

Performance Assessment of a Roadway Bridge Under Seismic Events

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Thesis submitted in partial fulfillment for the degree of Bachelor of Civil Engineering

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Dedication

*This thesis is dedicated to the kindest person that I know, to the woman
whose blood gave life, the woman who always has been there for me,
to my mother.*

*I have no words that could describe how grateful I am to have not only
the most intelligent, sweet, fearsome woman in the world as my mother,
but to have a person who always puts my needs in front of hers.*

*I am forever grateful to God for having you as a mother, because you
are my strength, my inspiration, my light and my rock.*

*I hope that someday I could give you as much as you had given me,
even do I know there is not enough time for such thing.*

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Resumen

TITULO: PERFORMANCE ASSESSMENT OF A ROADWAY BRIDGE
UNDER SEISMIC EVENTS *

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PALABRAS CLAVE: ÍNDICE DE CONFIABILIDAD, EVENTOS SÍSMICOS,
PUENTES VIGA-LOSA

DESCRIPCIÓN:

Los eventos externos representan las causas más comunes de falla en puentes, lo que podría indicar colapso parcial o total (falla de todas las partes esenciales de un puente). Además, la falla principalmente se produce durante la vida útil de la estructura. Uno de los principales factores ambientales son los terremotos, que tienen un alto impacto en los puentes debido a las irregularidades presentadas en la subestructura y la superestructura y las vulnerabilidades adquiridas por los códigos utilizados en su diseño. Las estructuras ubicadas en lugares con una alta frecuencia de eventos sísmicos podrían presentar una disminución en su rendimiento debido a la reducción en la resistencia al corte del concreto. Este artículo presenta un marco para obtener el índice de confiabilidad de un puente bajo eventos sísmicos, utilizando el método de superficie de respuesta y el método de confiabilidad de primer orden, basado en variables aleatorias que afectan la capacidad de la estructura (obtenida del análisis pushover) y las cargas sísmicas (obtenidas de los datos de aceleración pico del terreno de registros sísmicos). La confiabilidad del puente del estudio de caso se actualiza utilizando técnicas de inspección visual para detectar daños que podrían disminuir la capacidad de carga de la estructura. Los resultados muestran que la zona vulnerable en el puente es su pila más corta y la falla podría ocurrir debido a la alta concentración de corte en la rótula en la parte inferior de la pila.

* Trabajo de grado

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Abstract

TITLE: PERFORMANCE ASSESSMENT OF A ROADWAY BRIDGE
UNDER SEISMIC EVENTS *

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KEYWORDS: RELIABILITY INDEX, SEISMIC EVENTS, GIRDER
BRIDGES

DESCRIPTION:

External events represent the most common causes of bridge failure which could indicate distress, partial or total collapse (failure of all substantial parts of a bridge). Furthermore, failure mainly occurs during the service life of the structure. One of the main environmental factors are earthquakes, which have a high impact on bridges due to irregularities presented in both substructure and superstructure and vulnerabilities acquired by the codes used in its design. Structures located in places with a high frequency of seismic events could present a decrease in its performance due to reduction in concrete shear strength. This article presents a framework to obtain the reliability index of a bridge under seismic events, using response surface method and first order reliability moment, based on random variables that affect the structure capacity (obtained from pushover analysis) and the seismic loads (obtained from peak ground acceleration data of seismic recordings). The bridge reliability of the case study is updated using visual inspection techniques in order to detect damages that could decrease the structure load carrying capacity. Results show that the vulnerable zone on the bridge is its shortest pier and the failure could occur due to high shear concentration in the hinge at the bottom of the pier.

* Bachelor Thesis

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Introduction

Structures around the world are designed for a life cycle. When a structure is at the end of this period, it needs to be replaced by a new one, but usually, the cost of a new structure is higher than the cost of implementing maintenance interventions to the current structure which may continue operative (Azizinamini et al., 2013).

Bridges have a higher design life than common structures. Moreover, that period of time may change between countries and characteristics of the bridge such as traffic density or alternative route. In bridges, not all components need to be designed with the same service life because some are replaceable, but the overall general design life of a bridge should be 100 years as the Eurocode recommends (CEN/TC250, 2013); other design approaches like AASHTO LRFD suggest 75 years (American Association of State Highway and Transportation Officials, 2018). The service life is a long period of time, and it is necessary to implement management strategies and maintenance actions in order to achieve its service life (Azizinamini et al., 2013).

A deteriorating structure leads to increased costs. Bridge management is fundamental to maintain a satisfactory performance of the structure; that is the reason why information regarding the state of the bridges is essential when making decisions that involve the maintenance of the structure. Visual inspection is usually the first and the predominant evaluation technique used to obtain bridge information (Moore, Phares, Graybeal, Rolander, & Washer, 2001), and it also represents lower cost than most nondestructive test (Quirk, Matos, Murphy, & Pakrashi, 2017). With the data collected from routine visual inspection, it is possible to assess performance and to

provide important information on life-cycle inspection or repair strategies (Estes & Frangopol, 2003).

Failures in structures primarily occur in its service life. The main reasons for the collapse in bridges are the external events, both natural and man-made, and the lack of maintenance; furthermore, deterioration of bridge components is also an essential cause (Wardhana & Hadipriono, 2003).

Bridges are structures that face harsh environmental condition; therefore, bridges need to be able to withstand potential hazards such as flooding, earthquakes, wind and temperature changes. These events could generate major safety problems and bridge failure, as in the case of foundation scour (Decò & Frangopol, 2011).

Seismic events can be one of the main concerns when evaluating loads of the bridge. This is due to the difficulty of taking into account an entire set of possible earthquakes that may occur in the region. In addition, seismic events produce an increase of lateral force that can induce cracks or even failure of the key components of the bridge such as the piers, the joints or, in some cases, the superstructure (Dong, Frangopol, & Saydam, 2014).

It is important to estimate the damage state of the bridge associated with an earthquake scenario because bridges designed by traditional seismic design codes do not explicitly address the hazard level or the cost of the consequences. This type of approaches are outdated in comparison with modern approaches, which are based on performance goals or the current design codes that have a mixed approach with performance requirements in the form of no collapse or damage limitation (ultimate and design limit states) (CEN/TC250, 2013; Nutt, 1996; Prendergast et al., 2018).

The assessment of bridge performance under seismic events is more significant than ever, because the structural deterioration caused by seismic events affects bridge reliability and

sustainability, especially in bridges that are at the end of their service life and were designed with outdated seismic design codes (Dong et al., 2014).

1. Aim and Objectives

1.1. Aim

To assess the structural performance of a girder bridge under seismic events, based on reliability analysis and qualitative data obtained employing visual inspection.

1.2. Objectives

- To identify the vulnerable areas and failure modes related to a girder bridges under seismic events, by building a finite element model of the case study.
- To obtain the reliability index of the structure without damage, i.e., virgin reliability, employing reliability analysis methods.
- To estimate the resistance reduction of the bridge reliability based on the detected structural defects, using visual inspection techniques.

2. Methodology

Bridges are designed to withstand seismic action without collapsing, thus, retaining its structural integrity. Bridges that are at the end of their design life have a worse performance under seismic events due to structural deterioration over time. Visual inspection data can be used to identify structural damage that could impact the bridge performance; therefore, it is important to assess the performance of structures under seismic events. For this study, the following approach was adopted.

A qualitative research was chosen because it best served to answer the question and the purpose of the study. This design includes but is not limited to the use of observational methods, secondary data and a case study; this approach involves an inductive data analysis, assessing the bridge performance from the data gathered from its critical component. (Creswell, 2013).

The aforementioned approach would allow assessing the bridge performance under seismic events from a case study (viaduct “La Flora”, which is a multi-span prestressed girder bridge in Bucaramanga, Colombia), applying observation methods of collecting data as visual inspection and secondary data as drawings.

For this study, the methodology proposed in the quality specifications for roadway bridges, standardization at a European level (COST TU1406) was adapted to obtain a better fit to the case study (Bielecka et al., 2018). The following approach was adopted (see Figure 1).

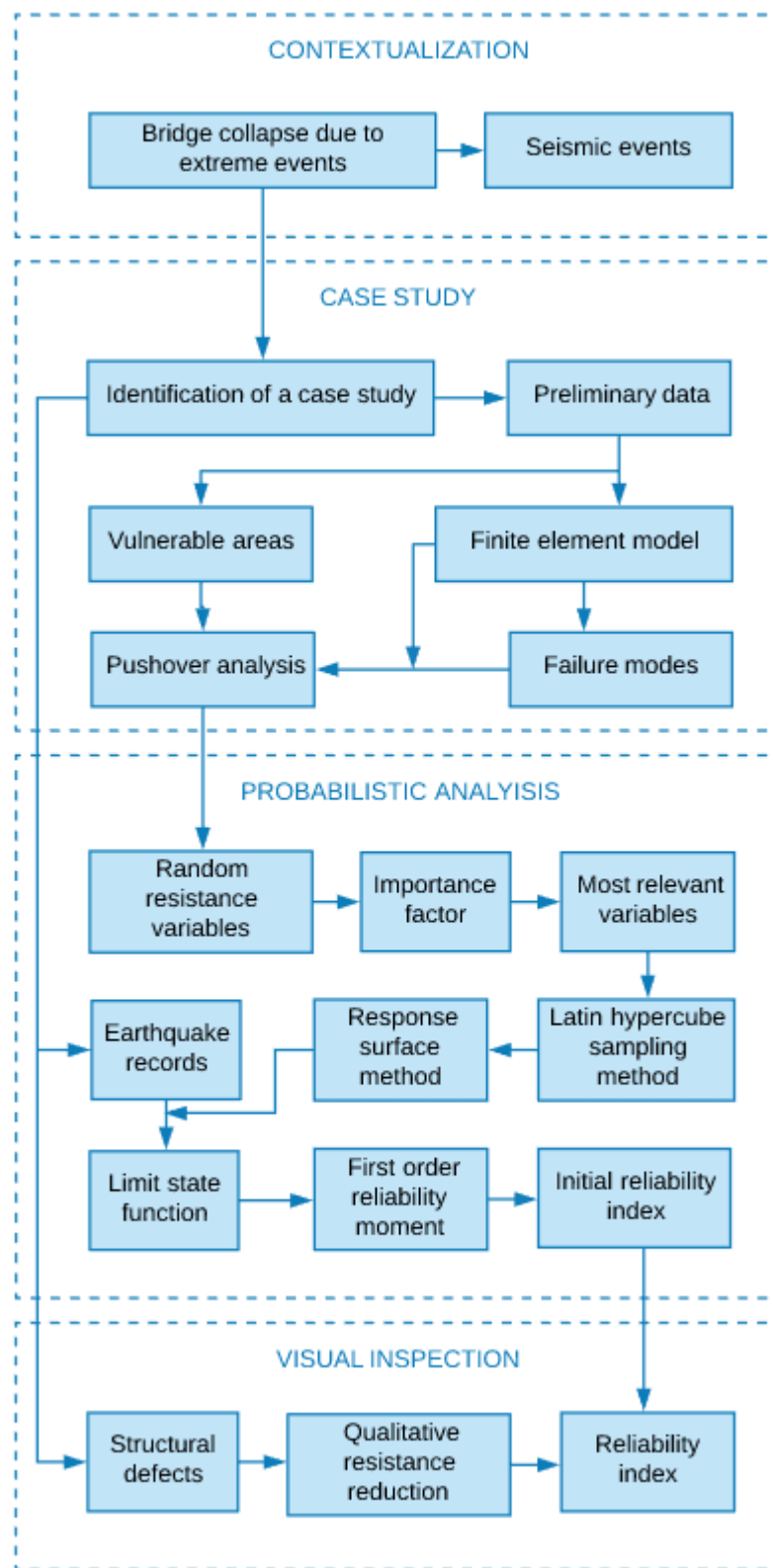


Figure 1. Methodology flow chart. Adapted from Bielecka et al., (2018).

The steps to take are the following. First, relevant literature, publications and studies will be reviewed to get detailed information on the vulnerable zones and failure modes of girder bridges in general and girder bridges under seismic events in particular.

Second, a numerical model will be built for the case study. The numerical modeling is undertaken using SAP2000, a software used for the simulation of the modal response of a structure. Data regarding failure modes and vulnerable zones will be obtained from the numerical model.

Third, the reliability index of the structure without damage will be obtained using probabilistic analysis, e.g., First Order Reliability Moment (FORM), which are reliability methods based on the ultimate limit state function. (Nowak & R. COLINS, 2000)

Next, qualitative data will be gathered from the viaduct “La Flora” employing visual inspection techniques, in this case, the methodology from the COST Action TU1406 will be used. This would ensure the validity of the data obtained from the previous literature review and the numerical model.

Finally, resistance reduction will be applied to the reliability index in order to assess the bridge performance under seismic actions. The resistance reduction would be qualitatively assumed based on the observed defects during the visual inspection.

The study intends to satisfy the need to assess bridge performance under seismic events. This study will focus on girder bridges, which is the most common type of bridge that uses girders to support the deck, and will use qualitative data gathered from visual inspection. This research does not intend to be used to update the bridge performance but to assess it.

This research aims to contribute to the COST Action TU1406, in which José Matos holds the chair of the project. This work has some limitations; nevertheless, this could be a starting point for

developing a methodology for assessing the performance of bridge under seismic events. Further work needs to be done in other case studies to validate this research.

3. State of The Art Review

3.1. Bridge Failure

Failure refers to the incapacity of a structure or component to perform its design and construction requirements. Likewise, bridge failure could indicate distress, partial or total collapse (failure of all substantial parts of a bridge).

Bridge failure has been caused by several factors and its combinations, some of those factors could be overloads, flood, material inadequacy, scour, collisions, lack of maintenance or earthquakes, and can be classified in three main categories: construction failure, in-service failure or extreme events. Loads due to extreme environmental events, such as earthquakes, scour and floods, are difficult to predict and quantify in the design of a structure (Schultz & Gastineau, 2016).

Different types of bridges are more vulnerable and more susceptible to specific causes. In Table 1, the most common causes for bridge collapse in function of the type of structure have been summarized (Deng, Wang, & Yu, 2015).

Table 1.

Most common causes for bridge collapse.

Type of bridge	Most vulnerable causes
Beam bridge	Flood, scour, earthquake, collision, overloading
Masonry bridge	Flood, scour, earthquake, overloading
Steel arch bridge	Overloading, wind
Steel truss bridge	Overloading, fatigue
Flexible long-span bridge	Wind

Note: Adapted from Deng et al., (2015).

3.1.1. Girder bridges. Beam or girder bridges are the most common bridge type. The most common failure modes for this type of structure includes deck misalignment, weak connections, inadequate support length of bridge decks, and pier failure due to shear, crushing, and erosion (Deng et al., 2015). Most frequent occurrences are associated to girder bridges (made of steel, concrete or both) (Wardhana & Hadipriono, 2003).

3.1.2. Extreme events. External events represent the most common causes of bridge failure. Furthermore, failure mainly occurs during the service life of the structure (Wardhana & Hadipriono, 2003).

Natural disasters such as floods, scours, earthquakes and hurricanes, are often unavoidable and can cause bridge failure. Scour and floods are major causes for bridge collapse. However, earthquakes lead to vertical and horizontal ground motion that can result in failure in bridge substructures. The horizontal ground motion directly affects the piers triggering shear failure. Furthermore, ground motion can lead to the liquefaction of the soil at the bridge foundation (Deng et al., 2015).

3.1.3. Bridge failure due to seismic events. Seismic events could cause bridge collapse, due to vertical and horizontal ground motion that lead to component damage such as shear-flexural failure of the piers, expansion joint failure or girder sliding (Choudhury, 2015).

Colombian engineering is lagging behind other countries in terms of rigorous quality control in the construction stage and bridge management and bridge maintenance. Even doe earthquakes don't represent one of the main causes of structural collapse, the inadequate inspection and maintenance could lead to bridge failure (Diaz, Moreno, Mohammadi, & Asce, 2009).

3.2. Probabilistic Analysis

Probabilistic analysis for assessment of existing structures are similar to the ones used in the design of a new structure. Therefore, providing new methods and information that were not available when the structure was designed. In the design of a structure there are some uncertainties regarding geometry, material properties, loadings and environmental conditions that reflects the imperfect knowledge about the structure (Faber, 2000).

3.2.1. Latin hypercube sampling. In the Latin hypercube design, only one point is located on each level of every random variable (see Figure 2). Thus, there are the same number of levels as analysis points and values of each point are selected randomly from each level. In Figure 2, the LHS method is used to define 5 samples (analysis points) that fully represents the two random variables (X_1 and X_2) by locating only one point on each of the variables levels (Choi, Grandhi, & Canfield, 2007).

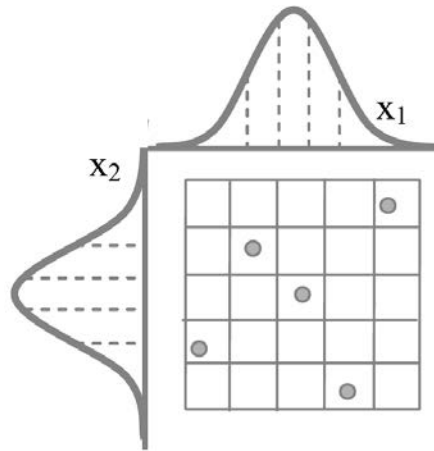


Figure 2. Latin Hypercube Sampling Method. Adapted from Choi et al., (2007).

3.3. Reliability Analysis

“Reliability is the probability that a system will perform its function over a specified period of time and under specified service conditions” (Choi et al., 2007, p.4). Structural reliability is concerned with the calculation and prediction of the probability of structural failure, in order to prevent tragic loss of life and property, and the goal is to choose design alternative that improve structural reliability and mitigate the risk of failure (Choi et al., 2007).

3.3.1. Limit state function. Structural failure happens when a structure cannot perform its function as intended. Limit state functions represents the boundary between the desired and the undesired performance of a structure under a certain failure mode (cracking, corrosion or buckling, among others)(Nowak & Collins, 2000).

Limit state functions can be classified in three types which are ultimate, serviceability and fatigue limit states functions. Ultimate limit states are related to loss of load carrying capacity, for

example, the formation of plastic hinges. Serviceability limit states are used for gradual degradation, maintenance costs or user's comfort, such as, permanent deformations or cracking. Finally, fatigue limit states are associated with loss of strength under repeated loads (Nowak & Collins, 2000).

3.3.2. Reliability index. The reliability index can be defined as the inverse of the coefficient of variation. According to the Hasofer and Lind definition the reliability index can be defined as the shortest distance from the origin to the line $g(Z_1, Z_2) = 0$ (see Figure 3). In equation 1, β is the reliability index, μ and σ are the mean and standard deviation of the random variable (Z_1 and Z_2), Φ is the normal distribution and P_f is the probability of failure of the structure.

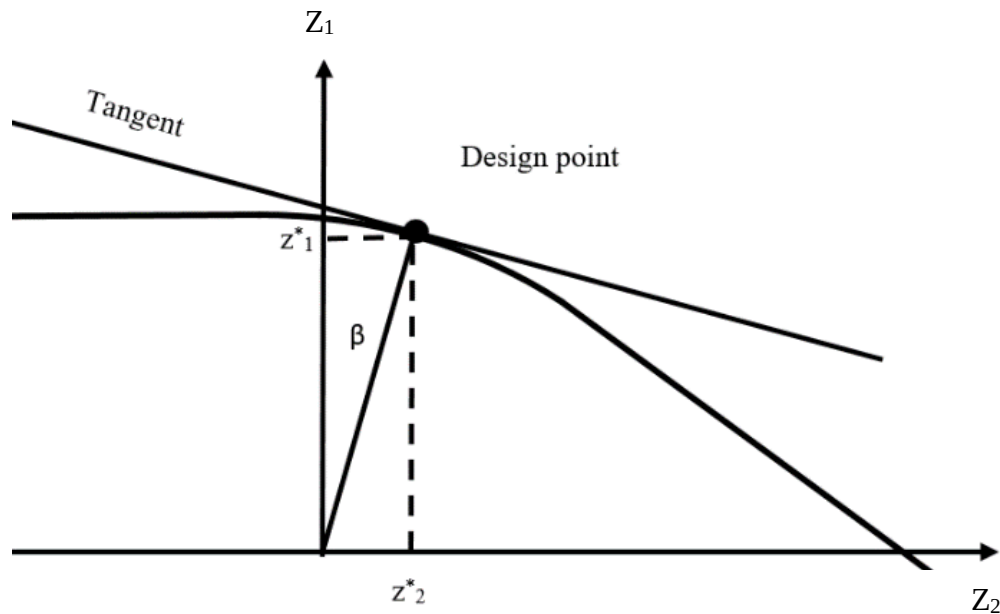


Figure 3. Hasofer-Lind reliability index definition. Adapted from Nowak & Collins, (2000).

$$\boldsymbol{\beta} = \frac{\mu_1 - \mu_2}{\sqrt{\sigma_1^2 + \sigma_2^2}} = -\Phi^{-1}(P_f) \quad (1)$$

3.3.3. Response surface method. Response Surface models are simple mathematical models that are designed to describe experimental outcomes (e.g., structural response) in terms of random variables (e.g., system conditions). Response surface model can be classified regression models (polynomials and non-linear functions) and interpolation models (polyhedral and radial basis functions) (Bucher, 2009).

First and second order polynomials are the most common used regression models, as response surface models, that allow to describe the relation between the structural response and the basic variables (Bucher, 2009). In equation 2, the general formulation for the first order polynomial can be found. Correspondingly, equation 3, presents the general fomilation for the second order polynomial.

$$\eta = p_0 + \sum_{i=1}^n \mathbf{p}_i \mathbf{x}_i \quad (2)$$

$$\eta = p_0 + \sum_{i=1}^n \mathbf{p}_i \mathbf{x}_i + \sum_{i=1}^n \sum_{j=1}^n \mathbf{p}_{ij} \mathbf{x}_i \mathbf{x}_j \quad (3)$$

3.3.4. First order reliability method (FORM). FORM is based on the Hasofer-Lind definition of the reliability index in standard Gaussian space. in order to perform FORM, it is necessary that the variables are uncorrelated Gaussian variables (zero mean and unit variance) (Bucher, 2009).

The design point is the point of failure in which the minimum distance β to the origin point within the failure domain. Equation 4, shows the general formulation for the ultimate limit state function $g(\cdot)$ based in the Hasofer-Lind reliability definition. Subsequently, equation 5 presents the formulation for the reliability index in Gaussian space (Nowak & Collins, 2000).

$$G(x_1, x_2, \dots, x_n) \approx G(x_1^*, x_2^*, \dots, x_n^*) + \sum_{i=1}^n (x_i - \mu_{x_i}) * \frac{\partial G}{\partial x_i} \quad (4)$$

$$\beta = \frac{G(\mu_1, \mu_2, \dots, \mu_n)}{\sqrt{\sum_{i=1}^n \left(\frac{\partial G}{\partial x_i} * \sigma_{x_i} \right)^2}} \quad (5)$$

4. Case Study “La Flora” Bridge

The viaduct “La Flora” is a nine-span girder bridge located in Bucaramanga, Colombia (see Figure 4). The bridge has 298.3m of total free length between abutments (see Figure 5), a deck width of 25m, with 3 lanes for each roadway. The deck is supported by 91 girders with a height of 1.63m (see Figure 6), that are distributed on the nine spans of the bridge. The substructure is composed on each axis by two 4x3m piers, that are connected to a cap beam with 2m of height and 3m width (see Figure 7).



Figure 4. Case study: “La Flora” bridge.

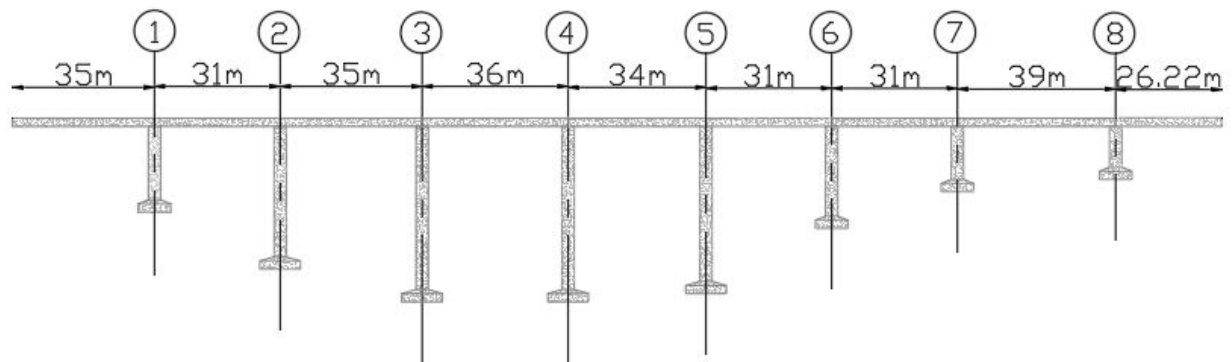


Figure 5. Bridge longitudinal axis.

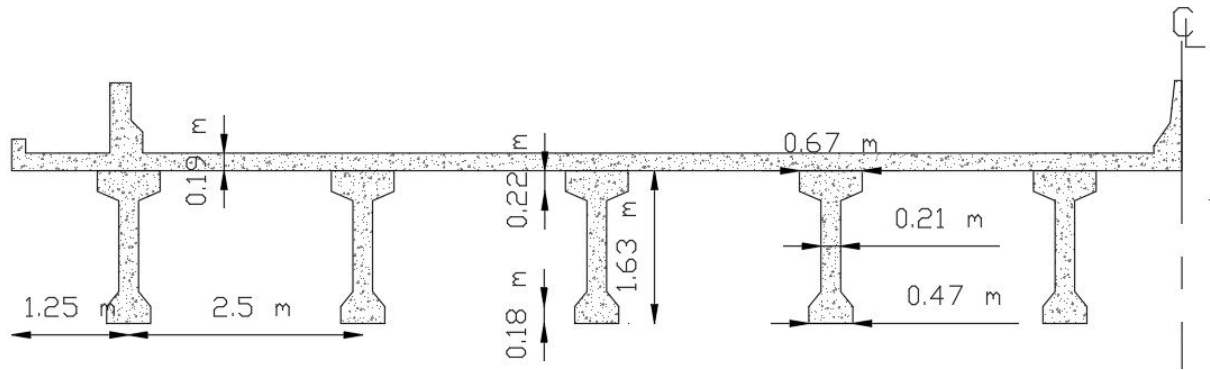


Figure 6. Deck cross section.

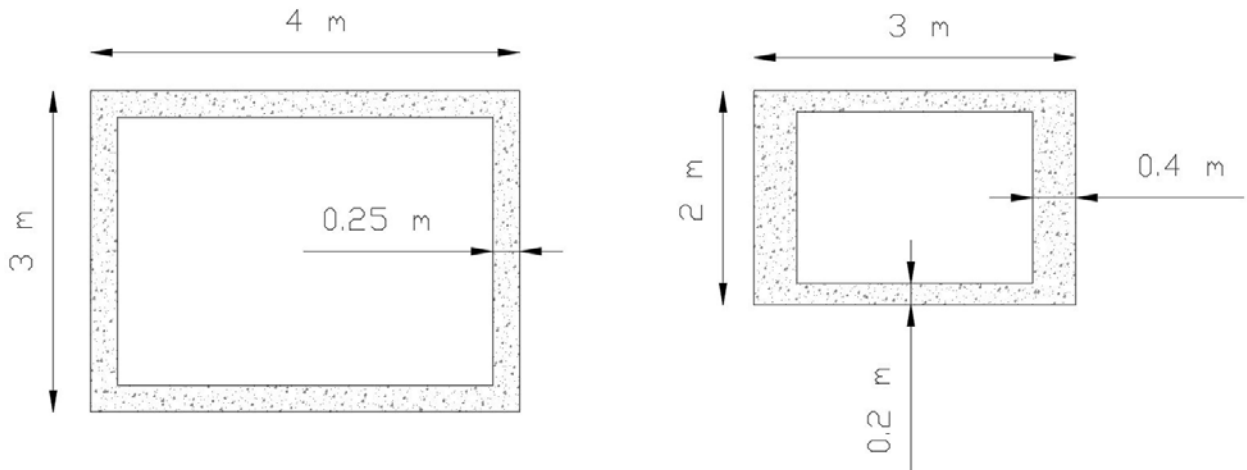


Figure 7. (a) Pier cross section (b) Cap beam cross section.

The cross-section of the superstructure, on each span, is formed by 10 pre-cast I-beams (see Figure 6) with 35MPa concrete, and a 28MPa concrete slab that has a thickness of 0,19 m and width of 25m. Girders were made using post-tensioned concrete in two stages, the first one after the concrete of the girder has been cast, then part of the tendons were stressed. The second stage involves the tensioning of the tendons that remained, this stage takes place after the remaining permanent loads were applied.

The span between axes 7-8 has a length of 39m, and its cross section is formed by the same elements, dimensions and materials that the other spans, with the exception that the number of girders increases to 11.

4.1.Numerical Model

First preliminary data is gathered, then the numerical model is build using the software SAP2000 (Computers and Structures, 2019) whose license was provided by the research group in materials and structures (INME by its initials in Spanish). This software is best suited for the analysis due to its seismic analysis options and its low computational time-consuming models. The case study is represented by a 3D model with 6 degrees of freedom. The types of objects, that are used to represent each of the bridge components, are frame objects for girders, piers and diaphragms, tendons objects for the pre-stressing steel, shell objects for the deck and joint objects that are automatically generated at the corners or end of all of the previous types of objects. All the objects have the same geometry that the physical member that represents (Sap, 2016).

In order to reduce computational work of the model, the material type that is defined for concrete and steel, have a linear and isotropic behavior. The parameters which are used to describe the behavior of each material, are obtained according to the Colombian code for bridge design (CCP-14 by its initials in Spanish) (Ministerio de Transporte, INVIAS, 2014c). In order to represent the nonlinearity of the material, plastic hinges are defined in the critical elements (Sap, 2016).

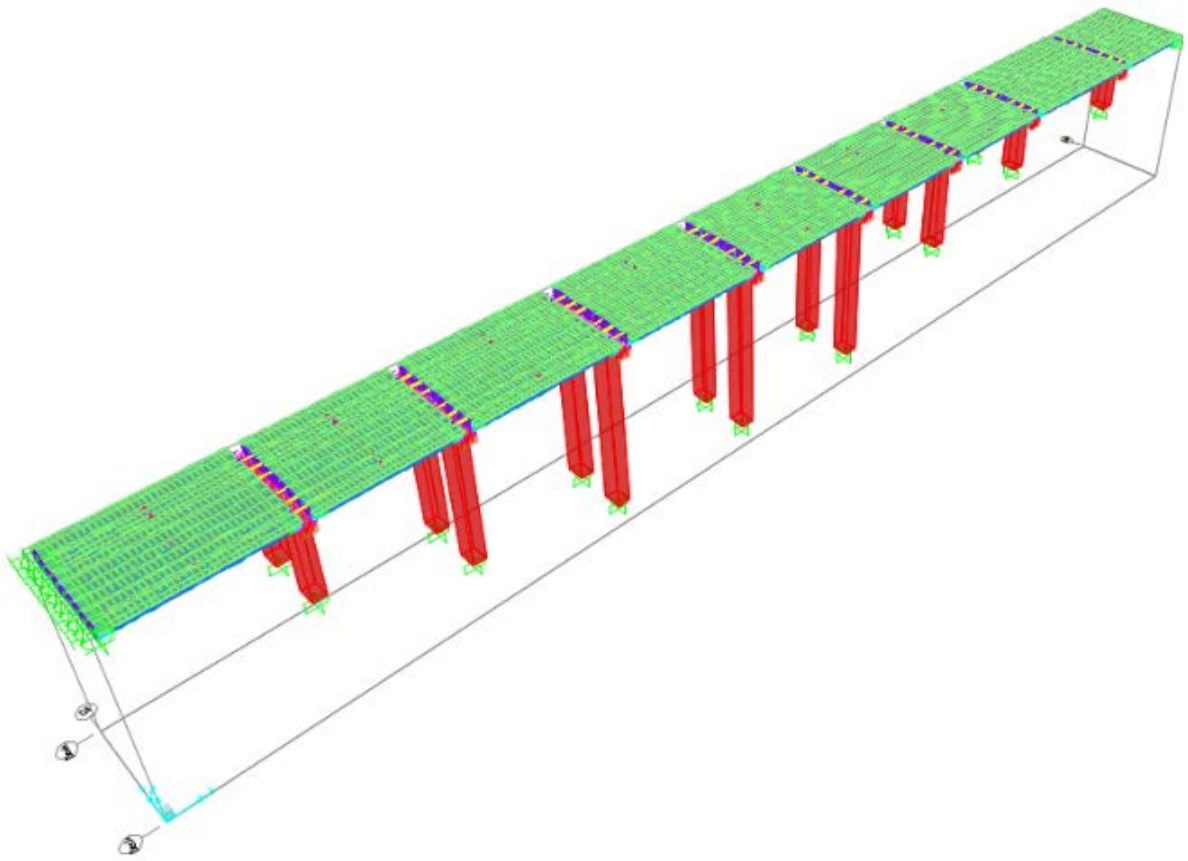


Figure 8. Bridge numerical model.

The load distribution is modelled according to the loads section in the CCP-14. The different loads considered are dead loads (DC) that includes the self-weight of all the bridge elements (including structural and non-structural elements), self-weight of the pavement (DW) and live load (LL) that consist of a lane load and a design vehicle (Ministerio de Transporte, INVIAS, 2014a).

Once the numerical model is set to the real conditions of the bridge (see Figure 8), vulnerable areas and failure modes regarding seismic events are found.

4.2. Vulnerable Areas

Significant seismic events always have been a medium to understand the deficiencies in our design codes. Bridges have inherited some vulnerabilities due to its design code. In California, codes remained nearly unchanged for the period between 1971 to 1994. After the 1971 San Fernando earthquake helped recognize the importance in ductility in the bridge response. In 1980 the design was oriented to the development of plastic shear of the piers. Bridges that were designed during that period of time could present the following vulnerabilities (Caltrans, 2006):

- Column shear failure of plastic hinge regions
- Shear failure of flared columns
- Unseating of expansion joint hinges

The case study (“La Flora” bridge) was built in 1995 the same year that the first Colombian code for bridge design (CCP-95) was issued. The first seismic resistance normative in Colombia was published in 1984 and was based on the translation of the ATC-3 released in 1977 (García, 2015). The following normative, that would include the post-1994 design criteria stated in the ATC-32, was issued in 1998 and was known as NSR-98 (by its initials in Spanish) (Caltrans, 2006; García, 2015).

4.2.1. Column behavior. The bridges that present irregularities in its substructure are more vulnerable to seismic events than regular bridges, this is due to the variation of the pier lengths. Bridges, in which its central pier is higher than the extreme ones, are more vulnerable to seismic events (Julieth, Megchún, & Gómez Soberón, 2011; Soberón & Soberón, 2017)

The effects of unequal column height in bridges could affect the global seismic performance of the bridge. In irregular bridges, the shortest column is the one that determines the seismic performance, which means that the shortest pier would carry a significant percentage of the base shear, which could lead to shear failure instead of flexural failure (Thomas, 2018).



Figure 9. Diagonal crack (plastic hinge region-base). Adapted from Caltrans, (2006).

Piers designed between 1971 and 1994, develop the yield moment of the section but degrade after repeated cycles due to degradation of concrete shear strength within the plastic hinge, leading to the shear failure of the hinge (see Figure 9) (Caltrans, 2006).

Figure 10 shows the critical zones of the shortest pier, in which the yellow ovals represent the place where is most likely to present bending failure, the red rectangles represents the places where

probably the pier is going to present shear failure, and the purple rhombus represent buckling (Hajdin et al., 2018). Due to the small height of the pier in comparison with its cross section, buckling is the less likely. The bottom of the pier is going to be the most vulnerable zone of the bridge due to the high concentration of the base shear.



Figure 10. Critical zone of the case study

4.3.Failure Modes

The case study presented in this article is an irregular bridge designed between 1971 and 1994, which indicates that have a high vulnerability to seismic events, more specifically the bridge could

present a shear failure of the hinge in the shortest pier. This part will focus on determining the failure mode of the bridge.

First, the modal analysis would be carried out in order to determine the fundamental modes, for this part 102 modes are found in order to consider at least 90% or better mass participation (Ministerio de Transporte, INVIAS, 2014b).

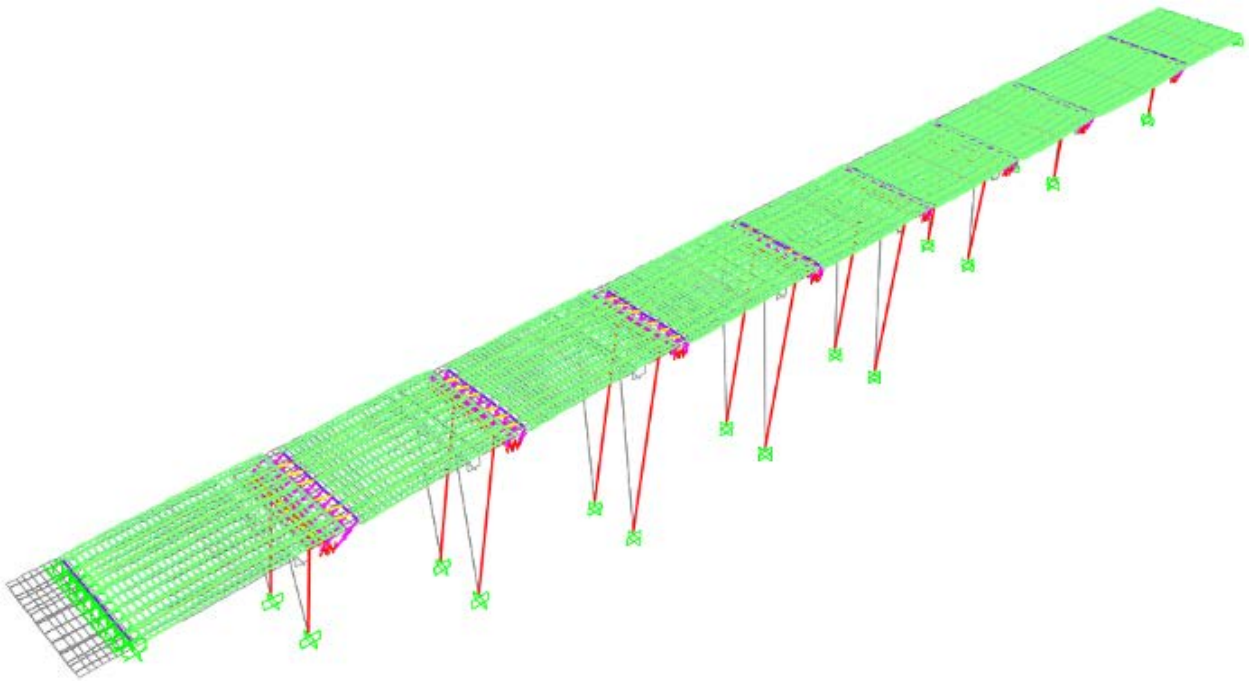


Figure 11. Longitudinal mode $T = 1.174$ s.

The first fundamental mode of the bridge is longitudinal (see Figure 11), meanwhile the second is transversal (see Figure 12) where the central pier presents the maximum displacement. As discussed before the bridge performance under seismic events is restricted by the shortest pier, which in this case is located at the end of the bridge. Further analysis will be carried out using the

longitudinal axis of the bridge due to the higher period ($T = 1.17$ s). Furthermore, the substructure is composed of a rigid frame, in which the longitudinal axis is the most vulnerable due to lesser stiffness of the frame.

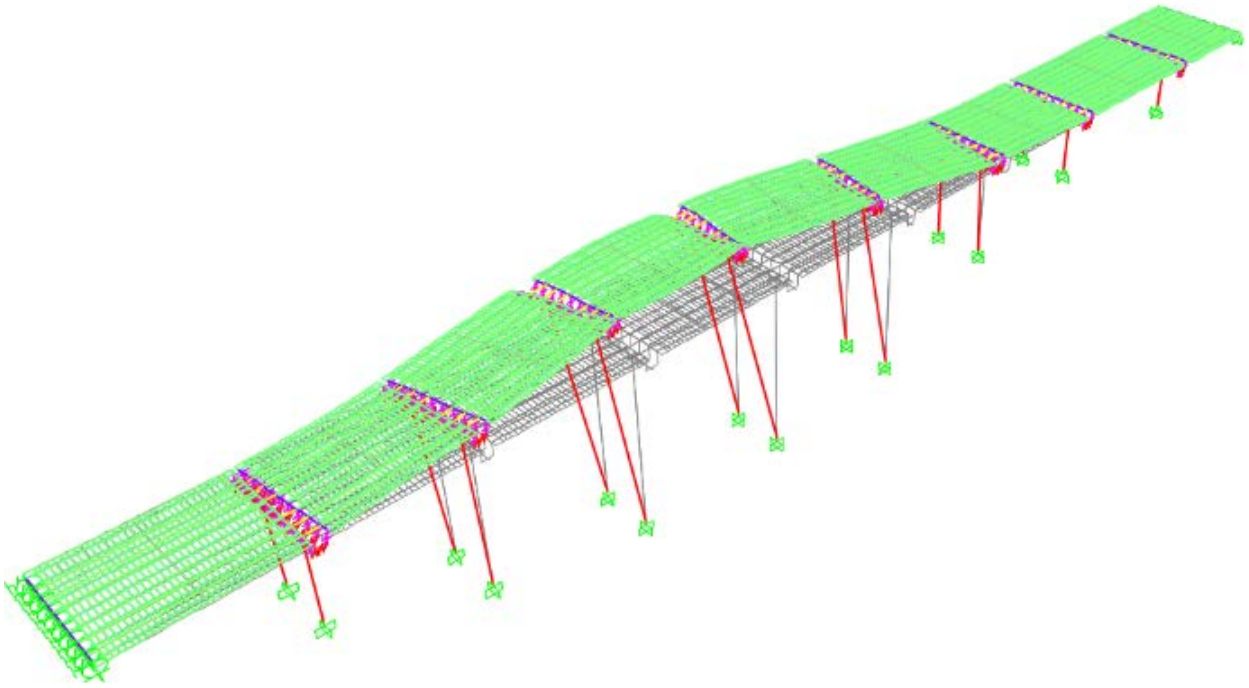


Figure 12. Transversal mode $T = 1.001$ s.

Second, pushover analysis will be performed in order to obtain the maximum base shear that the structure is able to withstand. In order to perform the pushover analysis, hinges will be defined at the top and bottom of each pier of the structure. Hinges allow simulating post-yielding behavior to a frame object (Sap, 2016).

In this case, the isotropic P-M2-M3 hinge is used. The moment-rotation diagram is used to model the physically non-linear behavior of a pier element. Hinge properties are generated

according to the Federal Emergency Management Agency (FEMA) recommendations (Cunha, 2012; Sap, 2016). Plasticity may be associated with force-displacement behavior (axial-shear) (see Figure 13) or moment-rotation (torsion and bending).

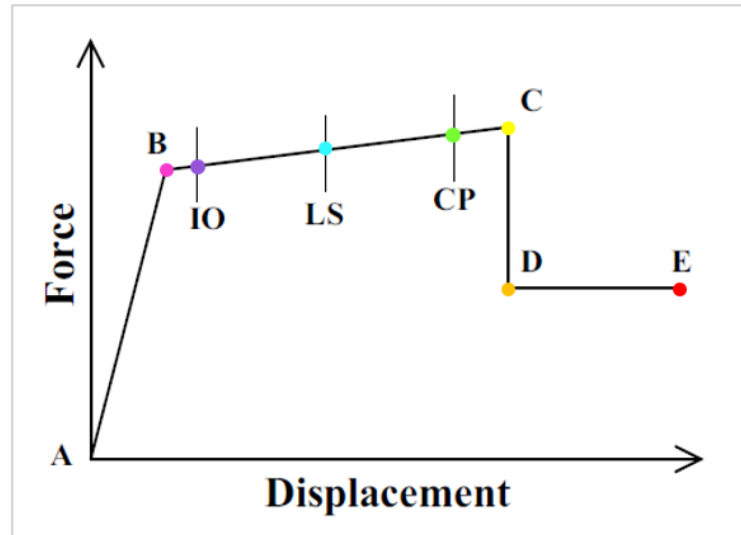


Figure 13. A-B-C-D-E curve. Adapted from Sap, (2016).

In Figure 14, one of the piers of each frame is shown, at the bottom of each pier a dot with a characteristic color is presented, each color represents a point in the A-B-C-D-E curve (see Figure 13). The shortest pier (the last pier to the right) has reached the point E (failure) while the other piers stay in the B-C range.

Taken the plastic hinge, at base of the shortest pier, as the critical section of the bridge, the pushover curve for flexural and flexural/shear is determined. In Figure 15, two pushover curves are shown, which demonstrate that the bridge failure is not purely flexural. The shear failure of the plastic hinge was expected due to the design code used (Caltrans, 2006).

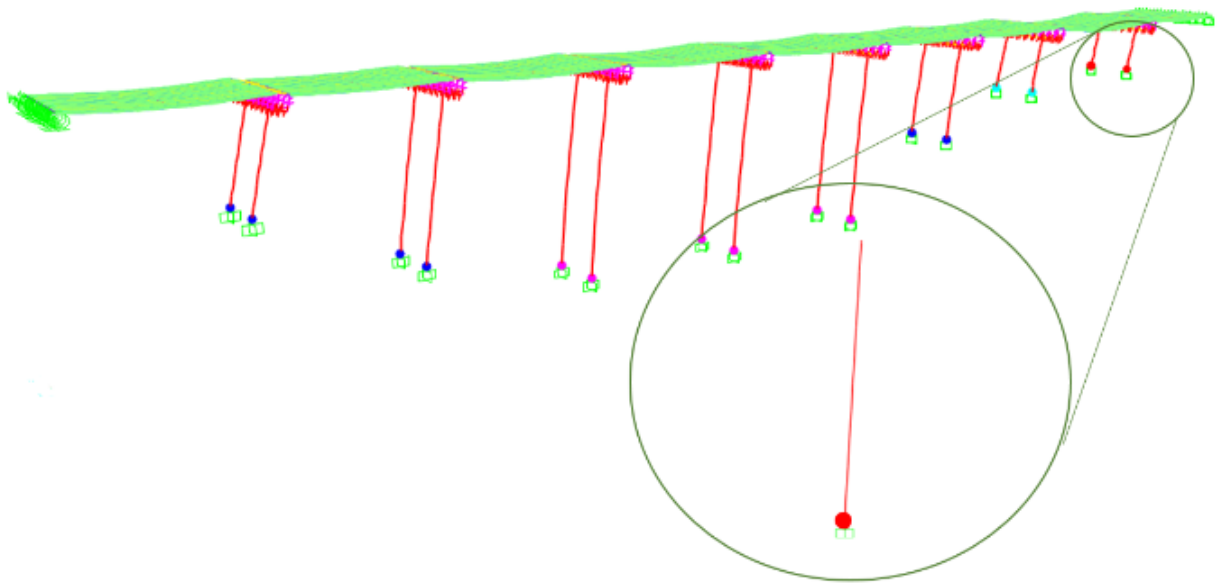


Figure 14. Plastic hinge formation at the base of the piers.

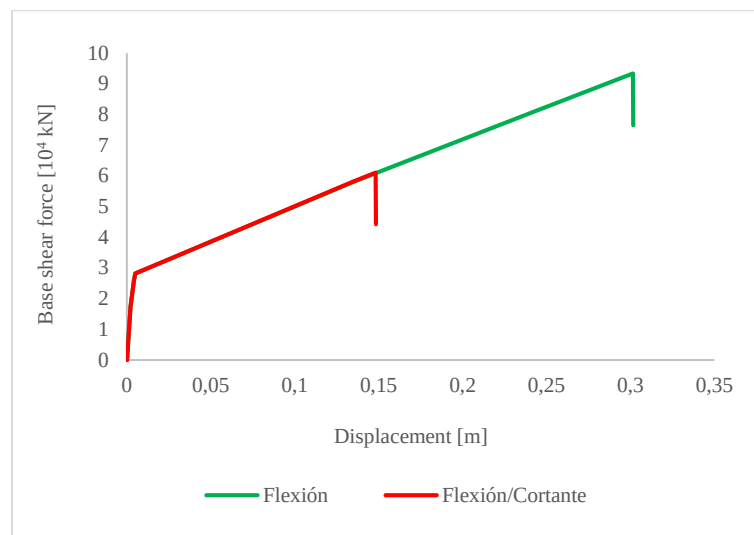


Figure 15. Pushover curve.

The pushover curve for the flexural/shear behavior is used as the reference point for the probabilistic analysis. The bridge failure occurs for an ultimate base shear of 61.069 kN as shown in Figure 15.

5. Probabilistic Analysis

In structural design, there are many uncertainties related to loading and loading-carrying capacities. Structures don't have a zero probability of failure due to the random variables involved in the design process. Thus, reliability of the structure needs to be fulfilled in order to achieve its design purpose for a specified lifetime (Nowak & Collins, 2000).

5.1. Random Variables

In order to obtain the reliability index of the case study, all the random variables involved in the load-carrying capacity of the reference model are identified (see Table 2). The random variables are related to material and geometry.

In Table 2, 15 random variables are presented, nevertheless, not all of those variables have a significant impact on the ultimate base shear capacity of the structure. Thus a sensibility analysis is performed using an importance factor b_k for each of the variables (Matos, Cruz, Valente, Neves, & Moreira, 2016).

Table 2.

Random variables used in the probabilistic analysis

ID	Description	Random Variables	Symbol	Nominal Values	Units	Bias	CV (%)	Reference
1	Concrete 21MPa	Compressive strength	f'_c	21	MPa	1.31	17	(Bischoff & Gross, 2016)
2	Concrete 28MPa		f'_c	28	MPa	1.24	15	
3	Concrete 35MPa		f'_c	35	MPa	1.19	13.5	
4	Concrete 21MPa	Elasticity Modulus	E_{cs}	55	GPa	1	8	(Wiśniewski, Cruz, Henriques, & Simões, 2012)
5	Concrete 28MPa		E_{cs}	60	GPa	1	8	
6	Concrete 35MPa		E_{cs}	64	GPa	1	8	
7	Concrete	self-weight	γ_c	24	kN/m ³	1.05	4	(Wisniewski, 2007)
8	Reinforcing steel	Yielding strength	f_y	420	MPa	1.13	3	(Bischoff & Gross, 2016)
9	Reinforcing steel	Bar area	A		mm ²	1	2	(JCSS, 2000)
10	Pre-stressing steel	Ultimate strength	F_{pu}	1870	MPa	1.06	3	(Wiśniewski et al., 2012)
11	Column dimensions	Breadth		3	m	1.005	4	(Bischoff & Gross, 2016)
12	Column dimensions	Width		4	m	1.005	4	
13	Beam dimensions	Width		0.67	m	1.01	4	(Wisniewski, 2007)
14	Slab	Thickness		0.19	m	1	4	
15	Column Concrete cover	Clear cover	C_c	5	mm		4	(JCSS, 2000)

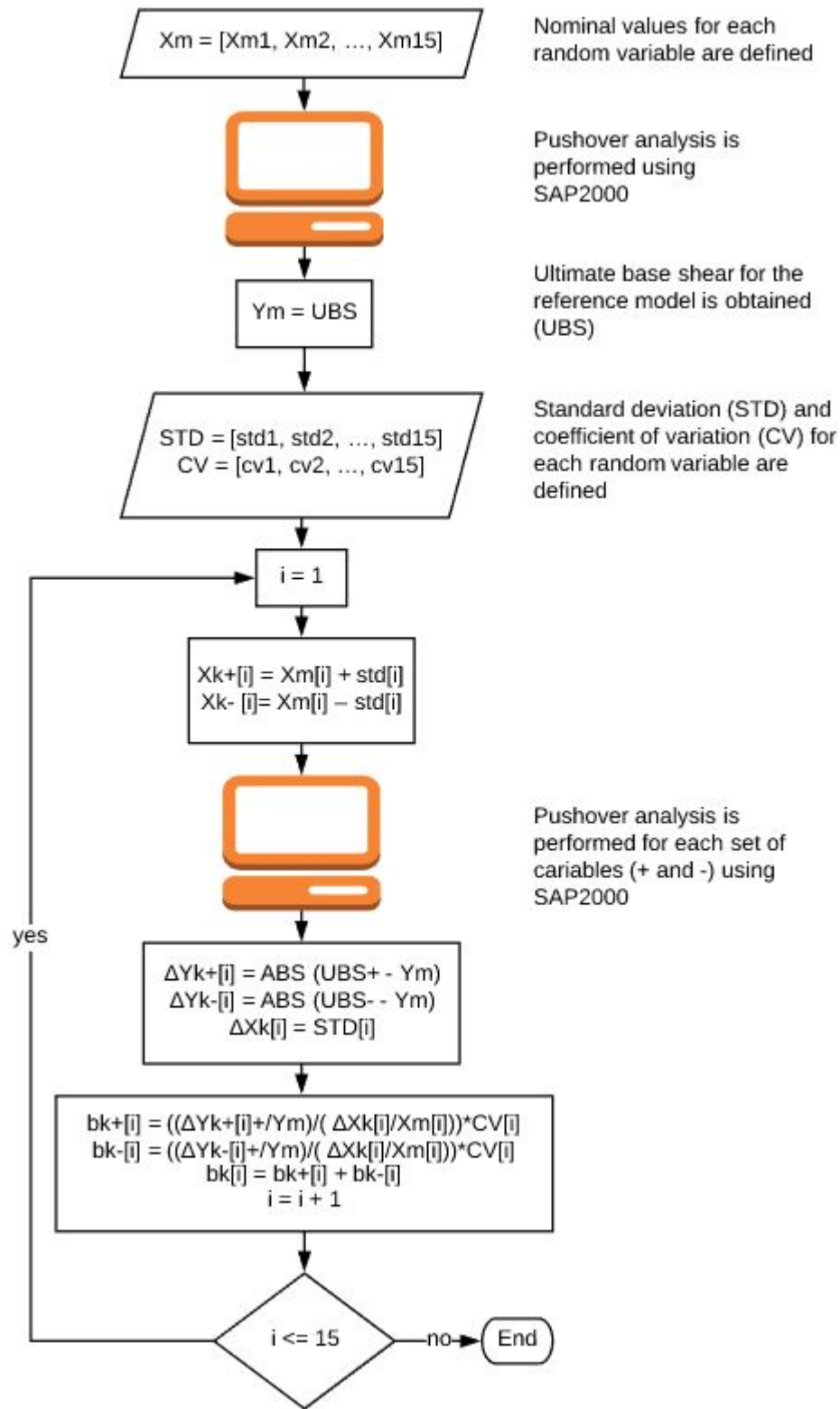


Figure 16. Importance factor flowchart. Adapted from Galvão, (2018).

$$b_k = \sum_{i=1}^n \frac{(\Delta y_k / y_m)}{(\Delta x_k / x_m)} * CV[\%] \quad (6)$$

Where Δy_k is the variation in the structural response due to a deviation Δx_k in the input from the parameter mean value x_m . y_k is the reference response, and CV is the coefficient of variation of the input parameter (Matos et al., 2016).

For each input parameter (i.e. random variables) the process in Figure 16 is followed. Figure 17 shows the importance factor for each parameter and the threshold value of 10% used to identify the most relevant variables.

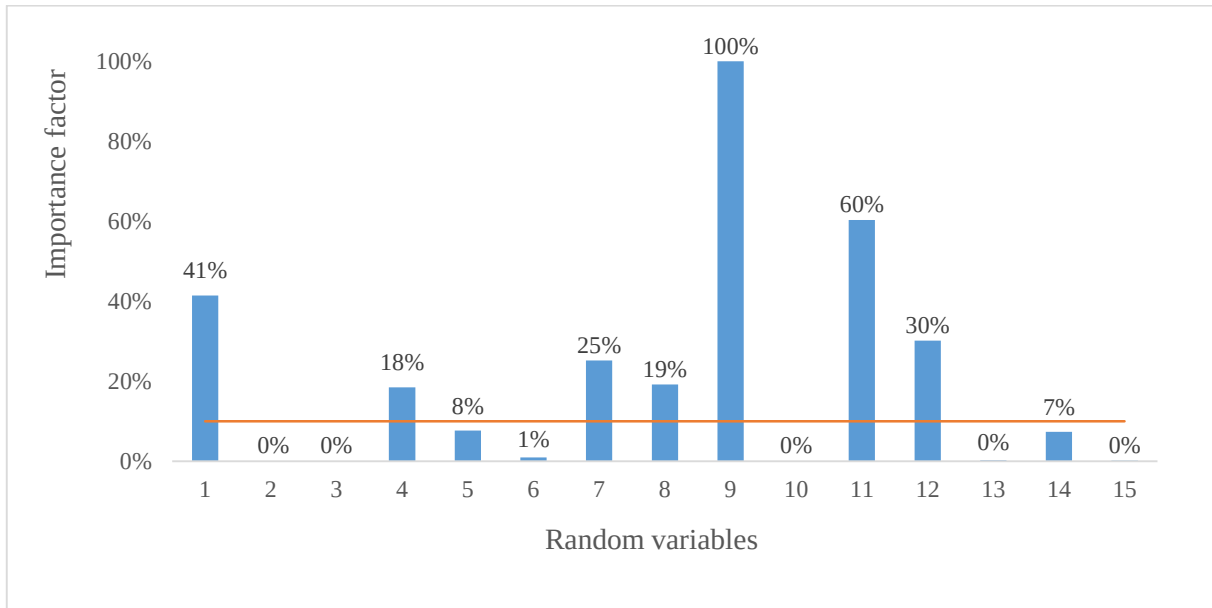


Figure 17. Importance factor b_k .

The variable with the most impact is the reinforcing steel bar area which is expected due to the importance of the reinforcement in preventing the flexural/shear failure of the plastic hinge at the bottom of the shortest pier. In the range of 60%-40%, the compressive strength of the concrete and dimension of the pier are found. The last three parameters that surpass the threshold of 10% are

elasticity modulus of the concrete used for the pier (21 MPa), the concrete self-weight, and the yielding strength of the steel reinforcement. The 8 parameters that miss the 10% bar are the compressive strength and elasticity modulus of the concrete used in the superstructure and dimensions of girder and slab, and the yielding strength of the pre-stressing steel of the girders.

5.2.Latin Hypercube Sampling Method (LHS)

In order to obtain the reliability index, it is necessary to generate a number of random samples. The LHS is used to reduce the number of simulations needed for reasonable results. In the LHS each random variable can be subdivided into n equals probability intervals (i.e. levels) and only one random value will be selected for each interval. In other words, there are the same number of levels as runs and the levels are assigned randomly to runs (Choi et al., 2007; Gunst, Myers, & Montgomery, 2006; Nowak & Collins, 2000)

In equation 2, the stopping criterion for the LHS is shown. This method is based on the Monte Carlo stopping rule that states that the standard error of the mean (S.E.M.) should be less than 1% of the mean (Schuyler, 1998). Where s is the standard deviation, n is the number of trials of the sample, and μ is the mean value.

$$S.E.M. = \frac{s}{\sqrt{n}} < 1\% \mu \quad (7)$$

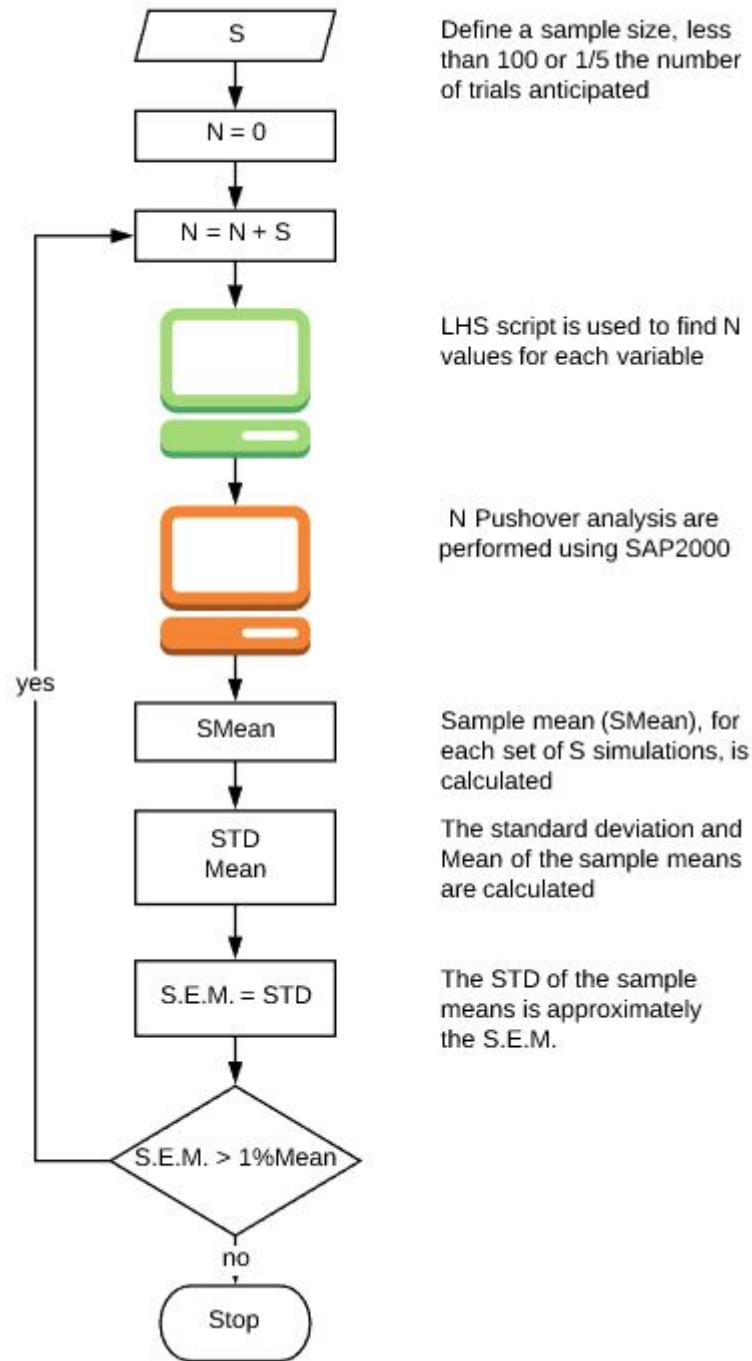


Figure 18. LHS method flowchart.

Following the methodology (see Figure 18), a sample size of 10 is used and 100 simulations are sufficient to fulfill the stopping criterion. Each combination of variables was generated using

a MATLAB (MathWorks, 2019) script and then the ultimate base shear for each of the files is obtained using SAP2000. Using the ultimate base shear of each simulation a probabilistic distribution of the structure resistance is found. The results of the ultimate base shear fit a normal distribution (see Figure 19).

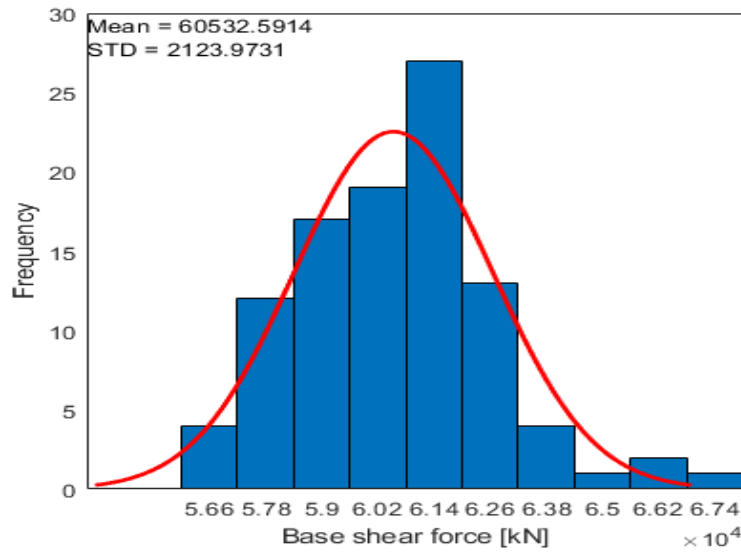


Figure 19. Load capacity curve.

5.3. Response Surface Method (RSM)

The RSM is used to represent the behavior of the structure in terms of contributions due to each individual random variable, and it is usually obtained by using regression techniques. Polynomial models of low-order provide a simple mathematical model with a small number of terms whose coefficients can be easily estimated (Bucher, 2009). Next the equations for the first order polynomial (FOP) and second order polynomial (SOP) are presented.

$$\eta = p_0 + \sum_{i=1}^n p_i x_i \quad (8)$$

$$\eta = p_0 + \sum_{i=1}^n p_i x_i + \sum_{i=1}^n \sum_{j=1}^n p_{ij} x_i x_j \quad (9)$$

Before obtaining the polynomials, the random variables need to be coded that will allow obtaining coefficient for each variable that is independent of the units of the variables. In order to obtain the coded variables, the following equation was used:

$$z = \frac{x - \mu}{\sigma} \quad (10)$$

Using a MATLAB script the first and second order polynomials are obtained. Figure 20 shows the plots of the coded sampled data (i.e. data obtained from the experiments) and the coded predicted data (i.e. data obtained by each of the polynomials). As presented the second order polynomial present a better fit than the first order polynomial, as shown by the correlation coefficient (r). Nonetheless, the difference is negligible and the first order provides a simpler equation with 8 coefficients (see equation (11)) against the 36 coefficients of the second order polynomial.

$$\eta = 60544.46 + 1360.38x_1 - 997.21x_4 + 654.17x_7 + 437.83x_8 + 120.68x_9 \quad (11)$$

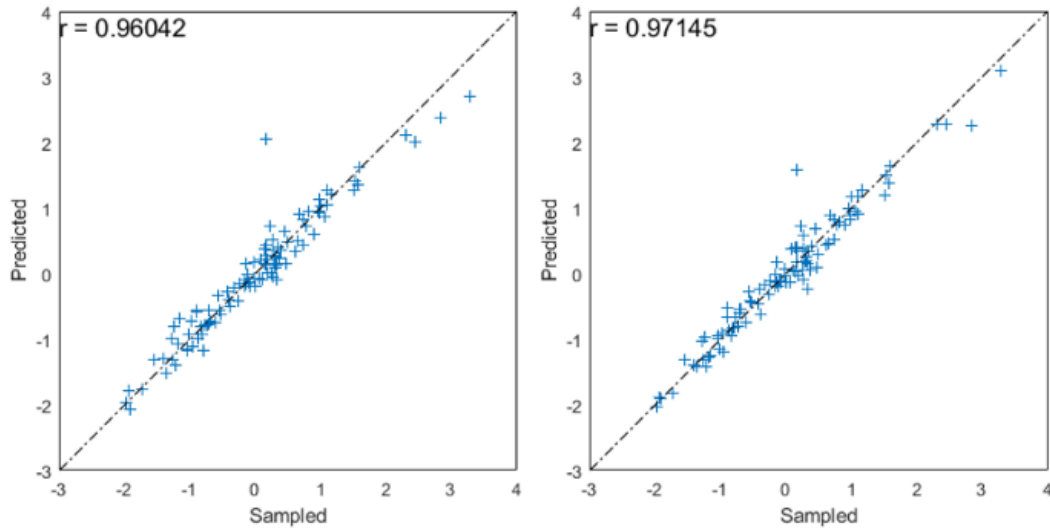


Figure 20. (a) FOP (b) SOP.

5.4. Seismic Loads

The resistance of the structure has already been discussed in this article in the form of the response surface of the ultimate base shear, but in order to obtain the probability of failure of the structure and thus the reliability index, the loading of the structure needs to be characterized.

Colombia is located in the Ring of Fire, a major area in the basin of the Pacific Ocean. Furthermore, Colombia has 3 tectonic plates which are the South American plate, the Caribbean plate and the Nazca plate. the case study is located in the city of Bucaramanga which has a seismic region better known as “the Bucaramanga Nest” which is the second highest active nest in the world (Bayona & Cruz, 2014).

First, the seismic data is recollected using the acceleration catalog of the Colombian geological service (SGC by its initials in Spanish) (Servicio Geológico Colombiano, n.d.). the seismic records are obtained from February of 1994 to April of 2019, and peak ground acceleration (PGA) from 567 seismic events in the north-south (NS), east-west (EW) and elevation (Z) directions are gathered.

Second, by using the square root of the sum of the squares (SRSS) method (see Equation (12)), the PGA of the NS and EW directions are combined. Subsequently, the data is adjusted to a probabilistic distribution. The records of the PGA fit an exponential distribution (see Figure 21)

$$\lambda = \sqrt{\sum_{i=1}^n \lambda_k^2} \quad (12)$$

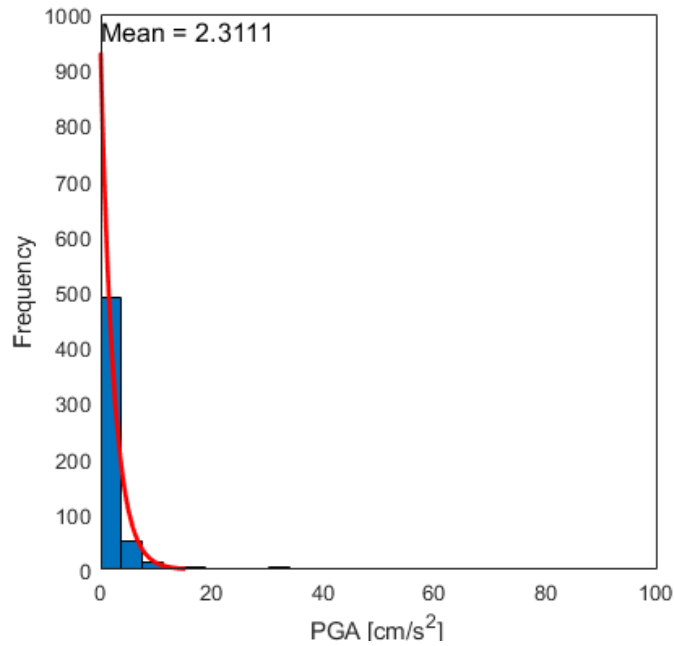


Figure 21. Exponential distribution for the PGA.

The resistance of the structure fits a normal distribution, in order to obtain the reliability index, the exponential distribution from the seismic data needs to be approximated to a normal distribution. Thus the Paloheimo transformation is used. The transformation consists of the approximation to a normal distribution that has the same mean value and pre-defined percentile close to the probability of failure (see Figure 22) (Matos & Henriques, 2015).

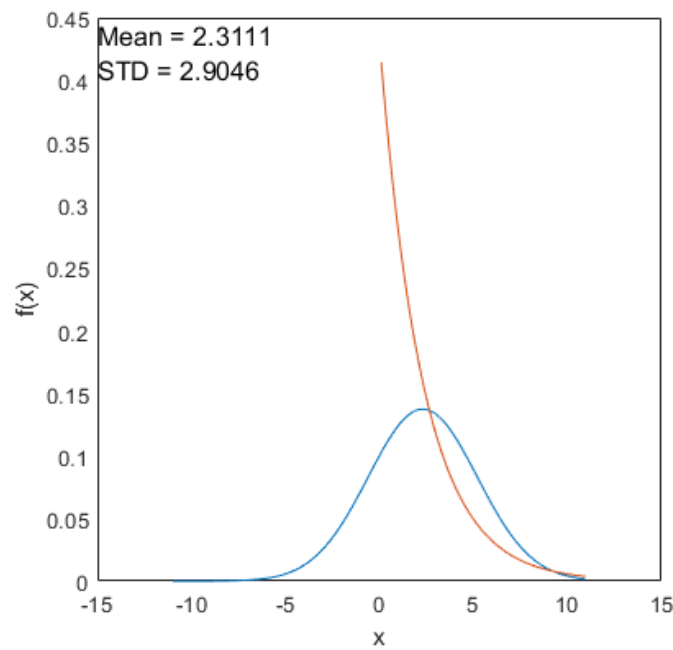


Figure 22. Paloheimo transformation of the PGA.

5.5.Limit State Function

Failure occurs when a part of a structure exceeds a specific limit and is unable to perform as required, that limit is called limit state function. The limit state function, $g(\cdot)$, can be defined as the margin of safety between the resistance, $R(\cdot)$, and the load of structures, $S(\cdot)$, (see equation (13)) (Choi et al., 2007).

$$g(X) = R(X) - S(X) \quad (13)$$

In order to consider the uncertainties of both resistance and load models, the variables θ_R and θ_S , are introduced. Which have mean values of 1.2 and 1.0, and coefficient of variation of 0.15 and 0.1, respectively. Both of the variables fit a lognormal distribution (Faber & Vrouwenvelder, 2000). Now the ultimate limit state function for the case study is presented.

$$g(X) = \theta_R * [\eta(x_1, x_4, x_7, x_8, x_9, x_{10}, x_{12})] - \theta_S * [pga * g] \quad (14)$$

Where θ_R and θ_S are the variables that represent the uncertainties in resistance and load models respectively, η is the FOP, found using the RSM, in function of the significant random variables. pga is the peak ground acceleration and g is the equivalent total gravity load, which is computed including the combination effects of the dead loads and live loads (Lu, Song, Yu, & Wang, 2008).

5.6. Reliability Index

The reliability index could be defined as the inverse of the coefficient of variation of the limit state function. Furthermore, the Hasofer-Lind method states that the reliability index is the shortest distance from the origin in the n-dimensionally space of reduced variables to the curve described by the limit state function (see Figure 3) (Nowak & Collins, 2000).

5.6.1. First order reliability method (FORM). FORM is an evolution of the Hasofer-Lind method. In the case of a nonlinear limit state function, linearizing the nonlinear function using Taylor series provide an approximate solution to the reliability index (see Equation 9) (Nowak & Collins, 2000).

$$G(x_1, x_2, \dots, x_n) \approx G(x^*_1, x^*_2, \dots, x^*_n) + \sum_1^n (x_i - \mu_{x_i}) * \frac{\partial G}{\partial x_i} \quad (15)$$

$$\beta = \frac{G(\mu_1, \mu_2, \dots, \mu_n)}{\sqrt{\sum_{i=1}^n \left(\frac{\partial G}{\partial x_i} * \sigma_{x_i} \right)}} \quad (16)$$

Where the partial derivatives are evaluated at mean values. The reliability index is obtained using a MATLAB script, which has a value of 5.0736. the value obtained correspond to a probability of failure of $1.0264E^{-6}$. This reliability index corresponds to the structure without any damage or degradation.

Also, it should be noted that this is only a relative value to the whole system, more analysis should be performed in order to obtain a proper scope of the bridge reliability. Moreover, this analysis is focused only into the effects of uncertainties in seismic loads, and analysis with uncertainties under other types of loads or limit states such as corrosion, traffic loads or human errors should be conducted.

6. Visual inspection

In order to update the reliability index, it is necessary to detect the possible defects, deterioration or even damage of the most relevant components of the bridge. Visual inspection is a useful tool for a first assessment of the structure. Each component must be rated from 0 to 5 according to Table 3, the final evaluation of the whole bridge needs to be performed by a really experienced bridge engineer, because there is no correlation between component rating and the structure as a whole, due to lack of influence of importance of a certain component on the structural stability (Stipanovic et al., 2017).

Table 3.

General rating condition

Grade	Condition
0	No damage
1	Very good
2	Good
3	Satisfactory
4	Faulty
5	Bad

Note: Adapted from Stipanovic et al., (2017).

The methodology used for the visual inspection was based on the methodology proposed in the quality specifications for roadway bridges, standardization at a European level (COST TU1406) and the national institute of roads (INVIAS by its initials in Spanish) (INVIAS & UNAL, 2006;

Stipanovic et al., 2017). The visual inspection is carried on the case study critical zones (i.e. the shortest pier) (see Figure 10), the following defects are found.

6.1.Cracks

Some smaller cracks were detected on the pier; these defects have a thickness of less than 0.5 mm. Are most likely a result from the construction process (shrinkage cracks) and doesn't represent any reduction in load-bearing capacity, serviceability or durability (Stipanovic et al., 2017).



Figure 23. Small crack detected on the pier.

6.2.Environmental effects

Damage at the base of the pier was detected. Nonetheless, the defect is most likely due to an external factor like an impact which allows the degradation of the concrete and the corrosion of the transverse reinforcing steel. Minor or light damage such as the one shown in Figure 24, have no impact on the load bearing capacities of the structure (Stipanovic et al., 2017).



Figure 24. Environmental defects detected on the pier.

7. Performance assessment

As shown in the visual inspection the defects that were found in the structure were minor and have no effect on the load bearing capacity. The shortest pier presents a very good condition and a grade value of 1. A resistance reduction factor must be qualitatively assumed, based on the structural defects the structure doesn't present any reduction on its reliability.

The resistance reduction factor could change in further inspections, monitoring or test performed on the case study, for that reason Figure 25 shows the reliability curve and the probability of failure for different resistance reduction factors (Hajdin et al., 2018).

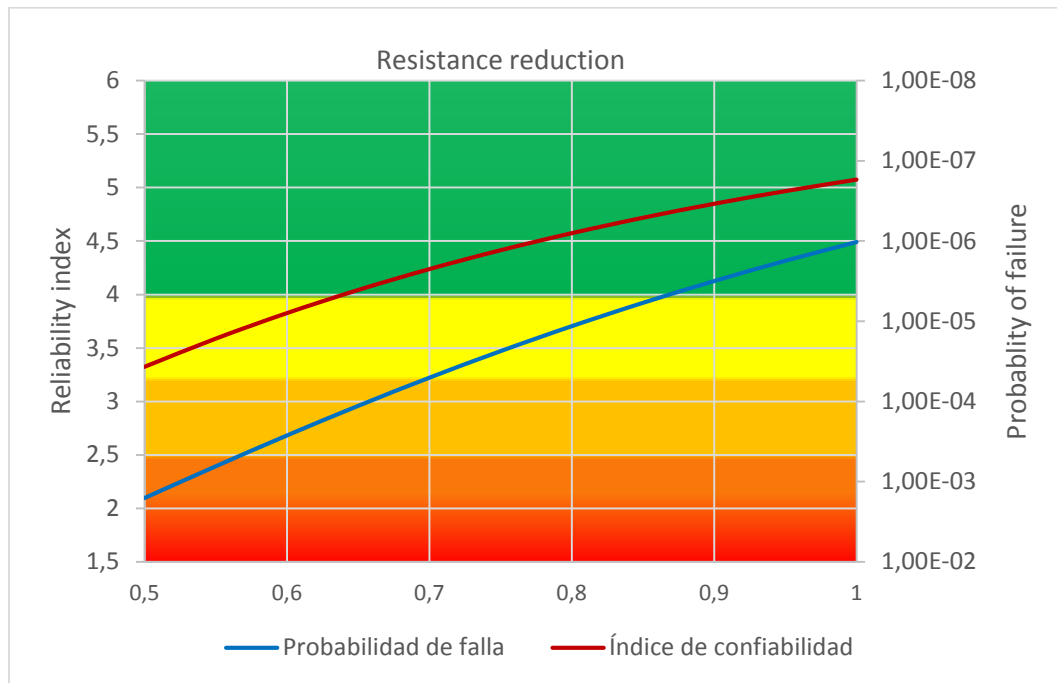


Figure 25. Influence of resistance reduction on the reliability index and probability of failure.

Using the correlation between reliability level and reliability index proposed by COST TU1406 (see Table 4), the urgency of intervention is established. Regular inspection needs to be carried out on the bridge in order to maintain the very good performance of the bridge (reliability level 1).

Table 4.

Reliability level and urgency of intervention of the bridge

Reliability level	β	Description	Urgency of intervention
1	> 4.00	Very Good (no faults)	Regular inspection
2	3.25 - 4.00	Good (minor faults well within tolerance)	Reassessment should be performed to update the period between inspections
3	2.50 - 3.25	Fair (tolerable faults, no restriction in use necessary)	Reassessment should be performed to plan an optimal time of an intervention
4	2.00 - 2.50	Poor (significant structural defects)	Reassessment and possible intervention shall be performed shortly after an intervention
5	< 2.00	Very poor (seriously deficient, mitigation measures necessary)	Immediate action/intervention is required

Note: Adapted from Stipanovic et al., (2017).

8. Conclusions

Bridge design codes change and evolve to overcome the deficiencies that became apparent after seismic events, that is the main reason why bridges have inherited some specific vulnerabilities related to the codes that were used in its design. bridges designed between 1971 and 1994, are most likely to present shear failure on the plastic hinge than flexural failure (Caltrans, 2006).

Bridges with irregularities in its substructure (i.e. different pier heights) are more vulnerable to seismic events, due to the high concentration of the base shear force at the shortest pier (Julieth et al., 2011).

In order to understand the behavior of the critical zone and the bridge itself under seismic events, the ultimate limit state function was defined by taking into account the randomness of the bridge loading capacity (i.e. the ultimate base shear force that structure is able to withstand) and the seismic loading.

The case study presented an initial reliability of 5.07 which indicates a probability of failure of $1.0264E^{-6}$. The aforementioned means that the structure surpasses the reliability target for safety of 4.7 (probability of failure of $1.3E^{-6}$) (Hajdin et al., 2018), which indicates that the bridge presents a very good performance under seismic events.

Using visual inspection techniques, structural defects were detected. Even doe the structure doesn't present any significant damage that would directly impact its load carrying capacities, more frequent inspections are recommended. One of the reasons that would explain the lack of seismic damage could be that the bridge was designed to withstand a high seismic event (PGA of approximate 0.33 g, calculated using de design spectra in the CCP-14) (Ministerio de Transporte, INVIAS, 2014b), and in the period of time between its construction and 2018, the highest PGA was found in 2015 a 6.3 seismic event that had a PGA of approximate 0.65 m/s² in both N-S and E-W directions.

More detail inspections need to be conducted in order to obtain more reliable data on possible defects that may not be found using visual inspection. Even doe the bridge doesn't present any significant damage, its reliability needs to be updated using data form monitoring and testing due to the bridge age and the possible degradation of concrete shear strength.

9. Future work

The initial reliability of the structure was calculated using values of peak ground acceleration in order to simulate the load solicitations of the structure and was only carried out for one fundamental mode of vibration. This was a first approximation to the reliability of the structures under seismic events, in future work, it is recommended to implement not only the acceleration generated by seismic events but also its duration, and the structure period of vibration for each fundamental mode (Perrault & Guéguen, 2015). This would allow providing a comparison between the reliability obtained using the response spectra defined by the normative and the one obtained from the seismic records.

The assessment of bridge performance under seismic events is more significant than ever, for the structural deterioration caused by seismic events affects bridge reliability and sustainability, especially in structures that were designed with outdated seismic design codes (Dong et al., 2014). This work should provide the framework for developing a methodology for performance assessment under seismic events that would allow obtaining valuable information for the maintenance or the mitigation alternatives.

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