consideration a certain level of safety, such as determining a design factor of safety deemed as acceptable, in order to avoid any situation that could result in a performance deterioration and can result in failure and/or collapse. In terms of EU member-states, these structures must meet the requirements of the Eurocode standards, which in turn can be considered as conservative in its analyzes. These approaches must be complemented by a well-planned public investment strategy in order to assess the actual structural condition of a country’s bridges and enabling the responsible entities to act in time, preventing situations that can result in large repercussions in the future.

6.2. Limitations

When dealing with a structural reliability problem from an academical perspective, several limitations need to be addressed in order to establish the groundwork for future studies.

For the purpose of this Master dissertation, the most important limitation regards the traffic data involved, which was adopted from the literature and considers a traffic spectrum that can be very different to the actual loadings the bridge is subjected to. This assumption was made because the Várzeas bridge does not have a BWIM system implemented, which makes it impossible to collect data regarding a vehicle's axle weight and distances between axles. The absence of a BWIM system also has the disadvantage that the loads modeled cannot be compared to the actual responses a system like this one would provide. Without additional information, it is assumed that the values registered by [20] are accurate.

In terms of the parameters that define a vehicle type, it is important to refer that the literature adopted for the purpose of this study deals with vehicles from the USA. This can create a transferability problem when dealing with European bridges, since the design specifications of EU and USA manufacturers are different for both axle weights and distance between axles values. It is expected that it will not greatly influence the results but needs to be addressed in future situations.

Regarding the behavior of the longitudinal beams of the Várzeas bridge, it was established that the limit state for static loadings was a constant stress value of 280 MPa, as established by VESAM Group in the original design of the structure. Although its value from an academical perspective is important, the material's parameters may suffer alterations throughout time.

Additionally, the subject of this study were the longitudinal beams of the Várzeas bridge, considered as the critical components of the structure. This can represent the basic effects of structural reliability of the structure but does not represent the structure as a whole. To overcome this limitation, it is important to address all the components of the Várzeas bridge.

Due to the use of a finite element model to simulate the response of the vehicle loadings on the bridge, it is important to take into consideration the differences between the computational model and the actual Várzeas bridge. In this regard, the finite element model will not take into consideration the weights of several structural components of the actual bridge (screws, rivets, etc.), as well as the effects between the tar and the other components, which will give the actual Várzeas bridge an extra stiffness that is not included in the computational model. the kind of discrepancies that can result from using a computational model of a structure.

In terms of loading, the bridge will only be subjected to the dead loads of the weights of its components and the static traffic loads, with the effects of temperature and wind on the structure being omitted. Although there are cases where the effects of temperature and wind can combine to about 30 percent of the total loads the actual Várzeas bridge may be subjected to, when consulting the database of the *Portuguese Institute for Sea and Atmosphere* (IPMA), there are not any extreme peaks of wind velocity and temperature registered in the region where the Várzeas bridge is located than can be deemed as influential. This can translate in a possibility of omitting these effects, since its influence would be low. Other effects, such as the effects of earthquakes, floods and exposure to corrosive environments are deemed as requirements that need to be addressed by several standards but were omitted for not being the purpose of this study.

In terms of fatigue assessment of the longitudinal beams of the Várzeas bridge for a period up to 100 years, the daily stress spectrum was set as a constant. This assumption allows to address the goals of the study but does not allow to determine a confidence interval por these values. To overcome this limitation, a possible approach is to establish a longer period as a constant (e.g. one year) and develop a daily stress spectrum for each day and determine the mean and standard deviation values for each sample point. With these parameters, it is possible to determine the confidence interval of the stress curve for one year for a certain degree of confidence.

Finally, this study only deals with static loads and the dead load from the structure's weight, as well as the fatigue assessment of the longitudinal beams throughout the years. To perform a thorough assessment of the bridge's structural integrity, resembling the real loading to which the structure will be subjected to, further studies need to take into consideration the dynamic effects of moving vehicles which, among other effects, will add horizontal loadings in both moving and breaking situations, as well as possible impact loads from road accidents. After confronting VESAM Group with the results of this study, it was referred that additional analyzes were made while originally designing the Várzeas bridge, such as lateral buckling and buckling due to a combined effect of bending and torsion, among other requirements from the Eurocode standards and other documentation. These analyzes need to be addressed in future studies in order to assess the actual level of safety that the original design of the Várzeas bridge has.

6.3. Future work

The field of structural reliability is large, and it will be the subject of further developments in the years to come. To further develop this study, several other analyzes need to be addressed in order to correctly assess the structural behavior of a bridge. The effects of environmental effects (e.g. temperature, wind velocity, earthquake action, floods, exposure to corrosive environments, etc.) and analyzing the effects of dynamic loads, breaking loads in terms of pavement degradation and impact loads from accidents are important to further assess the structural response of a bridge.

In terms of developing future works based on the design of computational experiments presented in this study, it is required to determine other conditions deemed as limit states (e.g. deformations at mid-span) to assess the structural reliability of the bridge when subjected to other effects. This can be done by implementing a Failure Mode, Effect and Criticality Analysis (FMECA).

When analyzing a structure when subjected to loads, it is important to develop an algorithm that can address all its components. This is a daunting task but can be addressed by analyzing the components individually and further develop an algorithm that deals with the structure though series and parallel groups of components. The work of (25) is an important progress on this matter, with Figure 6.1 illustrating the example of this author.

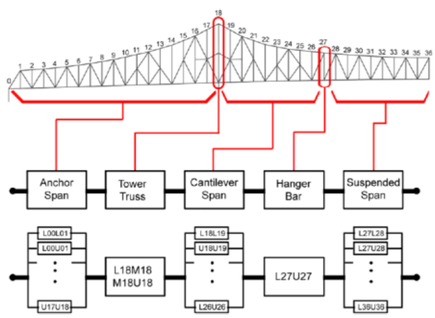

Figure 6.1- Structural reliability of a bridge through combinations of components [21]

From Figure 6.1, it is possible to observe that this methodology, while considering several simplifications, can assess the structural response of an entire structure with a good level of precision.

In terms of simulation techniques, an alternate approach to increasing the sample size when using MCS in order to obtain a convergent probability of failure is through variance reduction, achieved by using additional (a priori) information about the problem in hands. This is the basis for methods such as *Importance Sampling* (IS).

Figure 6.1 shows a schematic representation of the region analyzed by only MCS and MCS with IS.

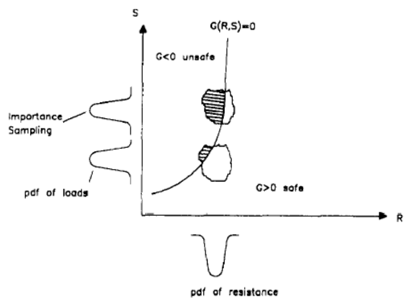

Figure 6.2- Schematic representation of MCS and MCS with IS (27)

Figure 6.2 illustrates that IS takes special emphasis in the region where the probability of failure is most significant, enabling to optimize computational problem by analyzing fewer sample points, only from a specific region where failure occurs.

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Appendix A- Bridge condition in different European countries

Table A1- Current situation and approved measures in several countries [52]

Country	Present situation	Approved measures
Portugal	• Lack of publicly available data created problems to implement maintenance strategies; • Problems regarding the 25 de Abril bridge and the Duarte Pacheco viaduct have flourished.	• Pressure from national parliament to publish data about the current state of these bridges.
France	• One third of the 12000 bridges in the country need reparation works and 7% of them can lead to an eventual collapse if ignored; • The average period of bridge reparation after detecting deterioration is of 22 years; • The budget available for short term investments is about half of what is estimated to be needed.	• New legislation package regarding public infrastructures planning approved; • 1 billion euros package destined to urgent repairs approved.
Germany	• Last renovation program took place throughout the 1990's; • 12.5% of the bridges are safe; • 12.4% of the bridges are in severe state and/or with traffic restrictions.	• 1.3 billion euros package destined to inspection and reparation actions approved; • Legislation package that mandates that a bridge must be thoroughly inspected every 6 years approved;
Spain	• Most of the bridges were built after the country's EU entrance; • Lack of public data of maintenance actions made since.	• National parliament has pressured the government to publish data about the current state of the national road network.
Bulgaria	• Over 200 bridges are in severe state.	• National government ordered the maintenance and repair of more than 200 bridges simultaneously, even if that requires the need of new financial loans from foreign entities.
Netherlands	• Heavy vehicles restriction in several bridges surrounding Amsterdam.	• National maintenance plan updated.

Appendix B- Bridge condition assessment techniques

Table B1- Description of the most common bridge condition assessment techniques [12]

Technique	Description
VI	Trained engineers recognize, register and evaluate the physical condition of different bridge elements using inspection manuals and defined codes. The primary and most common interval for inspections is 24 months.
LTR	Determine the live-load carrying capacity on an existing bridge by measuring the actual load the bridge can carry without distress. Condition ratings can be determined by allowable stress, load factor or load and resistance factor methods.
SHM	Encompasses a range of methods and practices designed to capture structural response, detect anomalous behavior and to assess the bridge condition based on a combination of measurement, modelling and analysis.
NDE	Several techniques introduced exploit various physical phenomena (acoustic, seismic, electric, electromagnetic, thermal, etc.) to detect and characterize deterioration processes without damaging the elements.
FEM	Numerical analysis to investigate the behavior and response of a bridge structural system. Usually calibrated using results of field inspection supported by NDE technologies or by static or dynamic tests on the structure.

Table B2- Advantages of the most common bridge condition assessment technique [12]

Technique	Advantages
VI	• Most significant aid for bridge condition evaluation. • BMS rely primarily on VI to record bridge components condition ratings, which are quantified and standardized through a priority-ranking procedure.
LTR	• Safe and conservative analysis methods. • The governing rating is the lesser of the shear capacity of the critical bridge component. • Development and updating the loading rating software is undertaken by AASHTO.
SHM	• Reliable and potentially real-time bridge assessment. • More meaningful than using load response data. • Can be deployed for short and long-term assessment. • Most appropriate method for movable bridges.

Table B2- Advantages of the most common bridge condition assessment technique(cont.) [12]

NDE	• Provide effective and accurate condition assessment. • Objectify the inspection process and make it faster and more reliable. • Integration of different techniques is best approach to identify several different damage states.
FEM	• Allows detailed visualization. • Can be created using data from visual inspection and then parameterized and calibrated using information from NDE and SHM. • Able to satisfactory capture short-term performance (e.g. load test).

Table B3- Limitations of the most common bridge condition assessment techniques [12]

Technique	Advantages
VI	• Subjective evaluation, results depending greatly on the qualifications of those conducting inspections and may vary when the person is not the same. • Considers only the observed physical health of the bridge and cannot detect hidden defects.
LTR	• Costly and time consuming. • Different rating methods may lead to differently rated capacities and posting limits for the same bridge. • No guidance to which method should be used for a specific situation.
SHM	• Wireless sensors rely on battery autonomy and power. • The size and complexity of the bridge being monitored could result in complex systems. • Often creates reliability issues. • Requires routine, on-site maintenance to support long-term operation.
NDE	• Applying only one technology provides limited information about the bridge condition. • No single technology can identify all deterioration phenomena. • Requires trained personnel for data collection and analysis.
FEM	• FE models typically require calibration. • Long-term assessment is a challenge due to advances in structural materials and construction methods.

Appendix C-Correlation between variables

Table C1- Correlation matrix between vehicle type 1 variables for the sample points generated

	AW_{11}	AW_{12}	AP_{11}
AW_{11}	1.000	-0.036	0.009
AW_{12}		1.000	0.008
AP_{11}			1.000

Table C2- Correlation matrix between vehicle type 2 variables for the sample points generated

	AW_{21}	AW_{22}	AW_{23}	AP_{21}	AP_{22}
AW_{21}	1.000	0.001	0.015	0.011	0.023
AW_{22}		1.000	-0.017	0.001	-0.001
AW_{23}			1.000	0.010	-0.013
AP_{21}				1.000	0.021
AP_{22}					1.000

Table C3- Correlation matrix between vehicle type 3 variables for the sample points generated

	AW_{31}	AW_{32}	AW_{33}	AW_{34}	AP_{31}	AP_{32}	AP_{33}
AW_{31}	1.000	0.018	-0.004	-0.011	0.056	0.027	0.030
AW_{32}		1.000	0.028	0.008	0.001	0.007	0.014
AW_{33}			1.000	-0.002	-0.013	0.044	0.013
AW_{34}				1.000	-0.016	0.007	0.004
AP_{31}					1.000	0.018	-0.008
AP_{32}						1.000	0.047
AP_{33}							1.000

Table C4- Correlation matrix between vehicle type 4 variables for the sample points generated

	AW_{41}	AW_{42}	AW_{43}	AW_{44}	AP_{41}	AP_{42}	AP_{43}
AW_{41}	1.000	-0.004	-0.018	0.012	0.028	0.008	0.053
AW_{42}		1.000	-0.025	-0.020	-0.003	0.014	0.011
AW_{43}			1.000	-0.011	0.017	0.031	0.029
AW_{44}				1.000	-0.008	0.011	0.021
AP_{41}					1.000	0.039	0.101
AP_{42}						1.000	0.044
AP_{43}							1.000

Table C5- Correlation matrix between vehicle type 5 variables for the sample points generated

	AW_{51}	AW_{52}	AW_{53}	AW_{54}	AW_{55}	AP_{51}	AP_{52}	AP_{53}	AP_{54}
AW_{51}	1.000	0.007	0.018	0.020	0.014	-0.012	0.085	0.041	0.029
AW_{52}		1.000	0.018	0.024	-0.010	0.013	0.023	0.001	0.049
AW_{53}			1.000	0.008	-0.020	0.025	0.026	0.027	0.004
AW_{54}				1.000	0.010	-0.024	-0.021	0.012	-0.023
AW_{55}					1.000	0.014	0.014	-0.012	0.018
AP_{51}						1.000	0.029	-0.067	0.018
AP_{52}							1.000	0.089	0.246
AP_{53}								1.000	0.077
AP_{54}									1.000

Table C6- Correlation matrix between vehicle type 6 variables for the sample points generated

	AW_{61}	AW_{62}	AW_{62}	AW_{64}	AW_{65}	AW_{66}	AP_{61}	AP_{62}	AP_{63}	AP_{64}	AP_{65}
AW_{61}	1.000	-0.016	0.016	0.013	0.032	0.013	-0.022	0.038	0.003	0.019	0.001
AW_{62}		1.000	0.022	0.001	0.023	-0.004	0.014	0.026	-0.018	0.029	0.042
AW_{63}			1.000	-0.007	0.008	-0.028	0.001	0.023	-0.034	0.025	0.004
AW_{64}				1.000	0.012	0.001	-0.007	0.026	-0.025	-0.007	0.010
AW_{65}					1.000	-0.001	0.002	-0.031	0.002	0.023	0.008
AW_{66}						1.000	-0.011	0.016	-0.006	-0.002	0.014
AP_{61}							1.000	0.099	-0.010	0.036	0.027
AP_{62}								1.000	-0.009	0.100	0.115
AP_{63}									1.000	0.015	0.003
AP_{64}										1.000	0.061
AP_{65}											1.000

Appendix D- Yearly traffic increases

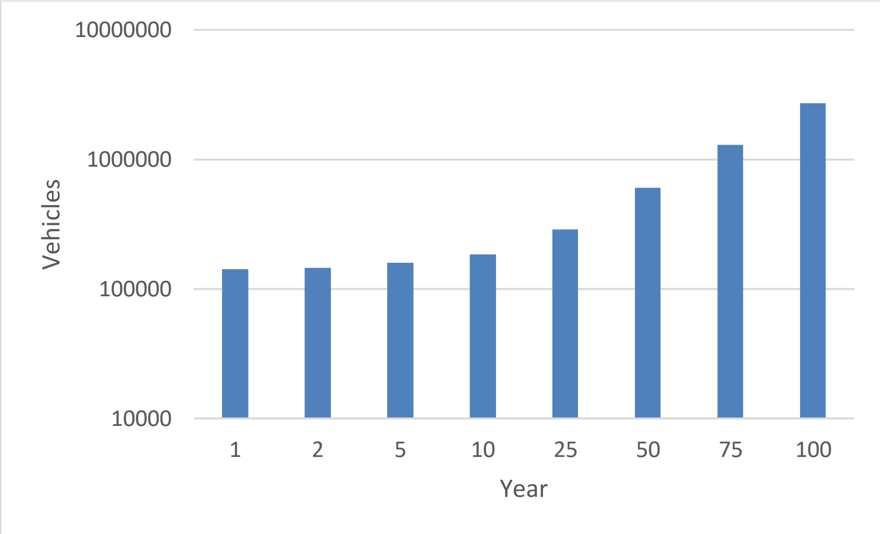

Figure D1- Yearly traffic, considering an increase of 3% per year

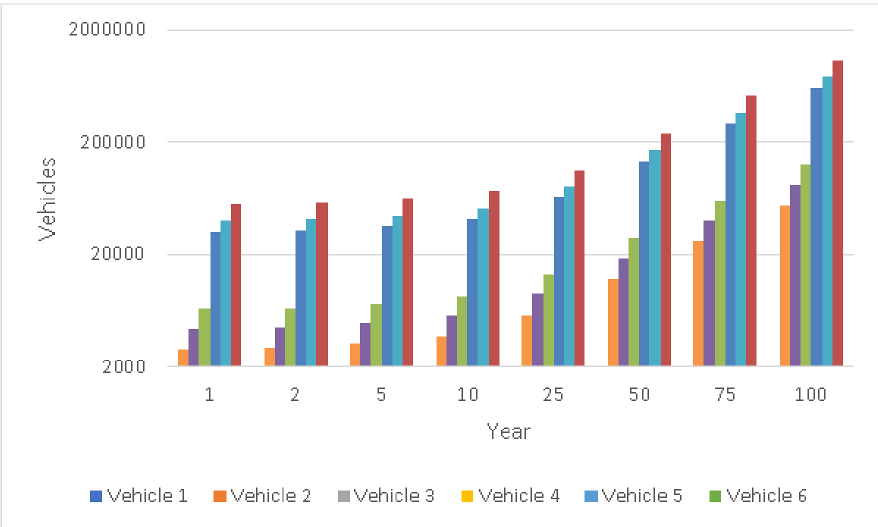

Figure D2- Yearly traffic per vehicle type, considering an increase of 3% per year

Table D1- Yearly traffic, both total and per vehicle type, considering yearly increases

Year	Vehicle 1	Vehicle 2	Vehicle 3	Vehicle 4	Vehicle 5	Vehicle 6	Total
1	31992	56025	6543	4362	39852	2847	141620
2	32952	57705	6739	4493	41047	2932	145868
5	36007	63055	7364	4909	44853	3204	159392
10	41740	73096	8536	5691	51995	3714	184772
25	65025	113873	13299	8866	81000	5786	287848
50	136139	238409	27843	18562	169586	12113	602652
75	293586	514134	60043	40029	365716	26123	1299630
100	614697	1076468	125715	83810	765718	54694	2721102

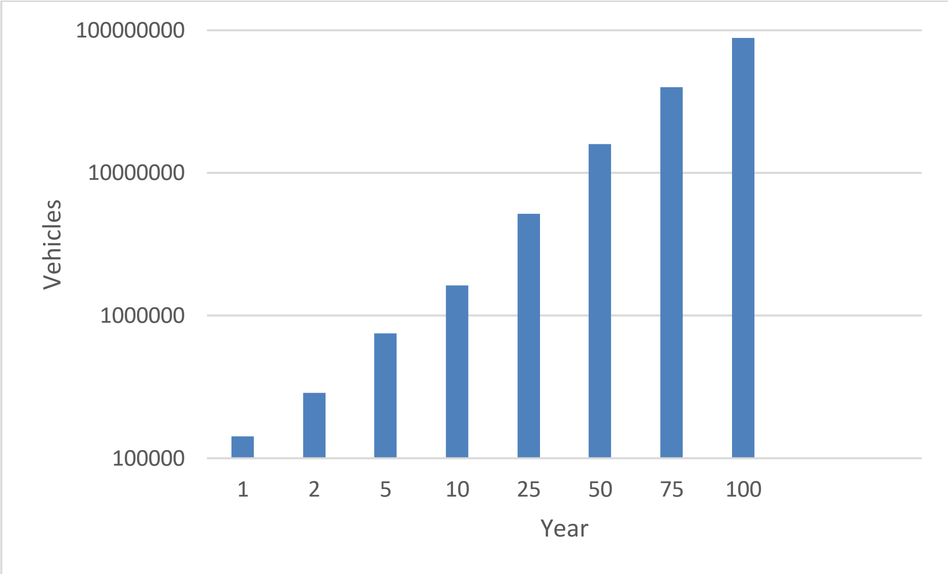

Figure D3- Yearly cumulative traffic, considering an increase of 3% per year

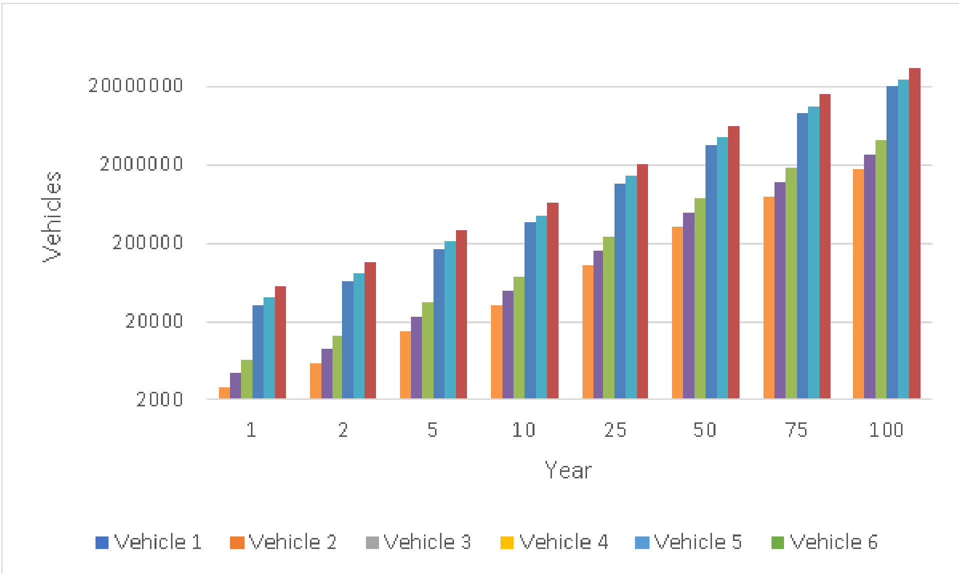

Figure D4- Yearly cumulative traffic per vehicle type, considering an increase of 3% per year

Table D2- Yearly cumulative traffic, total and per vehicle type, considering yearly increases

Year	Vehicle 1	Vehicle 2	Vehicle 3	Vehicle 4	Vehicle 5	Vehicle 6	Total
1	31992	56025	6543	4362	39852	2847	141620
2	64944	113730	13282	8855	80899	5779	287488
5	169848	297441	34737	23158	211577	15113	751874
10	366743	642246	75004	50003	456846	32632	1623474
25	1166317	2042475	238530	159020	1452863	103776	5162980
50	3608108	6318582	737913	491942	4494562	321040	15972148
75	9014018	15785505	1843504	1229003	11228618	802044	39902692
100	20038996	35092638	4098281	2732187	24962256	1783018	88707376