Investigating the Effect of Parameters on Seismic Performance of I Girder Bridges

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Investigating the Effect of Parameters on Seismic Performance of I Girder Bridges

A Thesis in Structural Engineering

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June 2021

Addis Ababa

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A Thesis
Submitted in Partial Fulfillment of the Requirements for the Degree of Master of Science in Civil Engineering

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ACKNOWLEDGEMENT

I am very glad to express my genuine appreciation and my deep sense of gratitude to my advisor Dr. Abrham Gebre for his continuous support, guidance, encouragement and constructive comments he provided me throughout the study in the successful completion of this thesis. I am very much indebted to his for suggesting this topic and helping me at every stage for its successful completion.

I would like also to express my deepest gratitude and to thank my family and relatives for all their material and financial support and encouragement they have given me, without their support I would not have been able to reach this stage. And I would like to thank my friends for sharing ideas and their comments. At last, I gratefully acknowledge my deep indebtedness to all other persons who helped me during the whole period of the work.

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ABSTRACT

One of the major components of transportation systems and networks are bridge structures, and earthquake is one of the natural effects which cause catastrophic damage to bridge structures and leads to the transportation networks to be malfunctioned. Therefore, bridge structures should be functional before and after earthquake because these bridges serve for emergency during and after the earthquake.

Different parameters may have different effect under different seismic loadings; therefore, their effect need to be assessed on seismic performance of the bridges. Undesired failures can be prevented through a comprehensive understanding of dynamic load nature and bridge's structural response while keeping the cost-safety balance. Many structural parameters such as geometry of the bridge and properties of the material of the bridge affect the dynamic response of the bridges. Furthermore, the bridge structural response could be significantly changed by characteristics of dynamic load. In most cases, involvement of numerous parameter and complexity require the designer to investigate the bridge response via numerical study.

Parametric studies were carried out using probabilistic approach for 2-span prestressed I girder bridges. The effects of bridge deck thickness (superstructure mass), concrete compressive strength, reinforcement steel yield strength of and skew angle on the bridges seismic response were studied using nonlinear time history analyses (probabilistic approach). The effect of the variation of these four parameters on the response of the structure is analyzed to determine their effect on seismic performance of the bridge. Computer software, CsiBridge 20 is used to analyze the bridge.

For this thesis, a 2-span prestressed I girder bridges with integral abutments is selected and a detailed inelastic three-dimensional model is created for bridges with different parameters. A set of nonlinear time history analysis (NTHA) for a ground motion suite has been performed using these created models to compute the fragility analysis of the bridges. Modified bridges of the original bridge are modeled by varying one of each parameter and simulated repeatedly for pushover analysis and assessment of seismic fragility of modified models. The above parameters affecting fragility curves and seismic demands of these bridges have been examined using the outcomes from fragility analysis.

In probabilistic seismic evaluation one of the popular tools is fragility curves. For a given seismic intensity level the probability of bridge reaching or exceeding a particular damage level is given by fragility curves which are conditional probability statements. In this study, different parameters of a two-span continuous prestressed girder concrete bridge have been assessed for seismic performance analysis. Results of dynamic analysis of the prestressed girder bridge subjected to suit of ground motions is used to develop analytical fragility curves.

It is observed that some parameters significantly affect fragilities, but not others; the skew angle and yield strength of reinforcement steel are the most influence parameters affecting the system fragility while the concrete compressive strength and bridge slab deck thickness (superstructure mass) have a very small influence on the bridge.

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List of Notations

P	Probability of entering a damage state
LS	Limit state
IM	Ground motion intensity measure
D	Structural demand
C	Structural capacity
$\emptyset$ [.]	Standard normal probability function
S_c	Median value of the structural capacity
β_c	Logarithmic standard deviation of structural capacity
S_d	Seismic demand
β_d	Logarithmic standard deviation for the demand
f_{cc}	Concrete compressive strength
E_c	Elastic modulus of concrete
f_y	Reinforcing steel yield strength
d_{bl}	Diameter of a longitudinal rebar
L_p	Plastic hinge length
μ_Φ	Relative curvature ductility ratio
Φ_{max}	Maximum curvature
Φ_y	Curvature at yielding of the tensile reinforcement
EDP	Engineering demand parameters
$\beta_{EDP\mid IM}$	Lognormal standard deviation (dispersion)
λ	Median PGA value
ζ	Dispersions

1. Introduction

1.1. Background

There are many components in the transportation infrastructure system, from this system bridges are some of the most critical components and transportation system's very essential part and also play the main role in development of economy for a country. By establishing links between two places, example between towns, cities and countries, smooth transportation would be ensured by bridges. For these structures, vehicular traffic under or across it or pedestrian interruption due to distress of bridge structure define the failure of bridge. Earthquake is one of the major causes of bridge failure. During an earthquake, bridge failure costs human lives and causes the transportation network interruption catastrophically and also affects the economy. Direct failure of bridges consequences ranges from property collapse to loss of lives and also indirectly such as emergency facilities failure cause access reduction and disturbs the activities of economy during collapse. Significant damages caused by many earthquakes to several bridges in the previous years.

In previous years around the world many bridges have been devastated by cause of one of the major natural hazards which is earthquake. The examples are Kobe earthquake (Japan) in 1995, Loma Prieta earthquake in 1989, Chi-Chi earthquake (Taiwan) in 1999 and Northridge (California) earthquakes in 1994. These earthquakes caused a lot of damages to bridges structures like severe damage to bridge components or collapse to a large number of bridges. Both bridge design and construction deficiencies are the main sources of bridges poor behavior during these earthquakes.

One approach towards mitigating bridge failure is to ensure that it is properly designed to satisfy performance needs associated with traffic and extreme environmental loads, as well as to meet maintenance and durability objectives. It can be appreciated that evaluating the condition of existing bridges and investigating the parameters effect on the performance of the bridge are essential steps towards mitigating vulnerability or bridge failures.

Under seismic load influence the behavior of bridge structures have been main points of interest over a long period of time for engineers. Although bridges, which resist earthquake, design and construction achievement have been significantly advances, the understanding of bridges seismic behavior gap still remain.

The focus of this thesis is investigating the effects of the modeling parameter uncertainties on the bridge's seismic response. The investigated parameters that will be studied in this thesis include: concrete compressive strength, deck sab thickness, reinforcement steel yield strength and skew angle of the bridge.

These bridge models are created or modeled in detail numerically in CSiBridge 20 software, which can capture the individual components nonlinear analysis response; many bridge models are developed depending on the parameters potential influence. These developed models are used to carry out nonlinear time history analysis (NTHAs). A set of NTHAs will be introduced for the fragility curve development for probabilistic approach. The influence of each parameters on vulnerability of the bridges which are mentioned above will be examined by employing fragility analysis result.

The tools that are emerging to assess the bridges seismic behavior are by using of the fragility curve. Empirical fragility curves or analytical fragility curves can be used for fragility analysis. Fragility curves developed empirically are based on bridges earthquakes damage data which are reported in the past. Seismic response data which is obtained from bridges numerical analysis could be used to develop fragility curve and this curve is called analytical fragility curve. Generally, there are three main parts in fragility analysis: (a) ground motions will be simulated, (b) bridges will be simulated (c) from the bridge models seismic response data fragility curve will be generated.

1.2. Objective of the Study

The objective of the study is investigating the effects of the modeling parameter uncertainties of prestressed I girder bridges to different parameters on the seismic performance of bridges using deterministic and probabilistic approach. The parameters that will be investigated in the thesis are:

- ✓ Bridge deck thickness,
- ✓ Compressive strength of concrete,
- ✓ Reinforcement steel yield strength, and
- ✓ Skew angle of the bridge.

The main objectives of this thesis are:

- Investigate the effects of each parameter on the seismic response and performance of prestressed girder bridges using probabilistic analysis. For probabilistic approach nonlinear time history (NTHA) analysis will be carried out to evaluate the bridge seismic performance. To accomplish this method statistical data evaluation will be used to analyze the effects of different parameters on the seismic performance of the bridges.

- Compare the seismic performance of RC bridges within the parameters and identify the parameter which could affect more the performance of the bridge under seismic action.

1.3. Outline of the thesis

This thesis is organized in 5 chapters.

Chapter 1 includes introduction, the background of various seismic analysis methods for assessing the seismic behavior of structures and the objective of the thesis.

Chapter 2 includes the literature review previous studies on the analysis of bridges and presents a review of available literature related to this study.

Chapter 3 describes the geometric and material property of the bridges that we use for the analysis. In addition, detailed bridge modeling for nonlinear analysis and method of analysis of the selected bridges is presented. Ground motion selection also included in this chapter.

Chapter 4 presents probabilistic analysis of the bridge using fragility analysis which obtained from a set of time history analysis. Then the effects of modeling parameters uncertainty on the seismic response of bridges are discussed.

Chapter 5 presents the conclusions and observations resulting from the study and further research recommendations are also included in this chapter

2. Literature review

2.1. General

Many bridges in Ethiopia were built without considering the seismic effect on the bridge performance and served 20 years and above. There was no seismic consideration during the design of the bridges. Based on these considerations, evaluating the seismic performance of the existing bridges is required. One of the different techniques to examine seismic performance is investigating the effect of various parameters. The preferred method for the investigation is probabilistic approach.

The probabilistic approach is a sensitivity analysis method that is used in the seismic analysis that let the analyzer to quantify the level of confidence in the analysis result, in connection to uncertainty in the input of the model. Uncertainty is always connected with input parameter values such as geometric uncertainty and material uncertainty. The seismic performance of the bridge is evaluated by commonly using fragility curves.

2.2. Factors affecting the properties of RC bridge column

2.2.1 Effects of Properties of Material

Paulay and Park (1975) studied the negative or positive impact of some parameters on reinforced concrete columns. These parameters are yield strength of reinforcement steel, concrete compressive strength, column ductility, and the amount of longitudinal reinforcement steel. From this study, it is found that improving these parameters results in a positive impact on crushing and yield limit states. But this study didn't discuss the relative effects of parameter change.

2.2.2 Effect of Detailing

(Mander, J. B., Priestley, M. J. N., and Park, R. (1988) and Sheikh and Uzumeri, (1982)) discussed the confinement effect on columns performance by conduction several analytical and experimental studies under cyclic and monotonic axial loads. This result shows that an increase in the confinement of the column will increase or improve the flexural strength and behavior of the column and an increase in shear resistance of the column.

2.3. Review of Previous Studies

Many parametric studies were conducted previously to examine various modeling parameters effect on bridges seismic response or performance. Chiu, Y.H., Liu, J.B. and Hwang, H. (2001) studied the sensitivity of behaviors of earthquake ground motions containing material properties (reinforcement bars and concrete), properties of soil, springs stiffness, and magnitude of earthquake for the abutment and foundation modeling, by selecting four span continuous concrete girder bridges located in the central part of USA. The parameters chosen for investigating the bridge's global response were bearing displacement and column curvature for bridges performance evaluation. From the results, they concluded that bridge columns and bearings are highly sensitive to seismic loads.

Nielson, B. G., DesRoches, R. and Choi, E. (2004) the properties of material effects on four different bridges have been studied. In this study, the selected bridges were multi span simply supported prestressed concrete girder bridges, multi span simply supported steel girder bridges multi span, continuous prestressed concrete girder bridges, and multi span continuous steel girder bridges. The conclusion from the study was that the multi span simply supported steel girder bridge's fixed bearing was more vulnerable to seismic load than the component of other bridges. And expansion joints, the bridge deck, columns, and bearings which are the main components of the bridges should be considered in the bridge fragility curve generation as they suggested.

Yigrem Zewdu (2013) studied two existing bridges, which are common to the Ethiopian highway construction, for earthquake load. Continuous concrete girder bridges vs simply supported bridges were assessed seismically. Linear modal response spectrum analysis was performed on analytical three-dimensional models of the bridge. An earthquake ground motions hazard level of 0.1 g acceleration were considered for this seismic evaluation. Responses in bearings at the support of the girders, deck girder, and RC columns were considered. From the result of the study, it has been found that larger demands were obtained from the longitudinal loading than transverse loading in both continuous vs simply supported bridges. Consequently, the continuous nature of the deck tends to transfer seismic demand to different structural components. Therefore, the demands of piers for continuous girder bridges become larger.

By using a simplified beam stick model and a detailed finite element model (FEM), a 3-span continuous concrete box girder bridge were analyzed and the results from both analyses were compared by Ahmed Abdel-Mohti and Gokhan Peckan, (2008). The report shows that if the bridge skew angle is less than 30^o, the simplified beam stick model was better than the finite element model. On the other hand, for bridge skew angle greater than 30^o, the preferable analysis model is the finite element model because higher mode effects can be captured. And also, the performance of the bridges using various skew angles from 0^o to 60^o was evaluated, and they concluded that if the skew angle is greater than 30^o the response of the bridges would significantly rise up. In

general, when the bridge skew angle is less than 30^{o} the change in response of the bridge structure would be slight. Abdel-Mohti et al. (2013) also added that when the skew angle increases, the bridge forces and deformations increase.

The effect of the span number on the bridge response was assessed by Dicleli & Bruneau (1995). In this study, two continuous bridges were used. The first bridge was a three-span bridge, and the second was a two-span bridge. Only one ground motion record is used to conduct a dynamic analysis of the bridge, so other additional researches are required by including other bridges, for example, simply supported bridge, and by utilizing more additional ground motion records in the THA in order to give concrete conclusions on the impact of span number on bridges response to seismic load.

R. Shafiei-Tehrany, M. ElGawady, W. Coffer (2011) studied the bridge nonlinear response and examined hinging of bridge column to the inelastic equilibrium condition. Pushover analysis was implemented to estimate the nonlinear (inelastic) seismic responses or characteristics of the bridge structure by considering ultimate limit state and design earthquake load. The impact of 3 various types of soils on the bridge seismic performance was also evaluated. The ductility capacity and the stiffness of the bridge structure has been increased by using dense sand. Additionally, the bridge post yielding stiffness is significantly affected by changing the soil type.

Park and Paulay (1975) studied the negative or positive impact of some parameters on reinforced concrete columns. These parameters are yield strength of reinforcement steel, concrete compressive strength, column ductility, and the amount of longitudinal reinforcement steel. From this study, it is found that improving these parameters results in a positive impact on crushing and yield limit states. But this study didn't discuss the relative effects of parameter change.

(Jamie Ellen Padgett 2007) studied seismic vulnerability assessment of retrofitted bridges using probabilistic methods. A range of potential retrofit measures for different types of bridges in California was evaluated ranging from response modification to capacity enhancement or partial replacement. This was achieved by generating detailed analytical models of the retrofitted bridges and subjecting them to suites of synthetic earthquake load. The impact of retrofit on the seismic demand of the bridges different critical components is obtained by developing the joint probabilistic seismic demand models (PSDMs). The potential for meeting different measures of performance is assessed incorporating both the uncertainty in the seismic demand and capacity for fragility curve development. The fragility curves help to understand and know the impact of retrofit not only on component vulnerability but also on bridge system performance. The results indicate that different retrofits become more effective depending upon the bridge type and damage level of interest.

(Samy Muhammad Reza 2012) studied seismic performance of multi span reinforced concrete bridge with irregular column heights. In this study, the selected RC box girder bridge had four spans with configurations of different column height. The method he used to study the impact of different parameters on the limit states of each bridge piers was a detailed parametric study utilizing factorial analyses. Three analysis methods have been used to calculate the seismic vulnerability of the bridges with irregular column height in the longitudinal direction. The methods are incremental dynamic analysis, nonlinear static pushover analysis, and fragility analyses. Results from the seismic performance evaluation of irregular bridges with varying column height, the effect of confinement have little impact on the first cracking and first yielding limit states. The smaller tie spacing increases the column confinement. The column confinement provides a higher base shear capacity for first crushing and maximizes the bridge ductility. The bridge with a moderate change in column height performs better than the bridge with a rapid bridge column height change.

2.4. Uncertainty in Modeling Parameters

Uncertainty in the seismic demand placed on high way bridge structures can be divided into two major classes. These uncertainties are aleatoric and epistemic uncertainty. Aleatoric uncertainty is related to built-in randomness within the processes or systems, and therefore can only be controlled and cannot be decreased or minimized. The available data collected previously is little, and it causes statistical error in the evaluation of the aleatoric uncertainty. In addition to this, there is uncertainty in the prediction of earthquake ground motions (because of limited available data), and this uncertainty is epistemic uncertainty. These uncertainties don't come from the method (technique) of application; they always reside in the system or the natural process. Epistemic uncertainty is related with ignorance or a lack of knowledge, and this uncertainty can be reduced by gaining further information and knowledge.

Both uncertainties are resided in both probabilistic and deterministic cases, even though in deterministic scenario uncertainties are masked by included in safety factors during the design process. It is very difficult to reduce sources of uncertainties from any problems, and are always present. Traditionally, natural earthquake ground motion randomness was regarded as aleatoric, but when dealing with synthetically produced seismic loads, the uncertainties build into these seismic loads cannot only be given to seismological processes but also characteristics of both site and path of the ground motion. More Aleatoric uncertainty arising from the natural, un-predictable variation in the performance of the bridge cannot be reduced but managed. Aleatoric uncertainties cannot be reduced by experts' knowledge although their knowledge is used to quantify the uncertainties. Material parameter uncertainties and structural geometric uncertainties are considered in this thesis in addition to the uncertainty from the earthquake loads.

2.5. Nonlinear method of analysis

Nonlinear analysis is a new focus for earthquake engineers nowadays, as it allows ways to determine the realistic behavior of structures in inelastic deformation under large earthquakes. Nonlinear analysis is mostly used in instances where the bridge does not satisfy the code requirements or the bridge possesses significant irregularities that need special consideration in the design. It is done when a bridge shows significant irregularities and basic linear analysis cannot realistically predict the behavior of such a bridge.

In nonlinear static analysis a stepwise raised load is applied incrementally until collapse of the structure or until attaining maximum loads. Nonlinear static analysis is limited to structures where dynamic behaviors of the bridge can be easily identified. The advantage of this method is that it includes the nonlinear behavior of the bridge material.

A nonlinear dynamic method is usually complex but is a rigorous method of analysis in which the material of the bridge can go through nonlinear behavior. The various analysis methods used for dynamic analysis are given in the following section.

2.5.1. Pushover Analysis

Pushover analysis consists of a series of sequential elastic analysis, superimposed to approximate a force-displacement curve of the overall structure. The lateral loads are monotonically increased until yielding of some elements of the structure. The load application will be carried out until a control displacement at the top of monitored bridge pier reaches a certain level of deformation or structure becomes unstable. The lateral forces in pushover analysis can be applied in two ways i.e., force controlled and displacement controlled.

In force-controlled application, the lateral load is applied to the bridge structure. This type of application is useful when the magnitude of loading is known. However, if the load that will be applied on the structure is not known, displacement-controlled method will be applied. The displacement at the top node of the bridge pier is fixed, known as target displacement. While applying the displacement at that node, which monotonically increases from zero to target value, the magnitude of load combination may increase or decrease.

FEMA 356 proposed an ideal force-deformation curve i.e., base shear versus displacement curve (capacity curve) with various performance levels is shown in the Figure 3.1. Five points labeled A, B, C, D and E are used to define the force-displacement behavior of the plastic hinge.

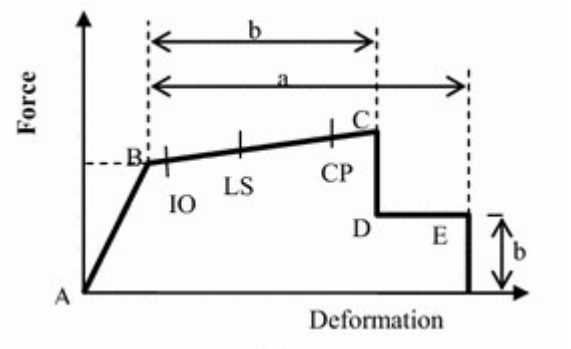


Figure 2-1: force deformation curve

A to B - Elastic state,
B to I.O. - Below immediate occupancy,
I.O. to L.S. - Between immediate occupancy and life safety,
L.S. to C.P. - Between life safety to collapse prevention,
C.P. to C - Between collapse prevention and ultimate capacity,
C to D - Between ultimate capacity and residual strength,
D to E - Between residual strength and collapse,
>E - Collapse.

1) Immediate Occupancy (I0): the damage of structure after earthquake or dynamic load is low or the capacity of the structure is not affected by the applied lateral load. Therefore, life threatening injury is very low.

2) Life Safety (LS): the structural damage due to seismic load or lateral load is significant (partial collapse of the structure) or severe damage to some structural members. Life threatening injury is small.

3) Collapse Prevention (CP): Life threatening injury is significant. Repairing of the structure is not feasible. The structure does not collapse but its lateral and gravitational load resistance of the structure is reduced significantly. Some structural members could collapse.

2.6. Development of Fragility Curves

Fragility analysis is a powerful tool by which earthquake damage of different structures is predicted and it provides a comprehensive seismic performance evaluation of different structures such as networks of transportation and bridge structures. In this thesis, fragility curves are developed through fragility analysis of bridge structure and show the probability of a bridge or a bridge element reach (entering) or/and exceeding the four damage states subjected to a suite of earthquake ground motions.

The bridges seismic performance is assessed or estimated by using fragility curves. Comparing the fragility curves (vulnerability functions) of the as-built bridge and bridges with the changed parameters gives the relative vulnerability of bridges. The probability of reaching or entering a damage state for a given intensity of earthquake load is obtained from fragility curves. And this conditional probability is presented as:

-

$$\text{Fragility} = P[LS|IM = y] \tag{2.1}$$

where *LS* is the damage limit state to the bridge structure or element, *IM* is the ground motion intensity measure and *y* is the selected ground motion intensity measure in terms of PGA (peak ground acceleration). The figure shown below depicts the continuous form of a set of fragility curves and their interpretation at particular ground motion intensity.

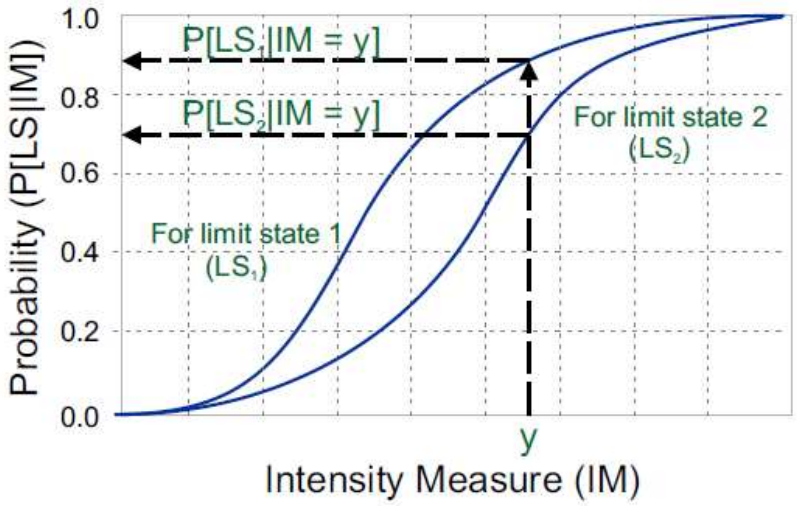


Figure 2-2: Fragility curve

2.7. Types of fragility curves

2.7.1. Expert Based Fragility Curves

Expert based development of fragility curves is one of the easy techniques to obtain fragility curves. Expert opinions are used to obtain the response data of the bridge, which are utilized to generate judgmental fragility curves. Opinions of researchers and engineers are the best choice if the information about the recorded data is not sufficient or complete. The probable damage distribution of various bridges subjected to different intensities of earthquake ground motions is estimated by expert panels of structural engineers. Based on the predictions of the experts, the probability distribution functions will be developed to represent the damage of the bridge structure and the bridge components at each intensity level of earthquake ground motion. Then the probability of a specified damage state is obtained from this distribution function.

2.7.2. Empirical Fragility Curves

Bridge empirical fragility curves became more common after the Northridge earthquake (1994) and the Loma Prieta earthquake (1989) as a result of damage data of a bridge and earthquake ground motions data. Empirical fragility curves give the damage level of a bridge for a given intensity of earthquake ground motion based on previous bridges damage data from events of earthquake. These fragility curves are developed by using the actual damage data of the bridge from the inspection of post-earthquake reports and spatial distribution of earthquake information. Then the fragility curves are presented by correlating the two data sets for given damage states and types of bridge.

Many researchers have given methodologies and generated empirical fragility curves from events of the recent earthquake. Some of these researchers are Kiremidjian and Bazos (1999). In 1999 both researchers together generated empirical fragility curves for bridges damaged in the Loma Prieta (M_w=6.9) and the Northridge (M_w=6.7) earthquakes.

Even though this method is relatively simple and easy, it has some drawbacks. One of the drawbacks is that it is hard to obtain a sufficient number of bridges belongs to one class of bridge that has the same particular damage state. Therefore, it is hard to get results that are statically significant (Shinozuka (1998)).

A second limitation of empirical fragility curves is that different inspectors give different damage levels. Therefore, there is a discrepancy between the damage levels when bridges are assessed after the earthquake and assign damage levels (Basoz and Kiremidjian, 1997).

2.7.3. Analytical Fragility Curves

Fragility curves can be developed through the analytical method in the absence of sufficient bridge damage data for empirical fragility curves. The capacity of the bridge structure can be related to damage state (damage level), and the demand of the structure relates to the intensity of the ground motion in fragility curves, which gives the probability of meeting or exceeding a damage level for a given intensity of ground motion parameter. The probability that the demand of a bridge structure exceeds the capacity of the bridge structure is determined by equation 2.2 as shown below. This is the appropriate model to assess the bridge structure fragility.

$$P_f = P\left[\frac{D}{C} \geq 1\right] \tag{2.2}$$

where P_f is the probability of meeting or exceeding a specific damage state, C is the capacity of the bridge structure and D is the demand of the bridge structure.

Fragility is often modeled by a lognormal cumulative distribution function where the structural demand and capacity are assumed to be lognormal or normally distributed (Hwang and Jaw, 1990). Thus, a closed form solution for the fragility may be presented as given in Equation 2.3 (Melchers, 1999).

$$[P_f] = \emptyset\left(\frac{ln\left(\frac{S_d}{S_c}\right)}{\sqrt{{\beta_d}^2+{\beta_c}^2}}\right) \tag{2.3}$$

Several methodologies were used by researchers to accomplish this task. The employed methodologies range from simple to rigorous. Some of the methodologies are introduced in the following section.

Elastic Spectral Response

One of the least time consuming and simple approach to developing seismic fragility curves is using the elastic spectral response of the bridge. Yu et al. (1991) assessed the seismic fragility (vulnerability) of highway bridges in Kentucky by using elastic spectral response. Each bridge pier was modeled as a single degree of freedom (SDOF) structures. Then the elastic response spectrum method is used to determine their seismic responses. Hwang and Jernigan (2002) applied and developed this methodology to able to assess the fragilities of the bridge structure.

Non-Linear Static Analysis

Nonlinear analysis of the bridge is considered as an improved method of the elastic spectral analysis, but it is a very time-consuming analysis method. A simplified nonlinear analysis method is a nonlinear static pushover analysis or capacity spectrum method. This method of analysis is not time-consuming. This method produces the nonlinear static pushover curve. Nonlinear pushover analysis has been used by many researchers to develop seismic fragility curves for different bridges due to low time consumption. Besoz and Mander (1999) and Dutta (1999) developed fragility curves for various classes of bridges in the USA. Shinozuka, M., Feng, Naganuma, T., Lee, J., and Maria, Q. (2000) also used pushover analysis on a continuous concrete girder bridge that has a three-span in Memphis.

2.8. Nonlinear Time History Analysis

Nonlinear time history (NLTH) analysis is also another method that can be used to develop seismic fragility curves. NLTHA is one of the most reliable methods of nonlinear analysis, although this analysis method is the most computationally expensive (Shinozuka, M., Feng, Naganuma, T., Lee, J., and Maria, Q. (2000)). Nonlinear time history analysis has been used by many researchers to develop seismic fragility curves for different bridges. The similar procedures to generate the fragility curve for all applications are depicted in Figure 2-3.

Obtaining a suite of ground motions is the first step in fragility analysis. The second step is to probabilistically sample the properties of the structure (geometric and material strength parameters) from the bridge analytical model. N number of different bridge samples will be created for various parameters. Next, the simulations of the bridge will be subjected to the selected ground motions and, each bridge sample will be analyzed with nonlinear time history analysis. Then the peak responses of key elements of the bridge structure will be collected for each simulation.

Then a probabilistic seismic demand model will be developed using the peak responses of the bridge component. A regression analysis of the peak response of the bridge structure and the ground motion parameters will be used to develop a probabilistic seismic demand model. Then the limits state of each bridge column will be determined. Finally, assuming a lognormal distribution, the capacity of the structure and seismic demand will be combined as given in Equation 2.3. The general procedure to generate an analytical fragility curve by using NLTHA is illustrated in figure 2-3, while there are some differences in methodology.

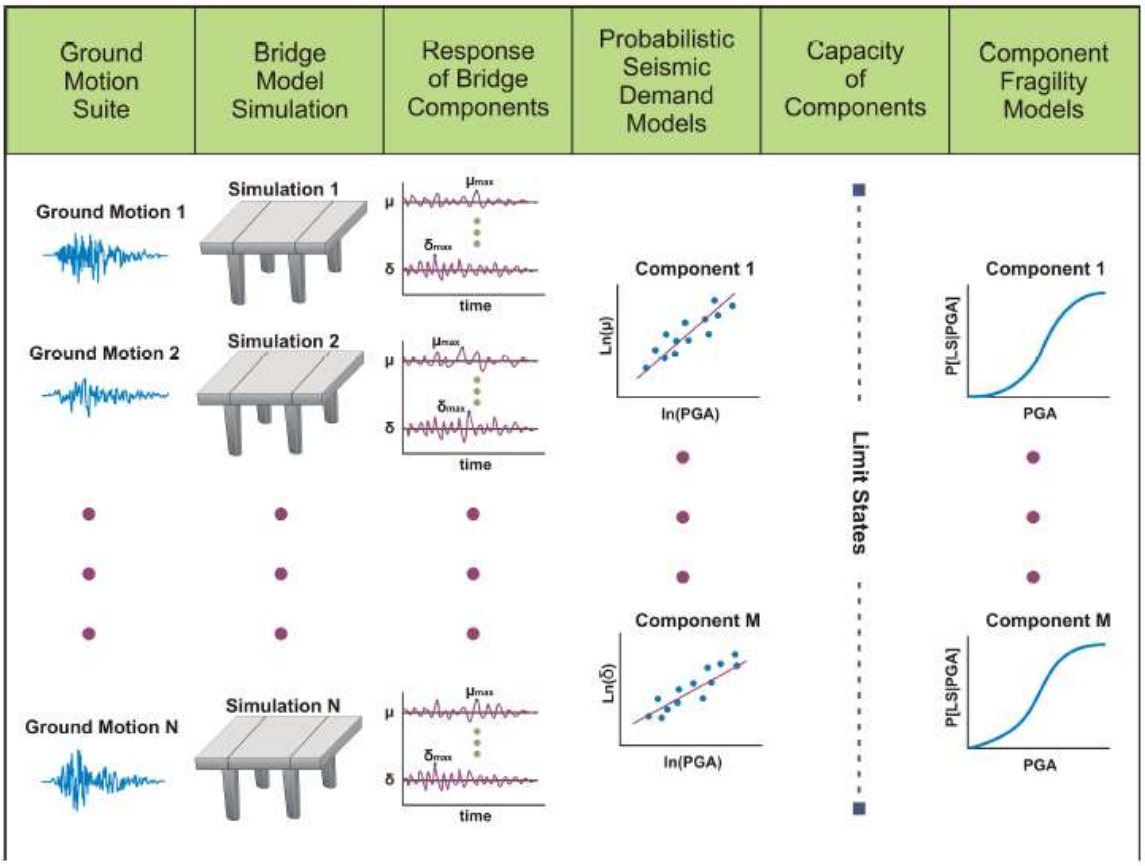


Figure 2-3**:** Analytical Fragility Curve Generation Using Non-linear Time History Analyses (Padget, J. E.)

3. Description and Modeling of Bridges

3.1. Bridge superstructure geometry

In this thesis the selected bridge is a continuous prestressed I girder concrete deck bridge. The bridge is designed based on AASHTO LRFD Bridge design Specifications. Concrete column bents and integral abutment are the structural element which support the superstructure of the bridge. As shown in Figure 3-1, the bridge has two spans with 33.5 m length of each span, and total length of the bridge is 67.0m. The superstructure of the bridge consists of 16.878 m wide (consists of four lanes 3.65 m wide each) and 200 mm. thick, continuous cast-in place concrete deck supported on AASHTO Type VI girders (1.828m deep, 1 m. wide top flange and 0.7112 m. wide bottom flange, 0.2 m thickness of web, area (A_g) is 0.7 m^2 and moment of inertia (I_g) is 0.3 m^4 spaced at 2.94 m) as shown in Figure 3-2. Four column reinforced concrete bent supports the girders, and in between the cap beam of concrete column bent and the girders an elastomeric bearing pad has been placed. The integral abutments support the girders at the end of the bridge.

The multi-column concrete bent consists of four circular columns with a diameter of 1.0367 and height of 5.486 m, 4.3 m is the center to center spacing between columns and also consists of concrete bent of 1.22 m by 1.22 m cap beam. The cross sections of the column and the cap beam are shown in Figure 3-6 and Figure 3-7. The vertical reinforcing bars of the column consist of 16 Φ 24mm, 420 MPa vertical bars and transverse reinforcement bar diameter (10mm) and tie spacing is 0.3 m. The skew angle of the bridge is 20 degrees, valid at each support location. Straight strands arranged along the girders with some strands deboned near the ends of the girders

3.2. Materials

Prestressed girders concrete strength: f_{cc} = 40 MPa
Deck slab concrete strength: 25 MPa
Substructure concrete strength: 25 MPa
Girder elastic modulus, E_c = 32 GPa
Reinforcing steel yield strength, f_y = 420 MPa

Prestressing strands

Diameter of low relaxation strand	= 12.7 mm
Strand area, A_{ps}	= 126.6 mm^2
Steel yield strength, f_{py}	= 1860 MPa
Prestressing steel modulus, E_p	= 200 GPa

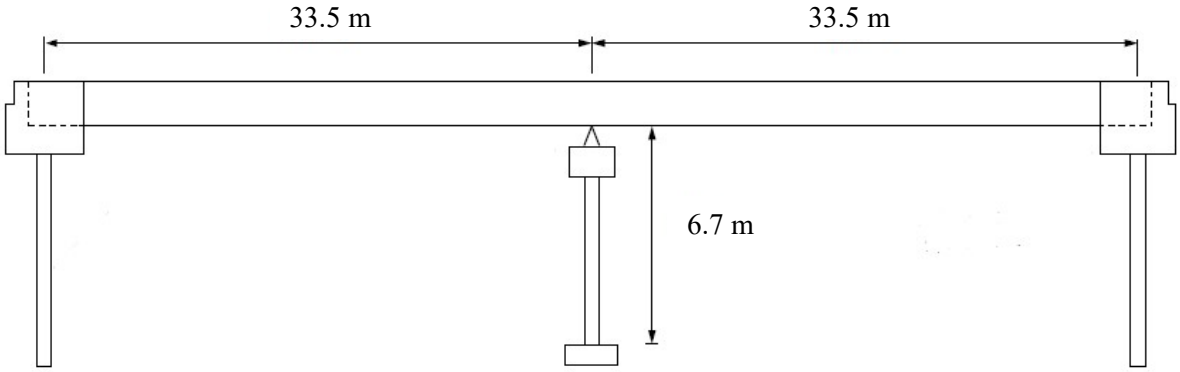


Figure 3-1: Elevation view of the bridge

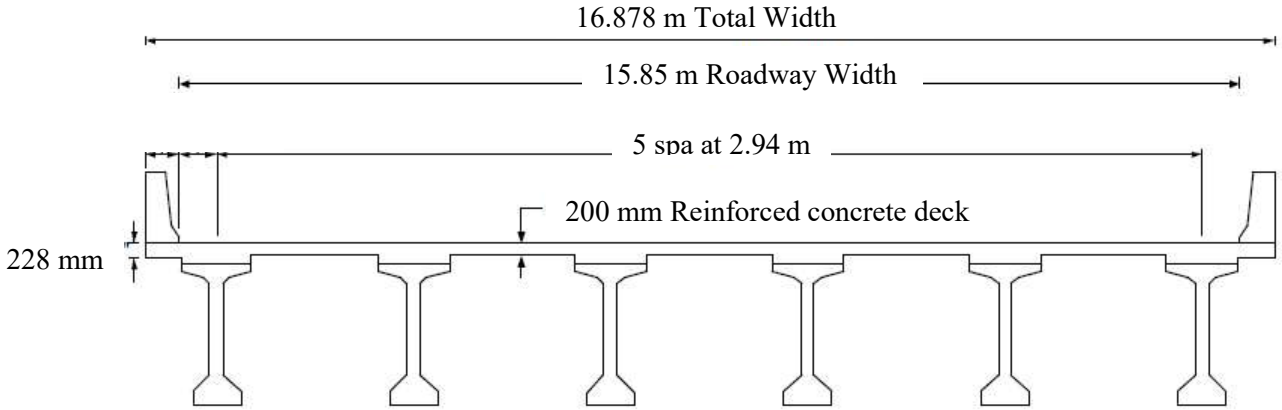


Figure 3-2: Bridge cross section

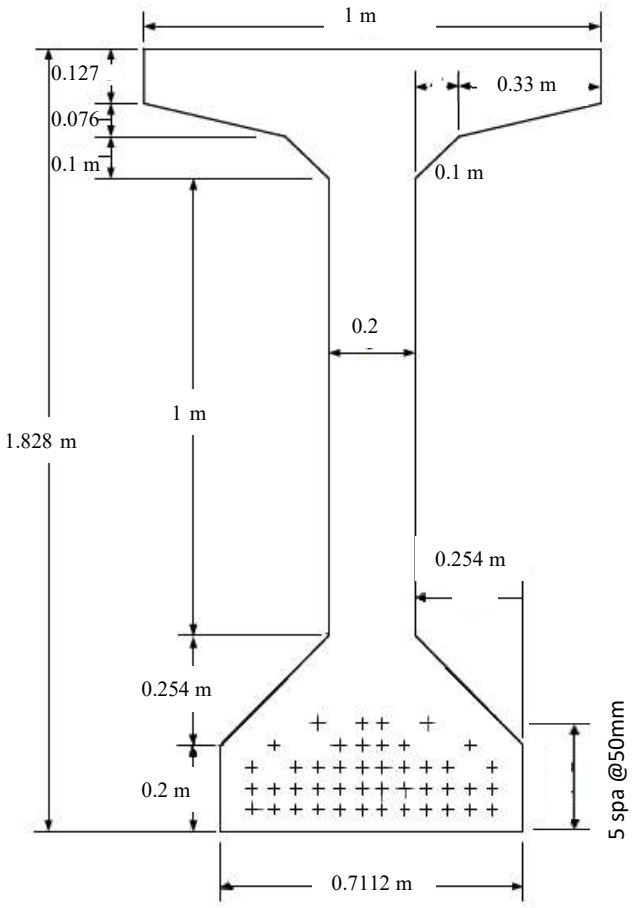


Figure 3-3: I girder beam cross section with 44 strands

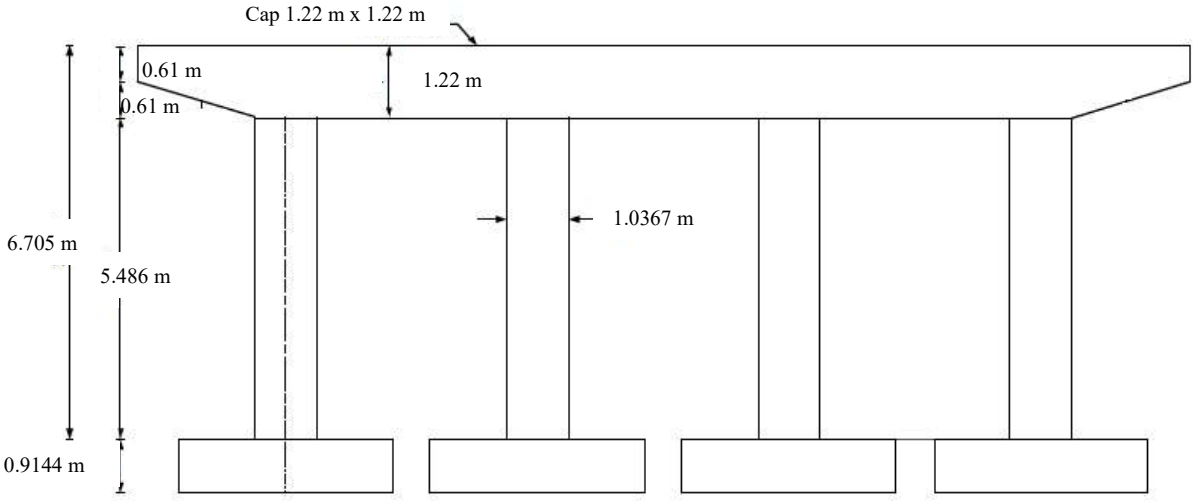


Figure 3-4: Intermediate bent

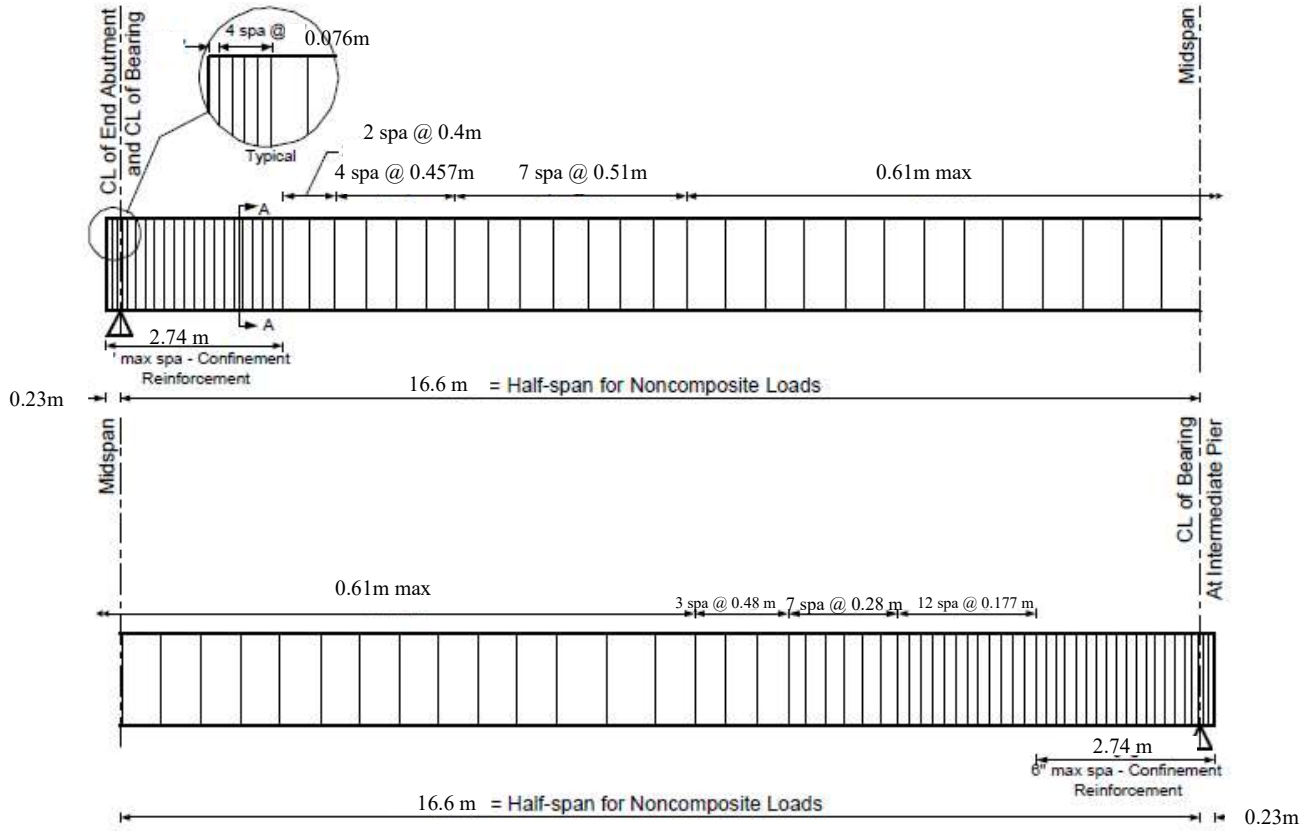


Figure 3-5 Beam transverse reinforcement

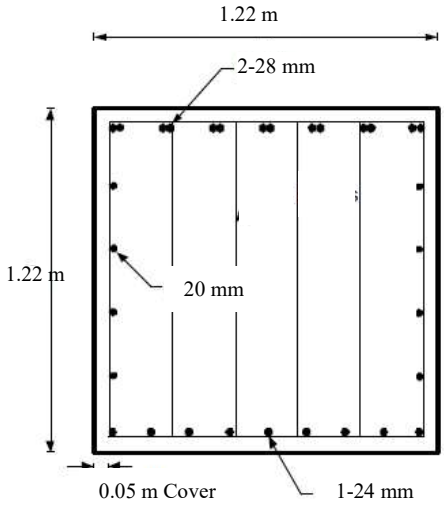


Figure 3-6: Cap beam cross-section

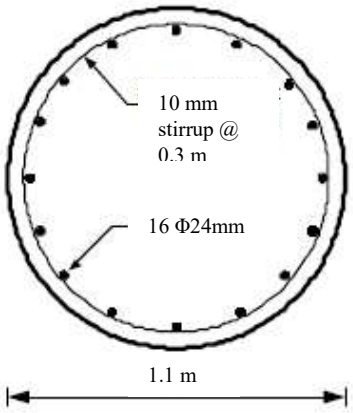


Figure 3-7: column cross section

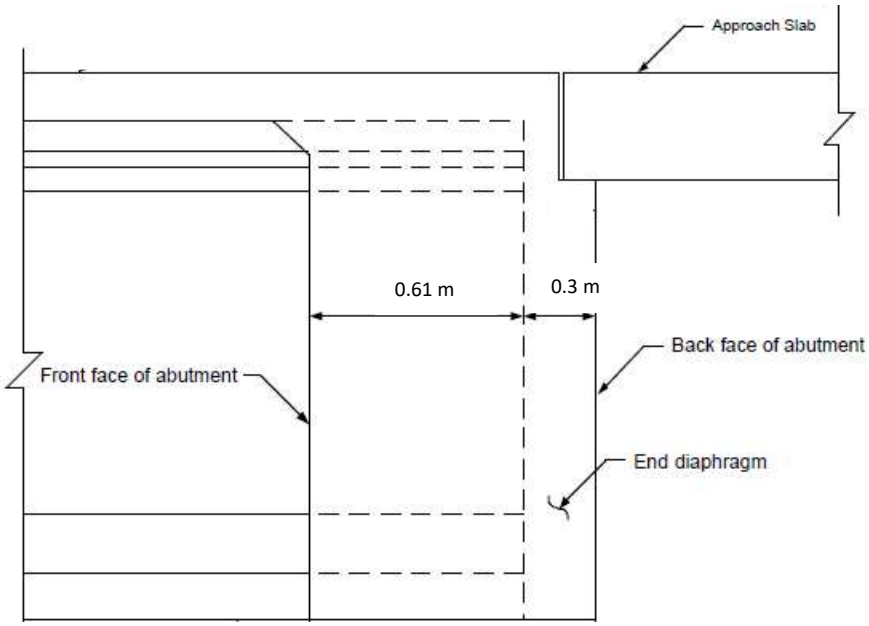


Figure 3-8: Integral Abutment

3.3. Modeling of Bridges

In order to calculate the seismic response of the bridge formation of analytical models of the bridge samples is needed, which are used for analytical fragility curves development. Using these analytical tools and mathematical models quantified seismic response of the bridge can be analytically determined, these responses are member forces, overall structural displacements, and local deformations of the bridge samples.

For lumped parameter models, structural properties such as stiffness, mass, and damping of the structure of the bridge are concentrated or lumped at discrete locations. In this method, to represent true seismic nature or characteristic of the bridge experience and significant knowledge is required even though for bridge modeling, simple mathematical formulations are sufficient. Idealized structural subsystems are the base for modeling of structural bridge components and force-deformation relationships of member end specify bridge components response characterization.

The other method of analysis is the finite element model (FEM). In the FEM, the structure of a bridge is discretized with a large number of tiny elements. Performance characteristics of the bridge structure are obtained from the constituent structural materials. Compared to lumped parameter models, the FEM geometric discretization effort as well as the time required for the structural analysis increase significantly.

In this study, 3-D FEM has developed by using CSiBridge 20 software for structural analysis purposes. The advantage of CSiBridge 20 software is various components of the bridge system could be modeled nonlinearly using plastic hinges and also available link element such as bearings.

3.3.1. Modeling of bridge components

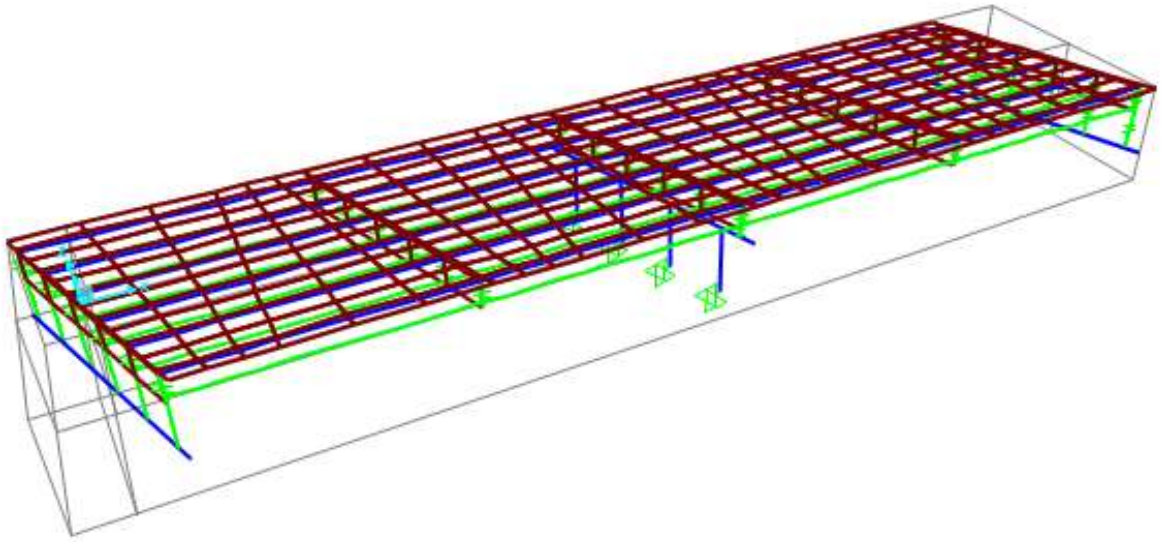


Figure 3-8: CSiBridge 20 model setup for analysis in this study

Structural component analytical modeling will form three-dimensional bridges. The bridges system performance is mainly dependent on the performance of each component as well as components connectivity. Therefore, the bridge seismic response accurate estimation relies on analytical models of the bridge component, which should represent the bridge's structural components the actual seismic characteristics. In general, structural elements of the bridges are divide into two sections, which are substructure and superstructure. The substructure of the bridge will constitute the bent systems and integral abutments. Prestressed concrete girders (precast) and reinforced concrete deck (cast-in-place) form the bridge superstructure. The bearings which are placed between the substructure and superstructure and used as an isolation unit are fixed elastomeric bearings (translation not permitted). For each component of these bridges, CSiBridge 20 software will develop comprehensive analytical models.

3.3.1.1. Superstructure

The superstructure refers to the section of the bridge located above the bearings containing prestressed concrete girders, reinforced concrete deck, and cross frames, and it links through the elastomeric bearings to the substructure; abutments, and column bent. The superstructure, which has two different materials, is composed of precast prestressed concrete girders and reinforced concrete deck (cast-in-place) working as a composite section. During earthquake events, the superstructure of the bridge remains in elastic range without any earthquake damage (Caltrans, 1996). Beam elements of the superstructure are modeled using linear-elastic beam-column

elements, this is because the superstructure is much stiffer than the substructure. Due to superstructure's in-plane rigidity, it is often assumed to move as a rigid body under seismic loads, and the entire modelling problem is reduced to modelling of the bents with geometric constraints simulating the rigid superstructure. Therefore, the superstructure is expected to stay linear.

3.3.1.2. Columns and Cap Beams

Bents are made up of cap beams and columns or only columns depending on the column number of the bent, which is classified as single-column or multiple-column bent. Columns are one of the most vulnerable components in the event of an earthquake; from the past bridge seismic failure experience, column failures are the main cause of the majority bridge failure. Therefore, columns are expected to display nonlinear or inelastic response under severe seismic load and are modeled with nonlinear elements (during earthquake events, plastic hinges are expected to be formed on the top or/and bottom of a column). Nonlinear elements will be used to model the plastic hinge regions while elastic elements will be used to model the rest part of the column (behavior of the columns, outside the plastic hinge zones, are assumed to be elastic).

The bent caps and columns are modeled using a 3D frame element, and the base of the column is assumed to be fixed. The top and bottom of columns are where it is likely to form plastic hinges. (Paulay and Priestley (1992)) The plastic hinge length equal to L_p can be obtained from:

$$L_p = 0.08L + 0.022f_y d_{bl} \geq 0.044f_y d_{bl} \ \ (mm, MPa)$$

Where

L = height of column in mm = 5,486.4 mm
f_y = reinforcement steel yield strength in MPa = 420 Mpa
d_{bl} = column longitudinal rebar diameter in mm = 24 mm

Plastic hinge length calculation

$L_p = 0.08x5486.4 + 0.022x420x24 \geq 0.044x420x24$
$= 438.9 + 221.76 \geq 443.52$
$L_p = 660.66$ mm ≥ 443.52 mm

3.3.2. Modeling of hinges

In the implementation of nonlinear time history analysis, the substructure part of the bridge must be modeled to account for the nonlinear behavior of the structural elements. In this thesis, a point-plasticity (lumped plasticity) approach is considered for modeling nonlinear behavior, in which the plastic hinge is assumed to be concentrated at a column specific points that are both ends of the columns. In this study, pier elements of the bridges are modeled with (P-M2-M3) flexure hinges at column top and bottom under seismic load.

Different options are available to define plastic hinge in CSiBridge 20 software. For this thesis, a plastic hinge is defined using a moment-curvature as the figure shown below and a plastic hinge length that is calculated previously. The bridge column moment-curvature relationship is calculated using a section designer in CSiBridge 20 software.

3.3.2.1. Moment curvature relationship

For the as-built bridge cross-section of the column, both the idealized and nominal moment-curvature diagrams are determined using section designer in CSiBridge 20 and shown in Figure 3-9. In this figure, the graph with a curved line represents the actual moment-curvature curves, and a straight line represents the idealized moment-curvature curves. The cross-section input data are reinforcement steel and concrete nonlinear material properties and the cross-section detailed configuration. Numerical integration methods are used to determine the curvature properties of the cross-section, which is section dependent.

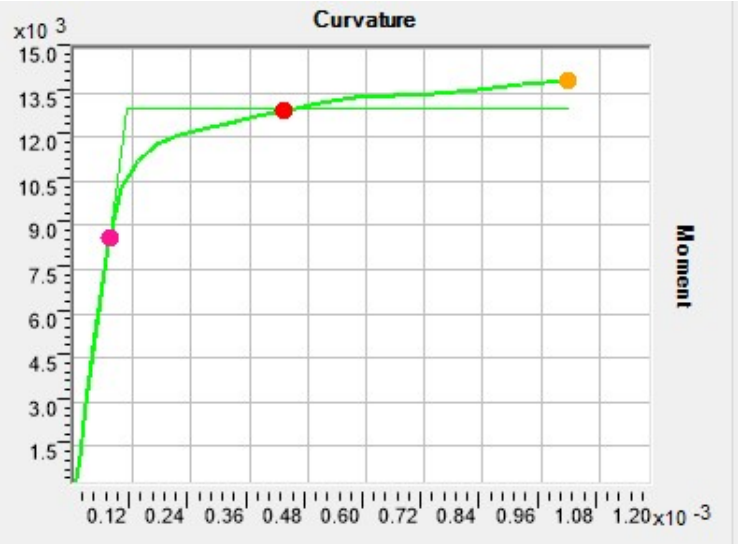


Figure 3-9: Moment curvature diagram

3.4. Method of analysis

3.4.1. Nonlinear time history analysis

The analysis method that has been utilized in this thesis is nonlinear time history analysis for structural component models and seismic response estimation of the bridges. Nonlinear response history analysis is accepted simulation tool, which is the most accurate, combining seismic excitation simulation and characterization of nonlinear component for seismic response of bridge estimation, although NTHA require a lot of time to run the analysis and considerable number of efforts for post processing.

One use of nonlinear time history analysis is for fragility analysis of the bridge. Fragility analysis is a powerful tool which is used for bridge damage or bridge performance prediction and also provides bridge structures compressive seismic evaluation. Fragility analysis is a probabilistic approach which offers the probability of failure of structure for the given damage state for a variety of seismic demands or earthquake loads.

The development of fragility curves can be either analytically or empirically. Empirical fragility curves are mostly obtained from the analysis of previous earthquake damage data of existing bridges. In most cases, the empirical fragility analysis method is not applicable because of the shortage of post-earthquake damage data. The analytical fragility analysis method is the common one due to the accessibility of sophisticated computers. These computers are high-speed computers that can perform many numerical calculations shortly. The uncertainties which are considered in fragility analysis are the geometry of the bridge, the material of the bridge, and ground motion.

3.5. Parameters used

Various input parameters used for the modeling of the prestressed I girder bridge models. These parameters are selected based on code requirements and practical applications. For example -Eurocode limits the reinforcement yield strength to be between 400 MPa and 600 MPa. AASHTO LRFD Bridge manual limit the skew angle requirement to be 0^o – 60^o for bridges and for highly seismic zones Eurocode 8 does not recommend bridge with skew angle more than 45^o. In the other case, there are no accepted limits on the concrete strength of bridges but practical concrete strength for bridges with normal strength of concrete are 25 MPa to 40 MPa. And also, AASHTO LRFD Bridge manual states that the minimum concrete deck thickness (depth) should not be less than 7 in. or 178 mm. List of the parameters used for modeling are tabulated in table 3.2.

Table 3-1: parameters used

Concrete compressive strength	25 MPa	30 MPa	35 MPa	40 MPa
Reinforcement steel yield strength	420 MPa	460 MPa	500 MPa	
Thickness of the deck slab	200 mm	250 mm	300 mm	
Skew angel of the bridge	0º	20º	30º	45º

3.6. Ground motion record selection

Uncertainties are important inputs for probabilistic approach method of performance evaluation of the bridge structures, which are involved for bridge structure capacity evaluation and for seismic demand determination. By combining earthquake hazards on which the structure is located and the bridge structure fragility function we can evaluate the seismic performance of the bridge structure.

One of the major components for the process of developing analytical fragility curve are seismic ground motions. For analytical fragility curves, the main sources of uncertainty are earthquake ground motion parameters, structural parameters variability (e.g., geometric parameter variability and material parameter variability) and definitions of damage state.

The main purpose in selecting the ground motions is to compile a ground motion database representing wide range of seismic forces that impose various degrees of seismic damage on the bridges. Selecting records for time history analyses is that the suite covers a reasonably broad range of intensity measure under consideration along with a range of spectral shapes, durations and pulse properties that may occur in the area.

The relation between response of bridge structure and intensity of earthquake ground motion has to be defined for assessing structural seismic performance. In other words, we should first estimate structural response subjected to earthquake or seismic load with specified ground motion intensities from structural analysis, then we can calculate the conditional probability of exceeding a certain limit state.

In this thesis, 15 ground motion data set have been selected, then these earthquake ground motion records have been investigated to every parameter and their variations using nonlinear time history analysis. The selected earthquake ground motions cover different levels of seismic hazard, wide range (from low seismic hazard level to higher seismic hazard level), to represent variability of different damage to the bridge for fragility analysis.

The earthquake ground motion properties affect the response of the bridges structure, which could be varying in a large range in terms of peak ground velocity, peak ground acceleration, duration and predominant period. The selected earthquake ground motions contain different seismic characteristics in order to determine the worst response of the bridge structure through nonlinear time history analysis. In this thesis, from (http://peer.berkeley.edu/smcat/), database for PEER ground motion, the fifteen ground motion records have been selected. The selected ground motion properties are stated in Table 3-2.

Table 3-2: Ground motion selection

RSN number	Earthquake name	Earthquake magnitude (M_w)	PGA (g)
RSN126	Gazli, USSR	6.8	0.701707
RSN181	Imperial valley #6	6.53	1.8929
RSN4876	Chuetsu-Oki	6.8	0.89479
RSN585	Baja, California	7.3	1.28
RSN779	Loma Prieta	6.93	0.5699
RSN825	Cape Mendocino	7.01	1.49357
RSN1051	Northridge 01, Pacoima dam	6.69	1.584945
RSN1063	Northridge 01, Rinaldi	6.69	0.874059
RSN143	Tabas, Iran	7.35	0.8539
RSN1231	CHI-CHI, Taiwan	7.62	0.8092
RSN4114	Parkfield 02, Fault zone 14	6	1.13037
RSN4480	L' Aquila, Italy	6.3	0.664046
RSN6906	Darfield, New Zealand	7	0.7634
RSN8157	Christchurch, New Zealand	6.2	1.6469
RSN3968	Tottori, Japan	6.61	0.94

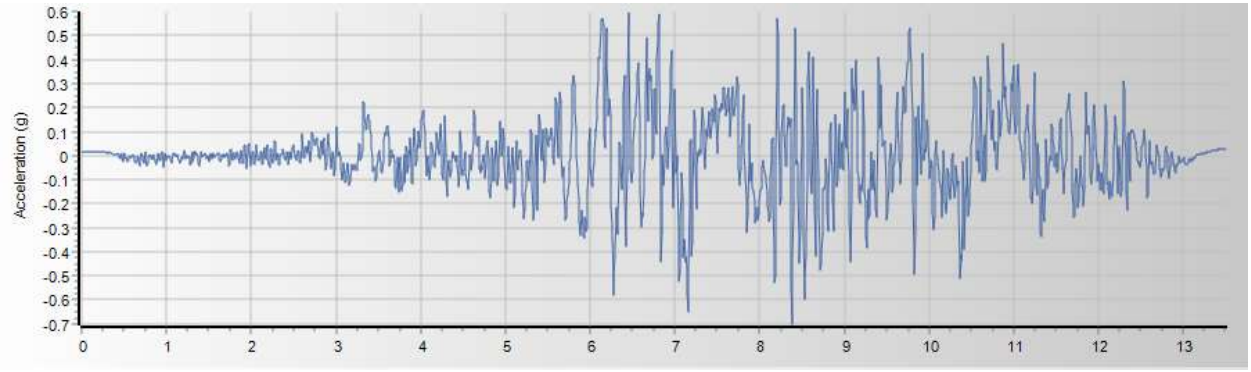


Figure 3-10: Gazli, USSR, Magnitude 6.8 M_w

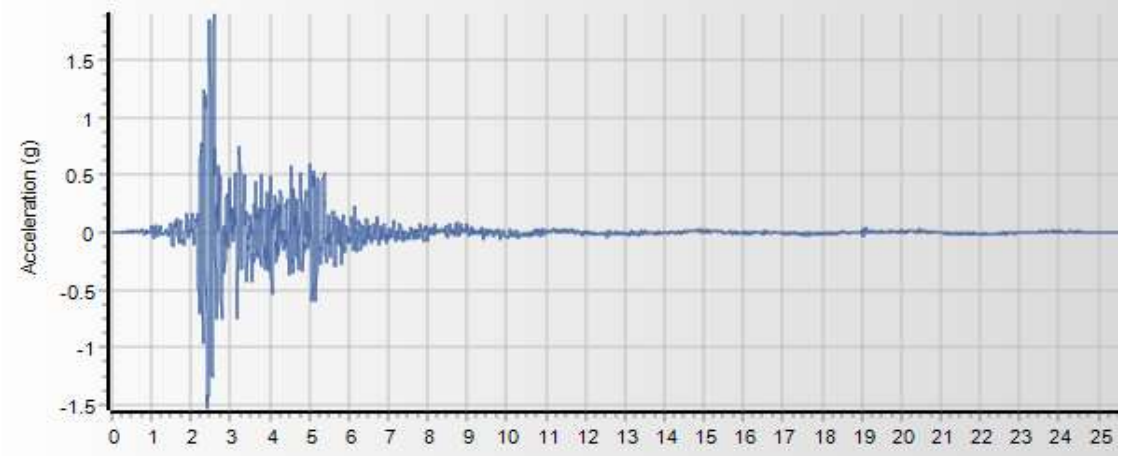


Figure 3-11: Imperial Valley, Magnitude 6.53 M_w

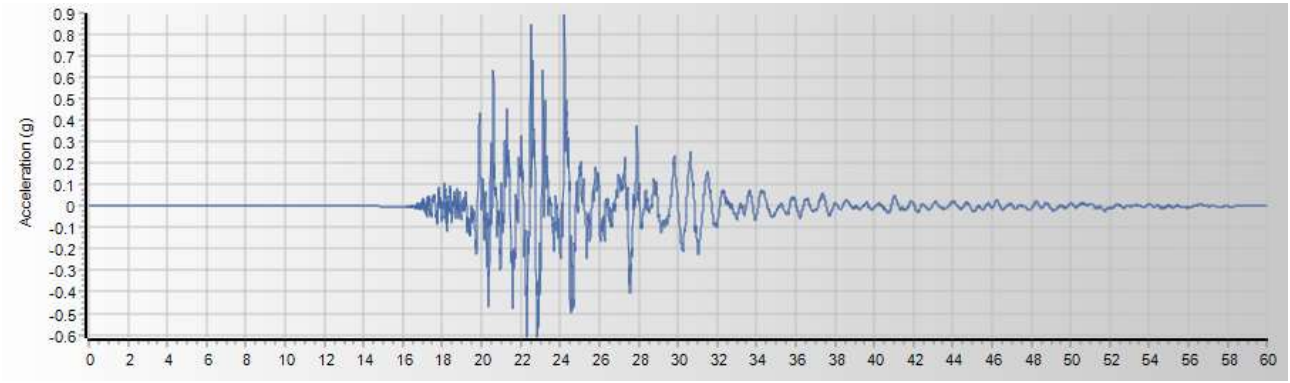


Figure 3-12: Chuetsu Oki, California, Magnitude 6.8 M_w

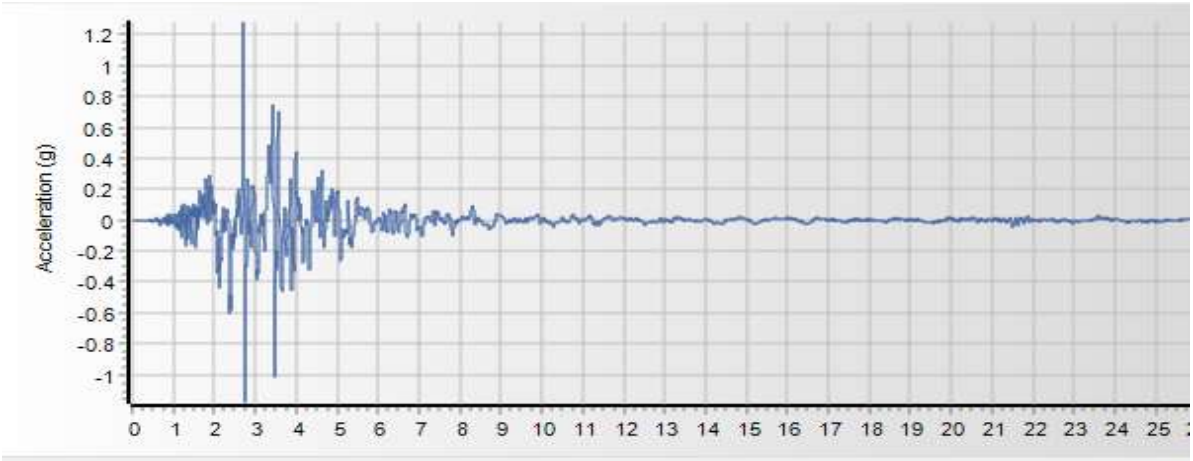


Figure 3-13: Baja, California, magnitude 7.3M_w

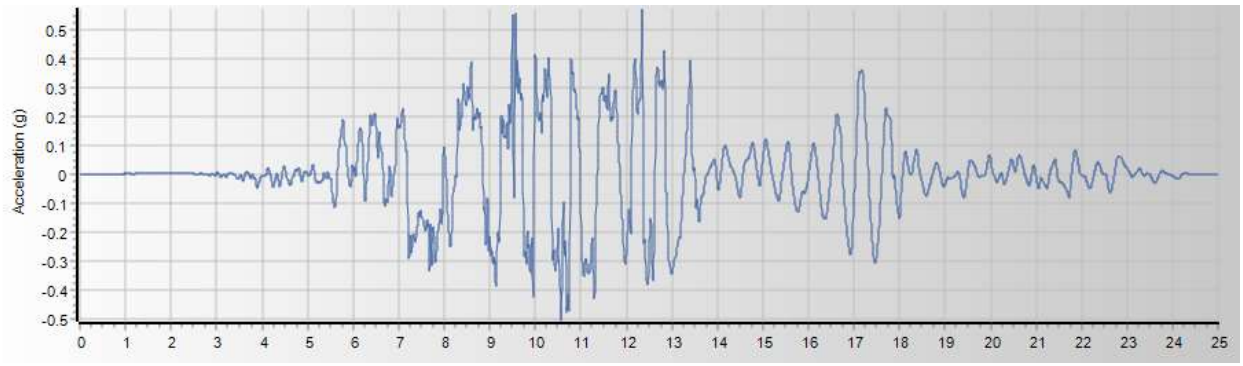


Figure 3-14: Loma Prieta, Magnitude 6.93 M_w

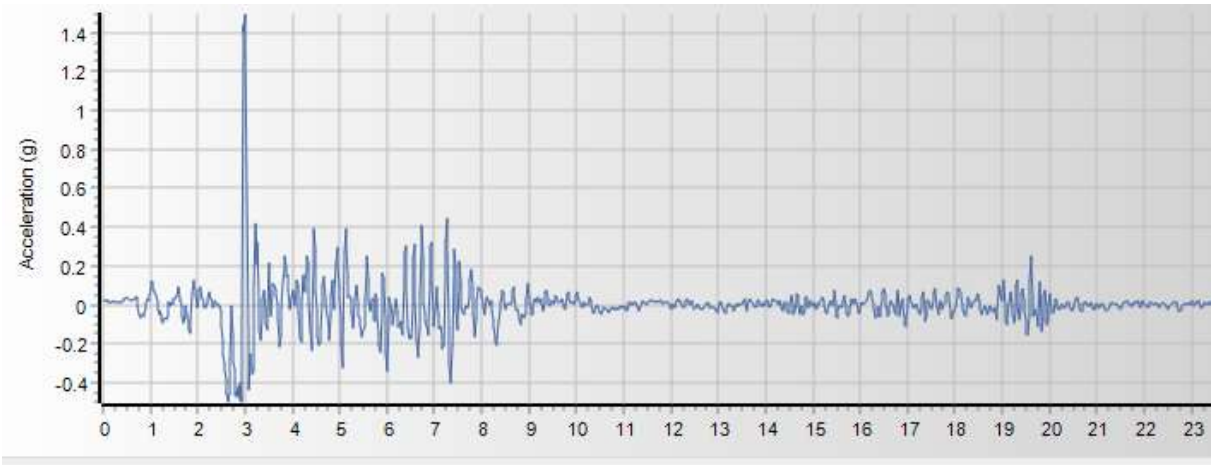


Figure 3-15: Cape Mendocino, Magnitude 7.01 M_w

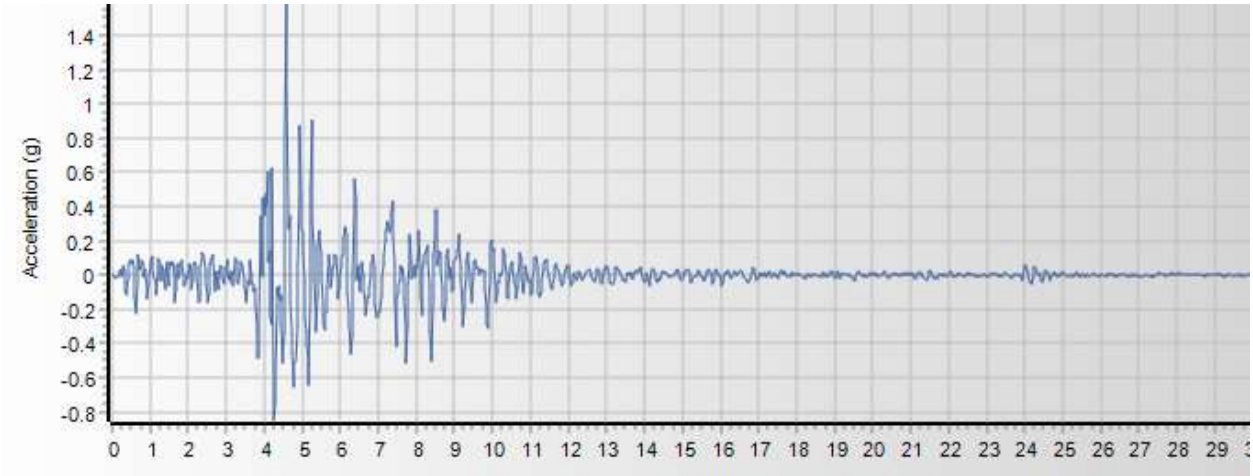


Figure 3-16: Northridge 01, Pacoima dam, Magnitude 6.69 M_w

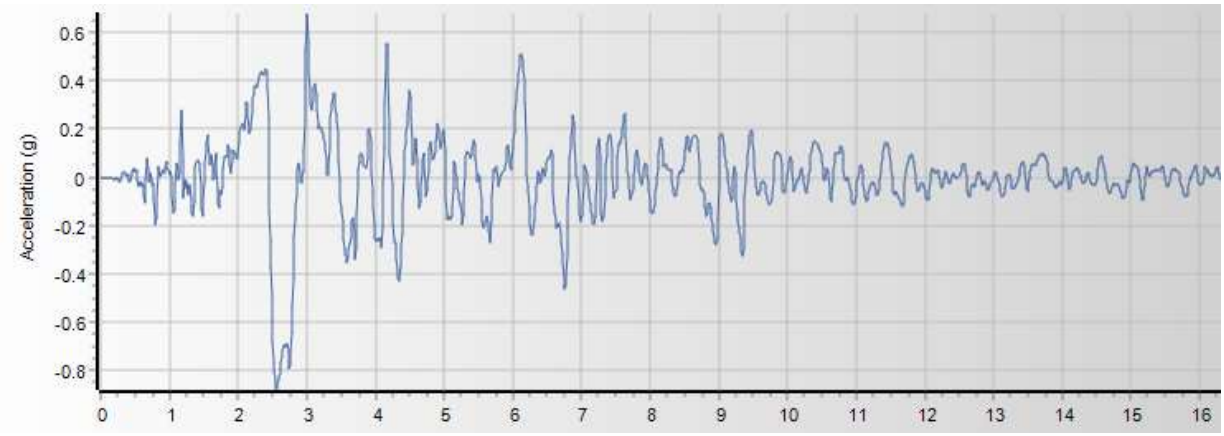


Figure 3-17: Northridge 01, Rinaldi, Magnitude 6.69 M_w

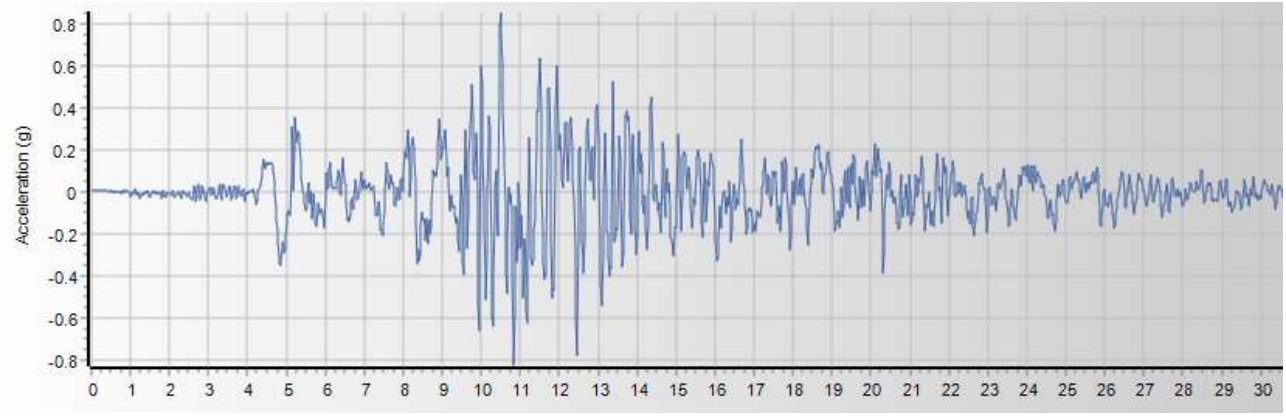


Figure 3-18: Tabas, Iran, Magnitude 7.35 M_w

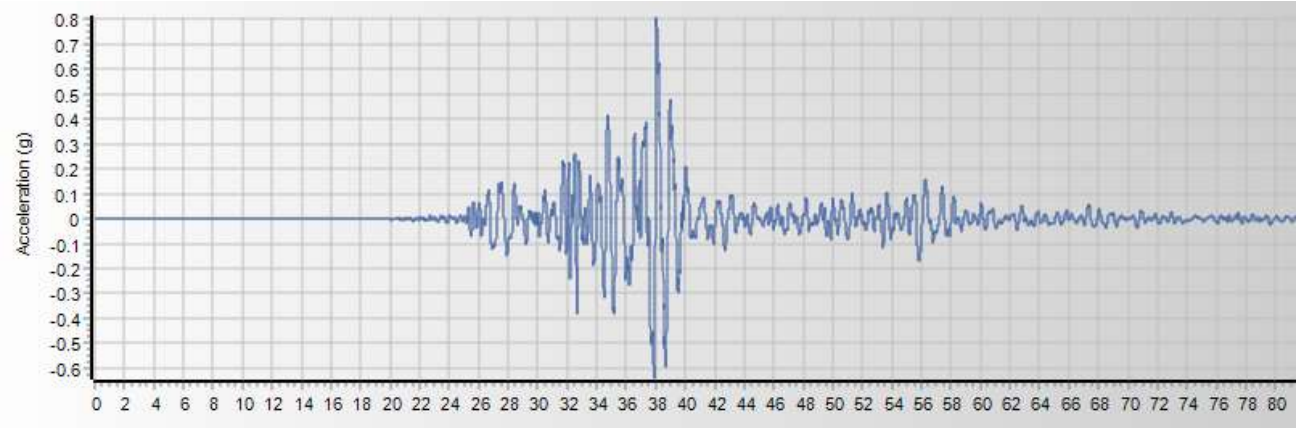


Figure 3-19: Chi-Chi, Taiwan, Magnitude 7.62 M_w

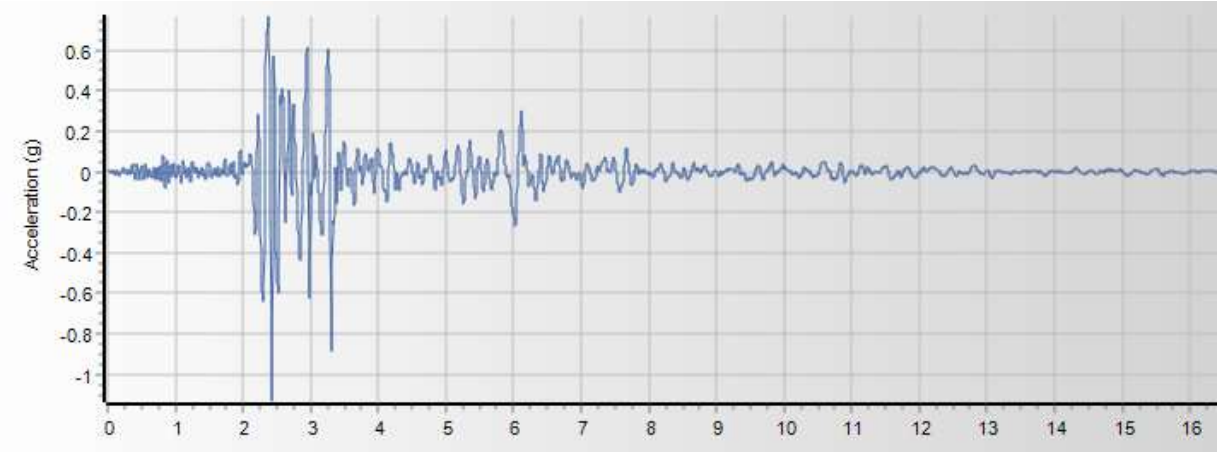


Figure 3-20: Parkfield 02, Fault zone 14, Magnitude 6 M_w

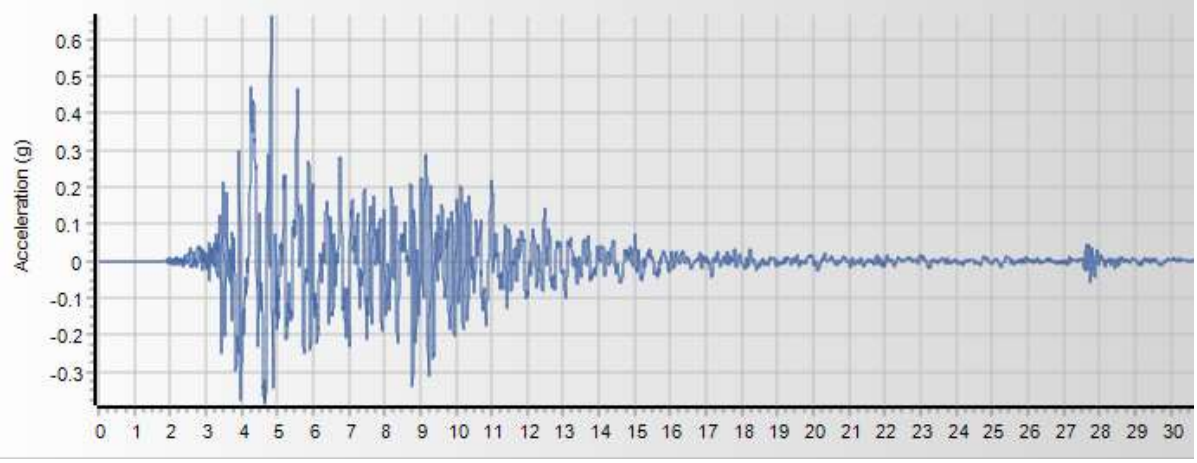


Figure 3-21: L' Aquila, Italy, Magnitude 6.3 M_w

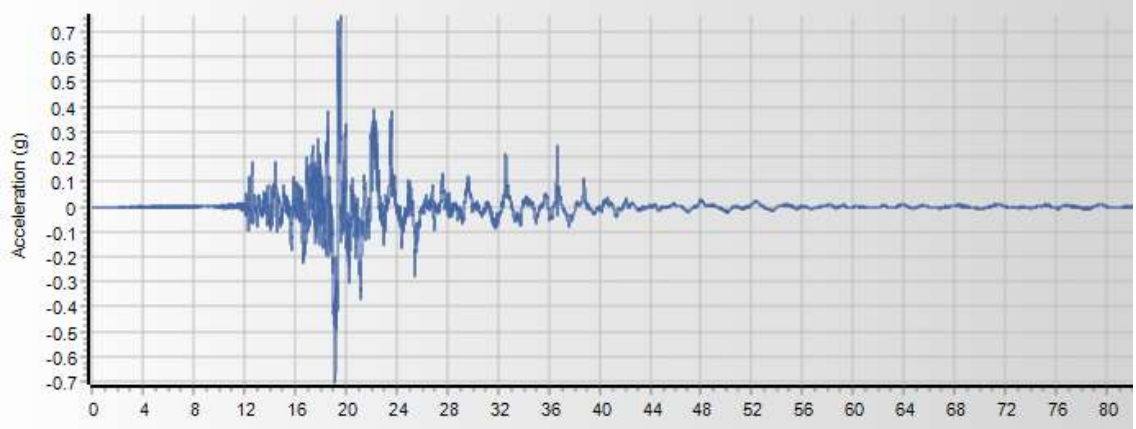


Figure 3-22: Darfield, New Zealand, Magnitude 7 M_w

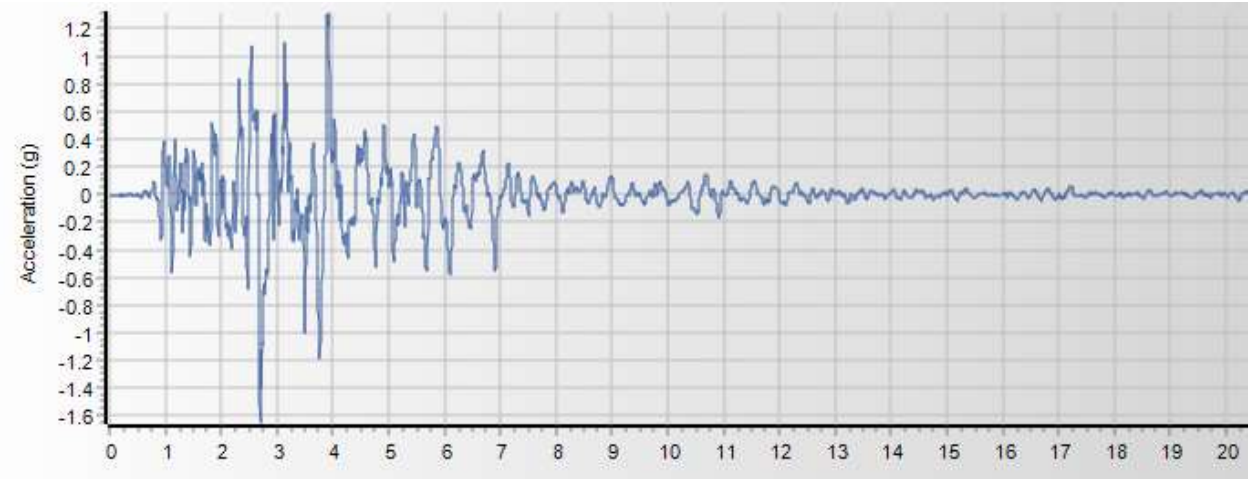


Figure 3-23: Christchurch, New Zealand, Magnitude 6.2 M_w

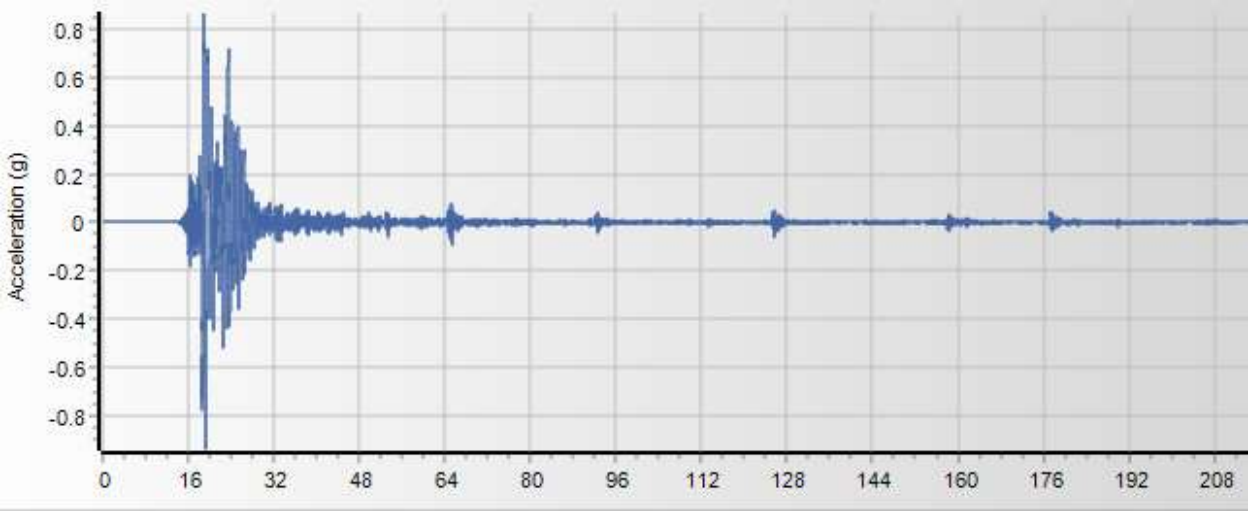


Figure 3-24: Tottori, Japan, Magnitude 6.61 M_w

The response spectra of 15 ground motions are presented in Figure 3-25.

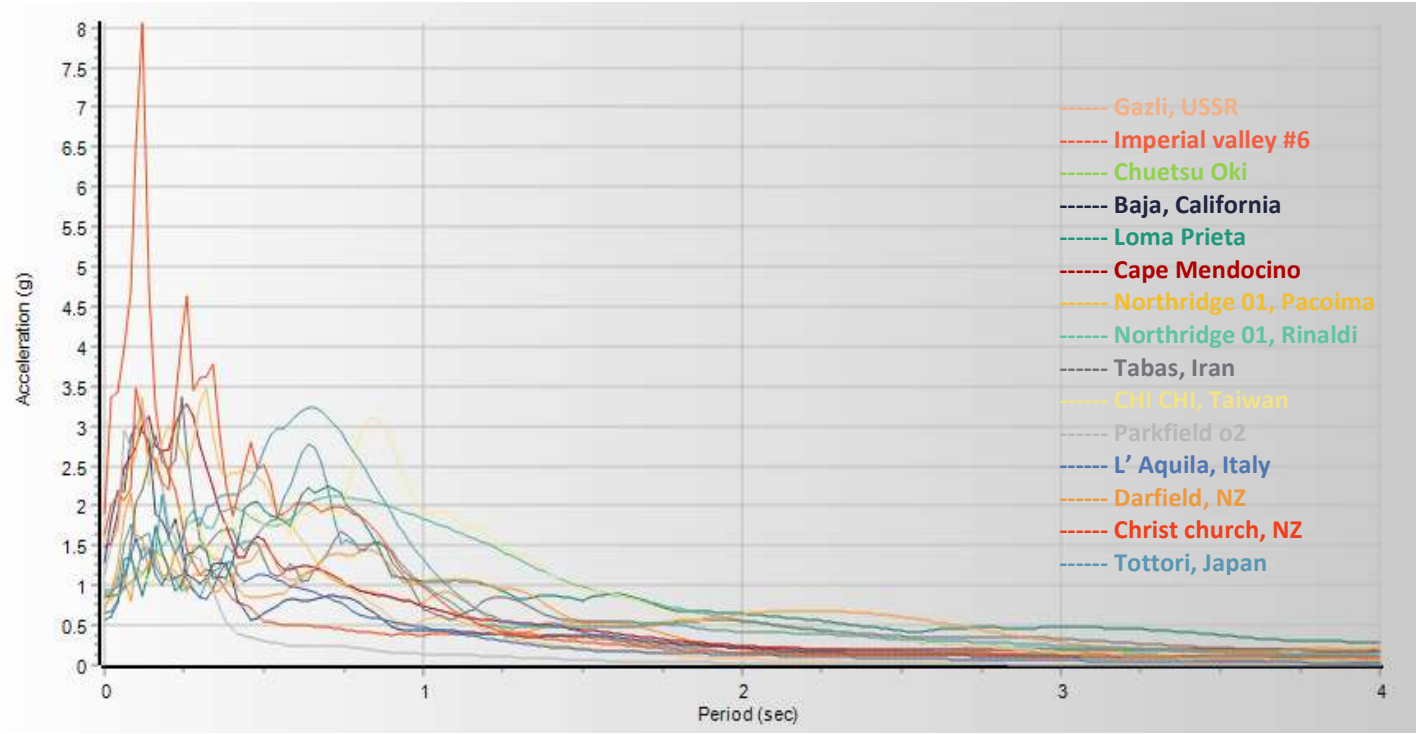


Figure 3-25: Response Spectrum of 15 ground motions

4. Fragility analysis

4.1. Introduction

Fragility analysis is a powerful tool by which earthquake damage of different structures are predicted, provides a comprehensive seismic evaluation of different structures such as networks of transportation and bridge structures. One of the most important elements in fragility analysis (or estimation of structural seismic loss) is fragility curves. For bridge damage estimation during the event of earthquakes, the fragility curve is a very valuable tool. For a given seismic intensity level, the probability of the bridge reaching or exceeding a particular damage level is given by fragility curves which are a statement of conditional probability. The curves could be generated analytically or empirically.

The empirical fragility curves are generated by surveying and studying conditions of bridge structures after seismic actions. These types of curves are difficult to be developed due to a shortage of bridge damage data. In the analytical fragility curve generation method, linear/ nonlinear THA can be used, which considers different uncertainties. The uncertainties are geometric (bridge modeling parameters), material properties, and ground motion. The analytical fragility analysis method is becoming more common these days due to the availability of computers with high-speed processing for numerical seismic analysis. In this thesis, the effect of compressive strength of concrete, deck slab thickness (superstructure mass), yield strength of reinforcement steel, and skew angle of the bridge parameters have been studied utilizing analytical fragility curves. These fragility curves have been generated from probabilistic seismic demand model (PSDM) results. These curves are generated based on nonlinear time history analyses under several ground motions as a function of peak ground acceleration (PGA).

One of the main goals of a fragility analysis is to study the uncertainties within bridge structures for assessment of seismic performance and quantify the potential damage of the bridge probabilistically. Uncertainty in material parameters, the geometry of the bridge structure, and ground motions, which affects the demand estimate, are the sources that are considered in this study.

The fragility functions are conditional probability functions that give the probability that a structure will reach or exceed a specific damage level for a selected ground motion. The conditional probability function is given by

$$\text{Fragility} = P[D \geq C|IM] \qquad \text{or} \qquad P_f = P\left[\frac{D}{C} \geq 1\right] \tag{4.1}$$

where P or P_f is the probability of exceeding a specific damage-state, D is the structural demand and C is the structural capacity.

4.2. Uncertainty in Fragility Analysis

Uncertainty estimation and modelling in fragility analysis is an important topic that needs to be addressed in the analysis in order to ensure a reliable vulnerability of the structure. The risks due to uncertainty should be kept and mitigated within an acceptable level, as all uncertainty is impossible to eliminate. There are two types of uncertainty: aleatoric and epistemic.

Aleatoric uncertainties include factors that are inherently random, and usually cannot be reduced and avoided but can be managed. Experts' knowledge cannot reduce aleatoric uncertainty even though their knowledge may be useful in quantifying the uncertainty. Epistemic uncertainties come from a lack of knowledge, which can be minimized depending on the quality and quantity of information we introduced into the problems and proper assessment of the physical and analytical variation of the parameters via methodological studies to improve the accuracy of the modeling parameters.

Epistemic and aleatoric uncertainties should be considered when modelling or performing a fragility analysis. For bridge structures analyzed under seismic loads, uncertainty comes from the capacity and demand of the analysis. Uncertainty from the demand comes from the earthquake action, which contains the ground motion load path and soil conditions. The uncertainties from the capacity come from the geometric and material uncertainty, where the material and characteristics of the designed bridge structure are considered random for the bridge structure.

4.3. Seismic Demand Calculation of the bridge

Seismic response of the bridge components is determined by performing nonlinear time history analysis (NTHA), which is believed to be the most rigorous method in estimating the inelastic seismic demands of the structures. Although NTHA needs excessive amount of run time and post-processing efforts are required, its capability in simulating inelastic seismic behavior is far more superior in comparison with the other analysis methods. Detailed analytical models of the bridges are developed according to Chapter 3 using CSiBridge 20 software.

Fragility curves are developed for each bridge parameter, which is represented by 11 bridge samples. For the NTHA, a total of 15 recordings are employed as the ground motion data set. To obtain the maximum response, a total of 165 analyses are performed.

4.4. Characterization of Damage States

The capacities of the structure of limit states, C, for bridges seismic performance analysis are quantitative performance measures of bridge components or bridge structure. They must have a relationship to the performance level of the bridge and the functionality or over all operation of the bridge structure, referred to as the damage state. The limit states should be mappable to the response level of the bridge structure or demand (D) placed on the structure referred to as an engineering demand parameter (EDP), to evaluate the fragility function as expressed in equation 4.1.

For the development of fragility curves, we need to establish the damage states. Different damage criteria have been established in order to assess the bridge condition, for example, ductility demand and column displacement. For this thesis, curvature ductility has been taken as the engineering demand parameter (EDP) for each damage state for the RC bridge column. And the four damage states are slight damage state, moderate damage state, extensive damage state, and collapse damage state. The nonlinear behavior of a column is defined through a moment-rotation (M-θ) or moment-curvature (M-φ) relation with possible degrading behavior.

The damage state of the bridge column is decided by comparing the bridge column seismic demands with the specified damage limit states according to the corresponding engineering demand parameters. When the seismic demand of the bridge is equal to or greater than the selected damage limit, then the bridge is said to be in the corresponding damage state. The above procedure is repeated for all samples of the bridges under different earthquake loads, then their level of seismic performance is estimated by determining the damage state of each sample of the bridge. In this thesis, to determine the seismic response of the columns of the bridge, many nonlinear time history analyses have been conducted for each bridge. Numerical models of the brides are created and earthquake loads with different levels of intensity are considered to simulate the bridge damage seismically by performing many seismic analyses.

4.5. Seismic damage limit states

One of the major steps in analytical fragility curve development is to determine the bridge damage parameters and their corresponding limit states. The reliability of the fragility curve is influenced by the damage limit state of the bridge. Therefore, specifying realistic damage limit states is needed for the fragility curves to be reliable and thus to estimate the level of seismic performance of the structure reasonably. We choose the damage limit state of the structure based on functionality or operation relation of the structure or the capacities of the structure.

The ultimate point beyond which the specified performance level of the bridge structure can no longer be satisfied is defined as a limit state. Also, operational and functional interpretation is given to every damage limit state. Deformation of the bridge components and the whole system of the bridge is related to damage to the bridge structure. The seismic damage of the bridge structures should have a good correlation with the selected engineering demand parameters (EDP). The seismic response of the bridge represents the bridge seismic damage in terms of the chosen engineering demand parameter (EDP). Therefore, the calculation of both the demand and capacity of the bridge use the selected EDP.

The qualitative description of the 4 damage states (Slight, Moderate, Extensive and Complete) are specified in Table 4-1.

Table 4-1: the qualitative description of the four damage states from HAZUS-MH (FEMA, 2005)

Limit State	Description
Slight	Minor cracking and spalling to the abutment, cracks in shear keys at abutments, minor spalling and cracks at hinges, minor spalling at the column (damage requires no more than cosmetic repair) or minor cracking to the deck.
Moderate	Any column experiencing moderate (shear cracks) cracking and spalling (column structurally still sound), moderate movement of the abutment (<2"), extensive cracking and spalling of shear keys, any connection having cracked shear keys or bent bolts, keeper bar failure without unseating, rocker bearing failure or moderate settlement of the approach.
Extensive	any column degrading without collapse – shear failure – (column structurally unsafe), significant residual movement at connections, or major settlement approach, vertical offset of the abutment, differential settlement at connections, shear key failure at abutments.
Complete	any column collapsing and connection losing all bearing support, which may lead to imminent deck collapse, tilting of substructure due to foundation failure.

In this thesis, the column curvature ductility ratio has been used to express the damage to the bridge column, as shown in Table 4-1. Generally, the capacity of the structural element, section, material, or the whole system of the structure goes through an excessive plastic deformation without losing its load carrying capacity to define the ductility. The damage level to a column was determined in terms of the relative curvature ductility ratio:

$$\mu_{\Phi} = \frac{\Phi_{max}}{\Phi_y}, \tag{4.2}$$

in which Φ_y and Φ_{max} are yield curvature and maximum curvature of the bridge column, respectively.

The damage limit states are given in table 4-2.

Table 4-2: column damage limit states

	Demand Parameter	Slight	Moderate	Extensive	Collapse	Reference
Column	Curvature Ductility	1.29	2.1	3.52	5.24	B. G. Nielson 2005
		1	1.58	3.22	4.18	Ramanathan et al. 2012
		4.89	9.15	12.46	13.08	Ramanathan et al. 2010
		1.44	2.7	3.92	4.18	Ramanathan et al. 2010
		1	2	4	7	Choi et al. 2004

Fragility curves are usually modelled by a lognormal cumulative distribution function where the capacity and demand of the structure are assumed to be normally or lognormally distributed. Thus, the fragility function is presented as shown in Equation 4.3

$$[P_f] = \emptyset\left(\frac{ln\left(\frac{S_d}{S_c}\right)}{\sqrt{{\beta_d}^2+{\beta_c}^2}}\right) \tag{4.3}$$

where Φ [.] is the standard normal cumulative distribution function, S_D is the seismic demand of the structure, S_C is the structural capacity median value, β_D is the logarithmic standard deviation of the demand and β_C is the logarithmic standard deviation for the structural capacity. S_c and β_c values for different demand parameters are given in Table 4-2 (B. G. Nielson (2005)). The damage limit states of RC bridge column curvature ductility specified by Nielson are implemented in this study and presented in Table 4-3.

Table 4-3 column curvature limit state for RC Bridge (B. G. Nielson 2005)

Damage state	**S_c**	**β_c**
Slight	1.29	0.59
Moderate	2.1	0.51
Extensive	3.52	0.64
Collapse	5.24	0.65

4.6. Probabilistic Seismic Demand Models

The relationship between the intensity of ground motion and evaluated demand of structural component (or response of the structural component) is offered by a probabilistic seismic demand model (PSDM) or a PSDM pairs one ground motion intensity measure (IM) and one demand measure (DM) to develop a relationship that can be used for structural component demand prediction. PSDMs are generated by regression analysis using the bridge column peak responses at each ground motion intensity measure and all ground motions taken for the analysis. From the regression analysis, parameters a, b, and $\beta_{EDP|IM}$ (used in equation 4.4 and 4.7) can be evaluated.

Nonlinear time history analysis (NTHA) is performed to determine the seismic response of the bridge components. NTHA is the most rigorous method to estimate the structure inelastic seismic demands. The bridge columns maximum response is predicted by taking the absolute maximum of the response time history of the corresponding EDPs. The response is obtained from the NTHA results.

When using analytical procedures, especially nonlinear time history analysis, probabilistic seismic demand models (PSDMs) are used to describe the seismic demand of the structure and the PSDMs are stated in terms of an appropriate IM. PGA and Curvature ductility have been taken as the ground motion intensity measure (IM) and the engineering demand parameters S_d (EDP), respectively. The local ductility (element- or member-level ductility) is related to the capability of the structural members, therefore curvature ductility is used to calculate local ductility (column ductility). In contrast, displacement ductility is used for global ductility (system-level). The relationship between IM and S_d (EDP) is established in the PSDM, as shown in equation 4.4.

$$S_d = aIM^b \tag{4.4}$$

The transformed space more easily facilitates the estimation of parameters a and b from equation 4.4 by using actual regression. The convenient way to change a highly skewed variable (variable with nonlinear relationships) into a more normalized dataset is logarithmic transformation. By taking the natural logarithm of both sides of equation 4.4, perform the transformation, which gives equation 4.5 in linear form. The logarithmic correlation between selected IM and median S_d is given in equation 4.5.

$$\ln(S_d) = \ln(a) + b\ln(IM) \tag{4.5}$$

Where S_d is the median value of seismic demand, both a and b are regression coefficients and IM is the seismic intensity measure.

The regression nature in transformed space is illustrated in Figure 4-1. The median value of the demand S_d (EDP) and demand variation about the median ($\beta_{EDP/IM}$) is obtained from this regression. In the transformed space, the variation about the median is assumed to be a normal distribution, as depicted in Figure 4-1. Furthermore, the dispersion about the median or mean ($\ln(S_d)$), which is written as σ in Figure 4-1, is the value of the conditional lognormal standard deviation (dispersion), $\beta_{EDP/IM}$. The dispersion of the demand, $\beta_{EDP/IM}$, a function of IM, is given in equation 4.7 (Baker and Cornell, 2006).

$$S_d = e^{\ln(a)+b\ln(IM)} \tag{4.6}$$

$$\beta_{EDP|IM} = \sqrt{\frac{\sum_{i=1}^{N}(\ln(S_d)-\ln(aIM^b))^2}{N-2}} \tag{4.7}$$

where, N is the number of ground motions.

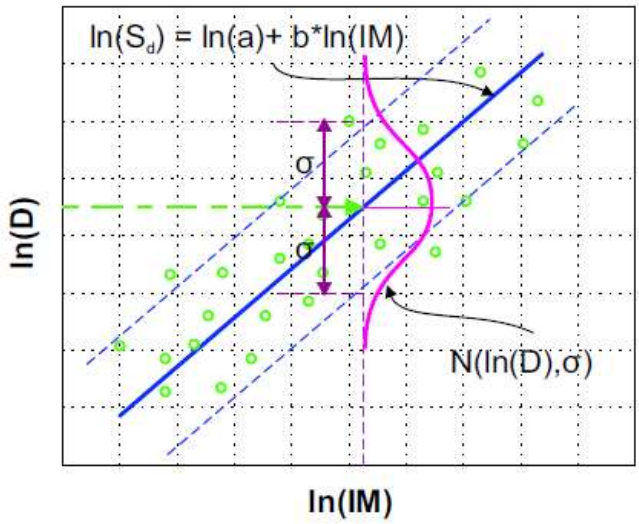


Figure 4-1: PSDM illustration in lognormal space

With knowledge of both the seismic demand and capacity placed on the bridge column, fragility curves for the components can be developed. These fragilities offer insight on the impact of different parameters on the column vulnerability. As previously indicated with a lognormal distribution of the component capacity and demand, the fragility can be evaluated as shown in Equation 4.8.

$$P[D > C|IM] = \Phi\left(\frac{\ln(S_d/S_c)}{\sqrt{\beta_{EDP|\,IM}{}^2+\beta_C{}^2}}\right) \qquad P[LS|IM] = \Phi\left(\frac{\ln(aIM^b)-\ln(s_c)}{\sqrt{\beta_{EDP|\,IM}{}^2+\beta_C{}^2}}\right)$$

$$\text{Or} \qquad P[LS|IM] = \Phi\left(\frac{bln(IM)+\ln(a)-\ln(s_c)}{\sqrt{\beta_{EDP|\,IM}{}^2+\beta_C{}^2}}\right) \tag{4.8}$$

Where S_c is the damage limit state. Equation 4.8 could alternately be written as shown in Equation 4.9 for ease of substitution of the regression parameters.

$$P[LS|IM] = \Phi\left(\frac{ln(IM)-\frac{\ln(s_c)-\ln(a)}{b}}{\frac{\sqrt{\beta_{EDP|\,IM}{}^2+\beta_C{}^2}}{b}}\right) \tag{4.9}$$

In equation 4.9, the denominator term is called modified dispersion, ζ, written in equation (4.10) and is used as proficiency measurement. A more proficient IM is indicated by a lower value of ζ thereby, the selected IM in the demand model has a lower uncertainty.

$$\zeta = \frac{\sqrt{\beta_{EDP|\,IM}{}^2+\beta_C{}^2}}{b} \tag{4.10}$$

And the median (λ) is $\lambda = \exp\left(\frac{\ln(S_C)-\ln(a)}{b}\right)$ (4.11)

Following the Equation from 4.5 to 4.9 and employing the limit states, bridge columns fragility curves are developed for specified damage states.

$$[D \geq d|IM] = \Phi\left(\frac{ln(IM)-\ln(\lambda)}{\zeta}\right) \tag{4.12}$$

4.7. Bridge Fragility Curves

Flow chart for fragility curve development

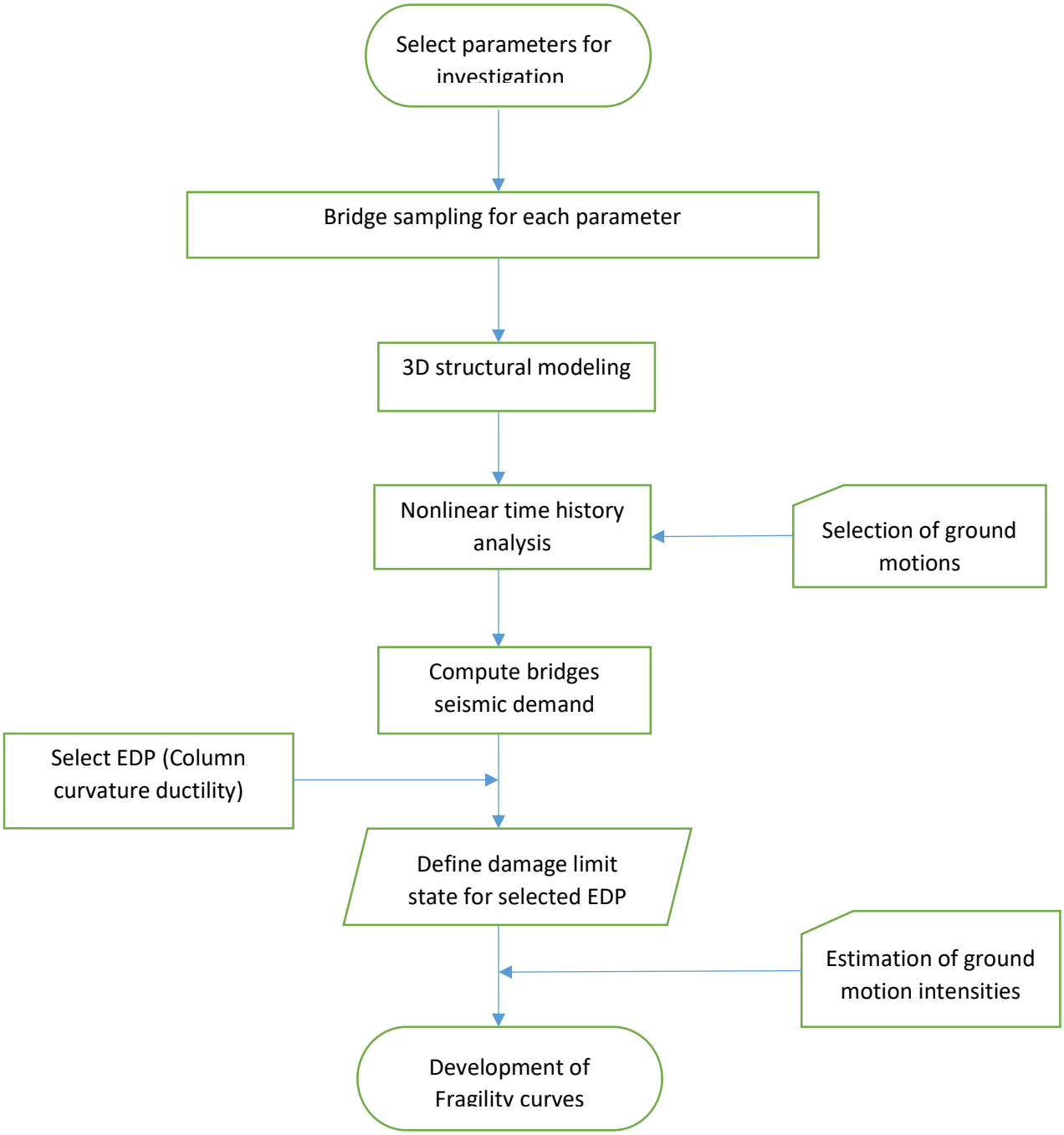
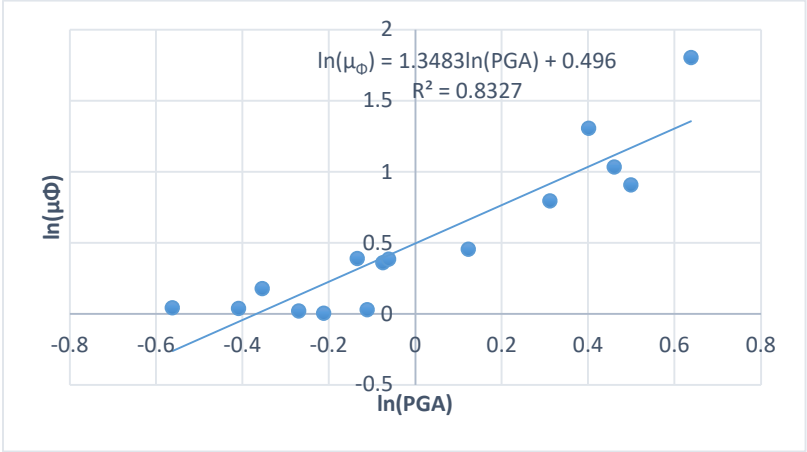


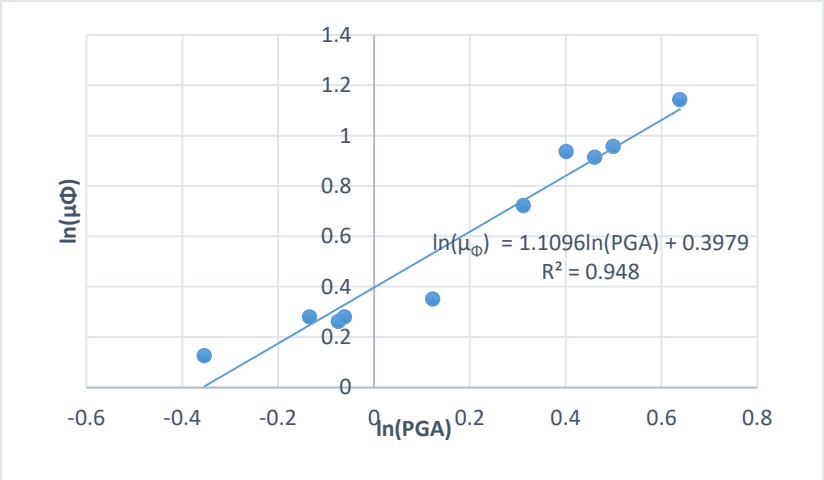
Figure 4-2 shows the probabilistic seismic demand model (PSDM) for three different steel strength parameters. This PSDM gives the relationship between the intensity of ground motion and evaluated demand of structural component (curvature ductility). The relationship between IM and curvature ductility is expressed as equation 4.4 and 4.5. As we can see from figure 4-1, the slop of the regression line is equals to b of the median and the interception point of the regression line is equal to ln(a) of the median, because of the transformed space. Therefore, from figure 4-2 (a) ln(a) = 0.496 and b = 1.3483 and 4-2 (b) ln(a) = 0.3979 and b = 1.1096. Using these results, we can calculate the median and the standard deviation of the normal distribution using equation 4.6 and 4.7, respectively as shown in figure 4-1. Finally, by applying cumulative normal distribution equation (equation 4.8 or 4.12) we estimate the fragility of the bridge, by inserting the damage limit states in to this equation.

420 MPa



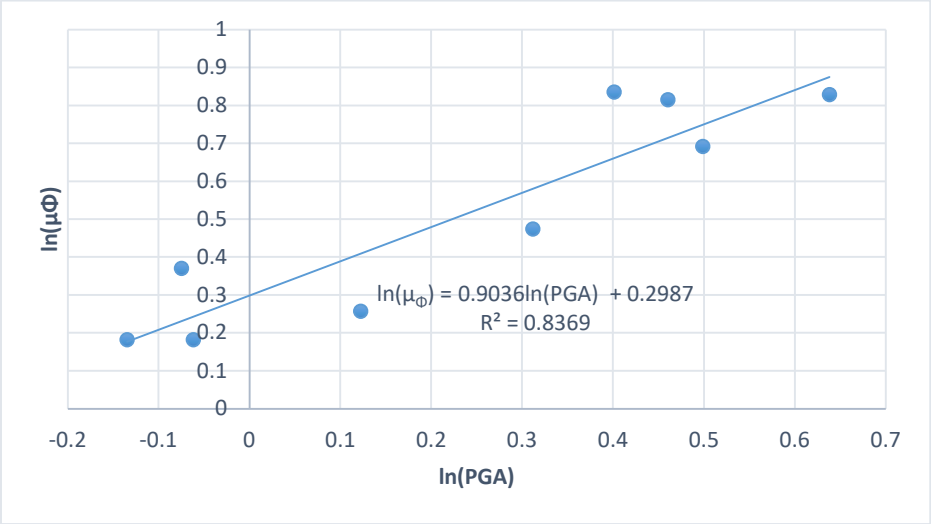
(a)

460 MPa



(b)

500 MPa



(c)

Figure 4-2: PSDM of yield strength of steel

Fragility curves generated for the bridge with a different yield strength of reinforcement steel are shown in Figure 4-3 to Figure 4-5, and the median (λ) values and the dispersions (ζ) are stated in table 4-4.

Table 4-4: Fragility curve for yield strength of reinforcement steel

Yield strength of reinforcement steel	Limit state 1 (Slight)		Limit state 2 (Moderate)		Limit state 3 (Extensive)		Limit state 4 (Complete)		ζ_{avg}
	λ	ζ	λ	ζ	λ	ζ	λ	ζ	
420 MPa	0.836	0.47	1.2	0.415	1.76	0.505	2.364	0.512	0.475
460 MPa	0.911	0.538	1.413	0.467	2.251	0.582	3.222	0.591	0.544
500 MPa	0.952	0.755	1.633	0.58	2.892	0.72	4.492	0.732	0.696

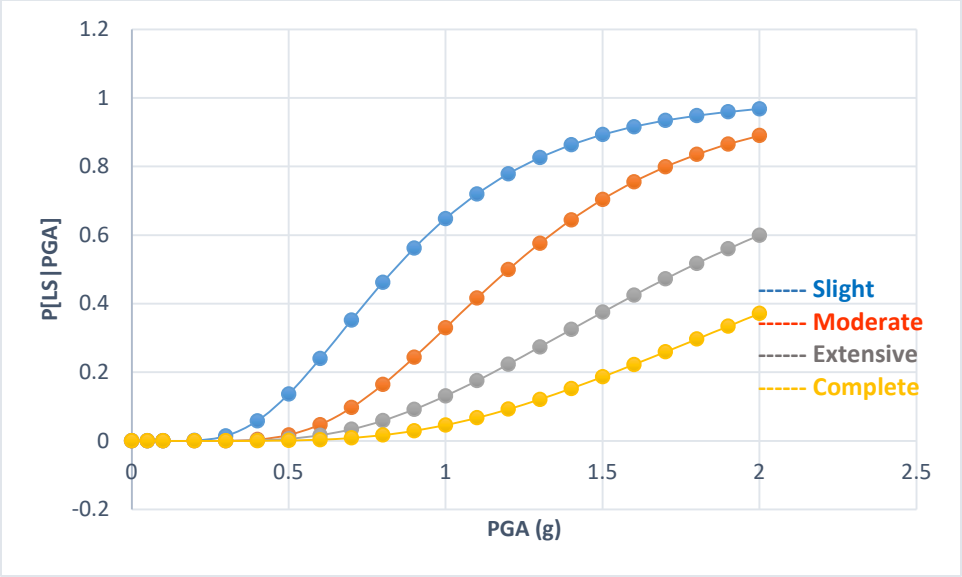


Figure 4-3: fragility curve for 420 MPa steel strength

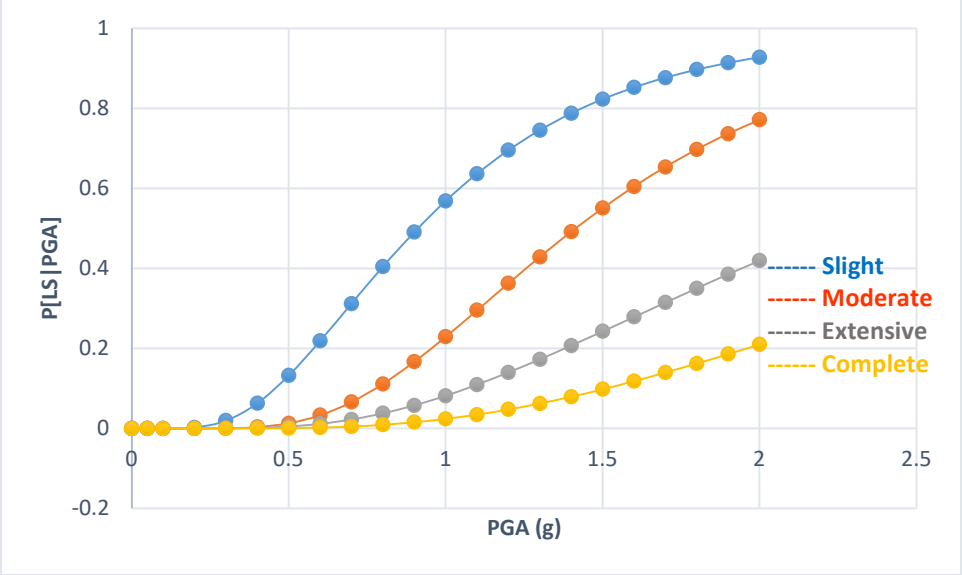


Figure 4-4: fragility curve for 460 MPa steel strength

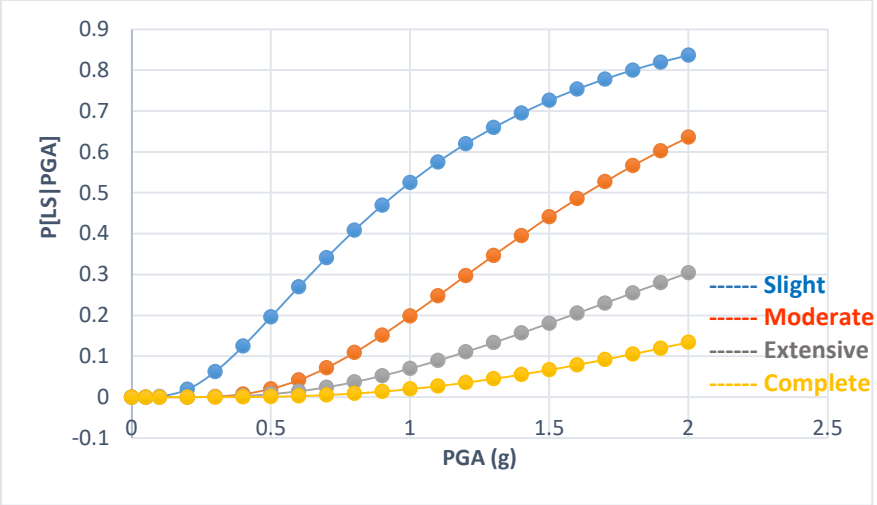


Figure 4-5: fragility curve for 500 MPa steel strength

4.8. Result and Discussion

In this thesis, fragility analysis has been used to investigate the effect of each parameter on seismic performance of the prestressed I girder bridge for probabilistic approach. To implement this method first suite of ground motions have been selected for different ground motion intensities. Next, sampled set of bridges with a consideration of uncertainty in their properties are defined. Then, each ground motion is paired with a sample bridge structure and nonlinear time history analyses are carried out for all bridge-earthquake pairs. Utilizing predetermined damage indexes, a damage state is selected based on curvature ductility of column of the bridge. Then a probabilistic seismic demand model is developed by using a regression analysis of the ground motion intensities (peak ground acceleration) and the peak structural response (column curvature ductility). Finally, fragility curves are developed by using a probabilistic seismic demand model and statistical data evaluation. In order to understand the effect of each parameter on seismic performance of the bridge, fragility curves are compared for each parameter.

The effects of the uncertainties of the reinforcement steel yield strength, concrete compressive strength, bridge deck slab thickness (superstructure mass), and skew angle of the bridge are investigated in this section. Fragility curves were developed for a two-span prestressed I girder bridge by applying selected ground motions to study the effect of these 4 parameters. By these curves, the vulnerability of the bridge in the transverse direction was evaluated to understand the effect of each parameter on seismic performance of the bridge and to compare the parameters effect.

With knowledge of both the seismic demand and capacity placed on each bridge, fragility curves for each parameter can be developed. These fragilities offer insight into the impact of different parameters on the bridge vulnerability. This fragility can be directly evaluated from the capacity estimates for each damage state, S_C, and β_C, presented in Section 4.4, as well as the parameters for the probabilistic seismic demand models (a, b, $\beta_{D|IM}$) from Section 4.5.

The fragilities of the bridges with different parameters are evaluated for each damage state (slight, moderate, extensive, complete) and compared with the original bridge's fragility curve to determine which parameter has significant effect. The fragility curves for the original bridge (as-built) and bridges with different parameters can offer insight on the effect of each parameter within the bridge system and indicate which parameter may be significantly affected by the earthquake effect. Fragility curves for all four parameters are drawn in the next section for all damage state of the prestressed I Girder Bridge. The figures compare the vulnerability of each parameter. Comparing the bridge fragility curve with different parameters provides insights on the parameter's impact on the bridge.

4.8.1. Fragility analysis for concrete compressive strength

Before fragility analysis is performed, the effect of concrete compressive strength on the seismic demand of the column is investigated through median (λ) demand models comparison. Figure 4-6 shows the comparison of the median values. Evaluating the relative change in the median value is a simple technique used for comparison of differences in the fragility curves. If the median value changes in the positive direction, the structure is less vulnerable, or the higher median value indicates a smaller contribution of the bridge column to the vulnerability of the bridge system. But if the median value changes in the negative direction, the structure is more vulnerable. The median values of all concrete compressive strength (25, 30, 35, and 40MPa) are calculated across all limit states and stated in table 4-5.

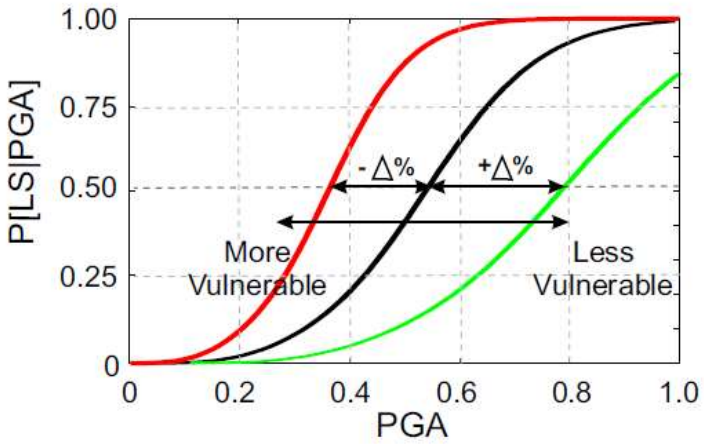


Figure 4-6: Illustration of percent change in median value

Table 4-5: Median for concrete compressive strength

Concrete compressive strength	25 MPa	30 MPa	35 MPa	40 MPa
	λ	λ	λ	λ
LS1	0.836	0.826	0.821	0.84
LS2	1.2	1.2154	1.2	1.2451
LS3	1.76	1.8286	1.7903	1.8896
LS4	2.364	2.5047	2.4383	2.6054

Table 4-5 summarizes the median value of fragility curves for concrete compressive strength. As indicated in the table, an increase in the concrete compressive strength slightly decreases the median value or doesn't have any effect on limit state 4. The medians PGA value of the slight and complete limit states associated with the column curvature ductility increases from 0.836g to 0.84g (4.8%) and increases from 2.364g to 2.6054g (10.21%), respectively, when the concrete compressive strength increases from 25 MPa to 40 MPa.

From a general comparison of the median values, an increase in bridge column concrete compressive strength slightly increases the median PGA value comparing to the lowest concrete compressive strength. The corresponding fragility curves (slight and complete) for the prestressed girder bridge with different concrete compressive strength are presented in Figure 4-7 and Figure 4-8. And fragility curves for four limit states (slight, moderate, extensive, and complete damage limit states) are drawn in Figure 4-9. These figures compare the original bridge fragility curve to the fragility curves of the bridges with different compressive strength of concrete (25, 30, 35, and 40 MPa).

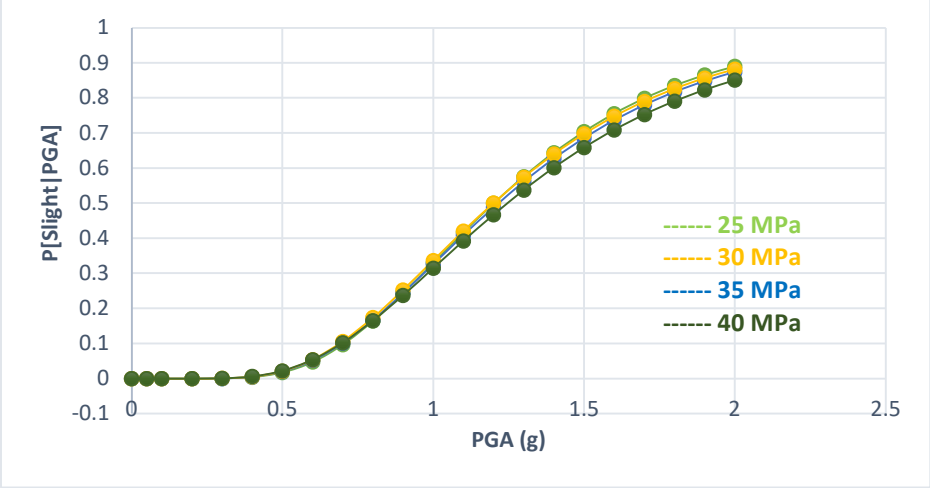


Figure 4-7: fragility curve comparison for slight limit state (LS1)

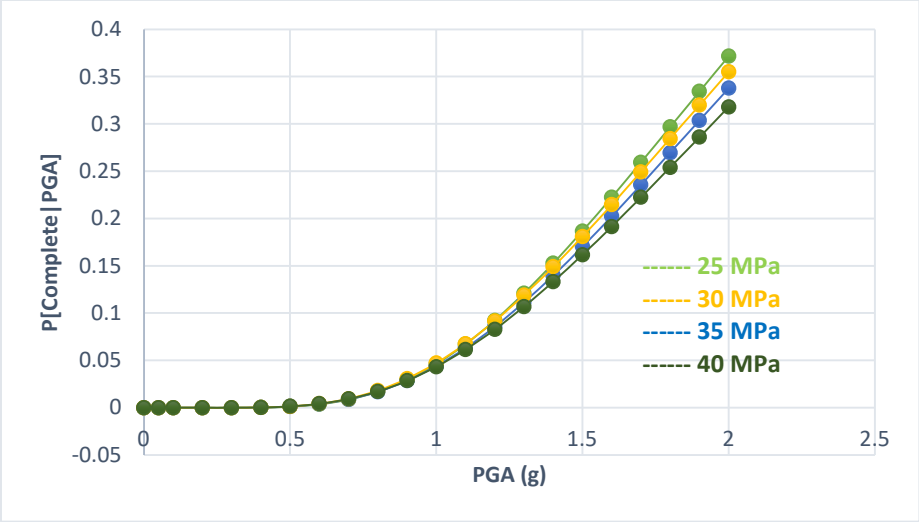


Figure 4-8: fragility curve for complete limit state (LS4)

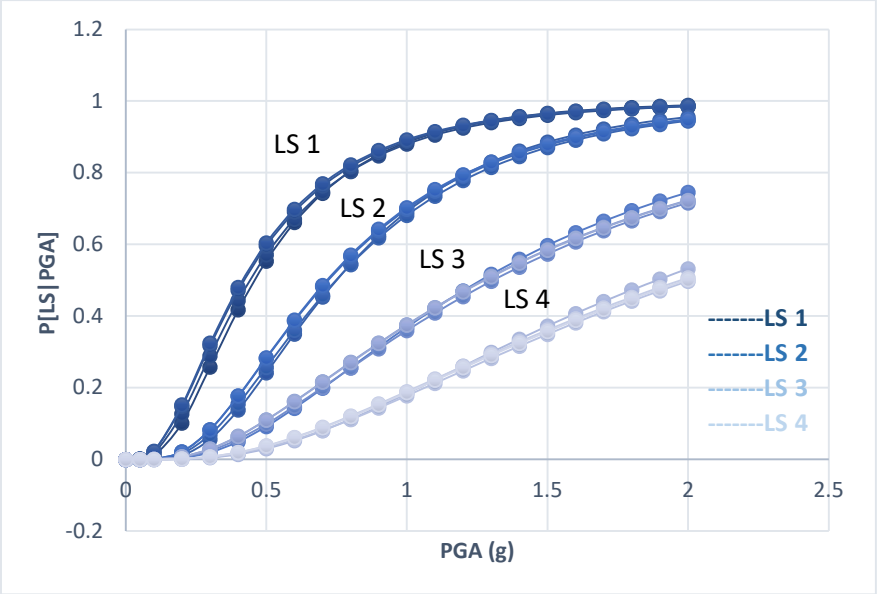


Figure 4-9: fragility curve comparison for all damage limit states

The fragility curve comparison of the bridge column has also been performed for all damage states, as shown in Figure 4-9. Increasing the bridge column concrete compressive strength decreases the bridge structure vulnerability for a slight damage limit state; the bridge column with the highest compressive strength of concrete is 1.4% (maximum) less vulnerable than the bridge column with the lowest compressive strength of concrete. For LS2 to LS4, the bridge column with the highest compressive strength of concrete is 3.935%, 5.886%, 5.3635% (maximum), respectively, less vulnerable than the bridge column with the lowest compressive strength of concrete. From this comparison, we can conclude that observations from Figure 4-9 (for all four damage states) reveal how varying the concrete compressive strength doesn't affect the vulnerability of the bridge.

4.8.2. Fragility analysis for Reinforcement steel strength

Table 4-6 demonstrates the comparison of the median and dispersion values for bridges with various reinforcement steel strengths. The median PGA value increases with increasing the reinforcement steel strength. As an illustration, when the reinforcement steel strength increases from 460 to 500 MPa, the median value for LS1 increases from 0.911 to 0.9524g (4.544%). And the median PGA decreases from 0.911 to 0.836 (8.232%) when the reinforcement steel strength decreases from 460 MPa to 420 MPa for LS1.

Table 4-6: Median for yield strength of reinforcement steel

Steel yield strength	420 MPa	460 MPa	500 MPa
	λ	λ	λ
LS1	0.836	0.911	0.9524
LS2	1.2	1.4135	1.6331
LS3	1.76	2.2515	2.8925
LS4	2.364	3.2225	4.4927

A comparison of the fragility curves in different damage states has been performed. Figure 4-10 to 4-13 depicts the system fragility curves for all yield strength of reinforcement steel.

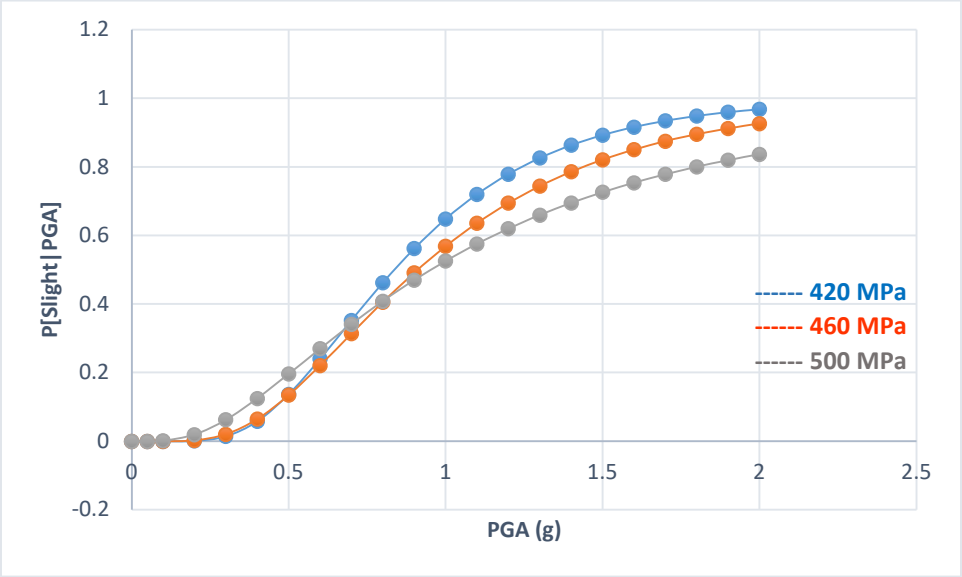


Figure 4-10: fragility curve comparison for slight limit state (LS1)

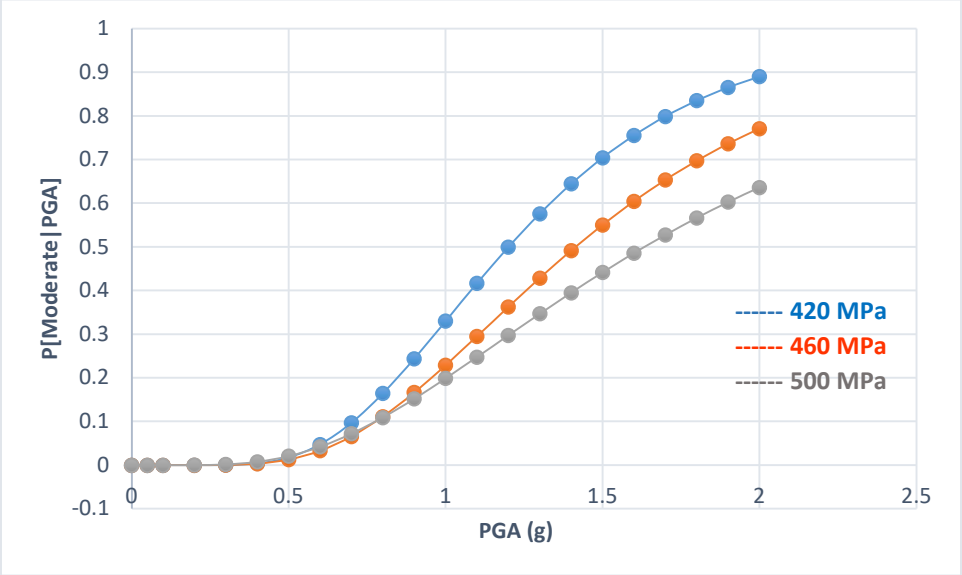


Figure 4-11: fragility curve for moderate limit state (LS2)

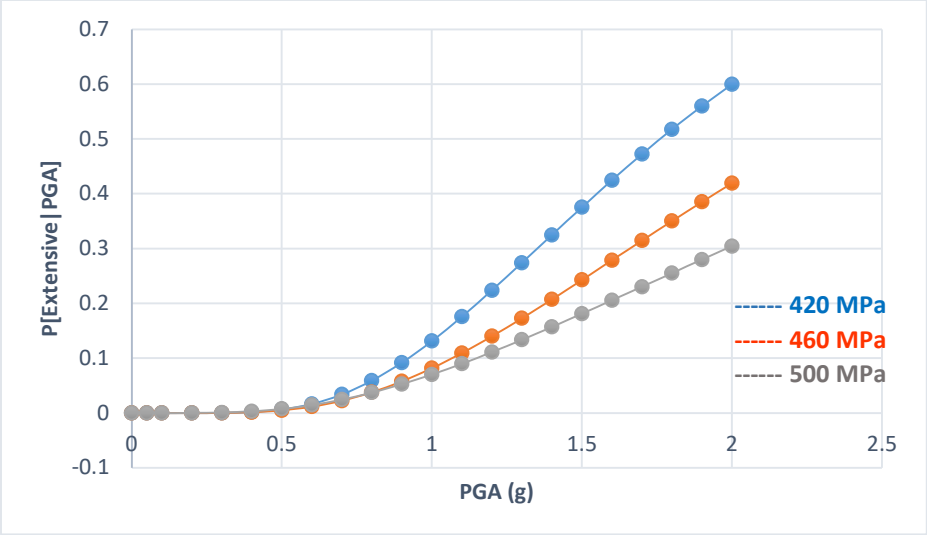


Figure 4-12: fragility curve comparison for Extensive limit state (LS3)

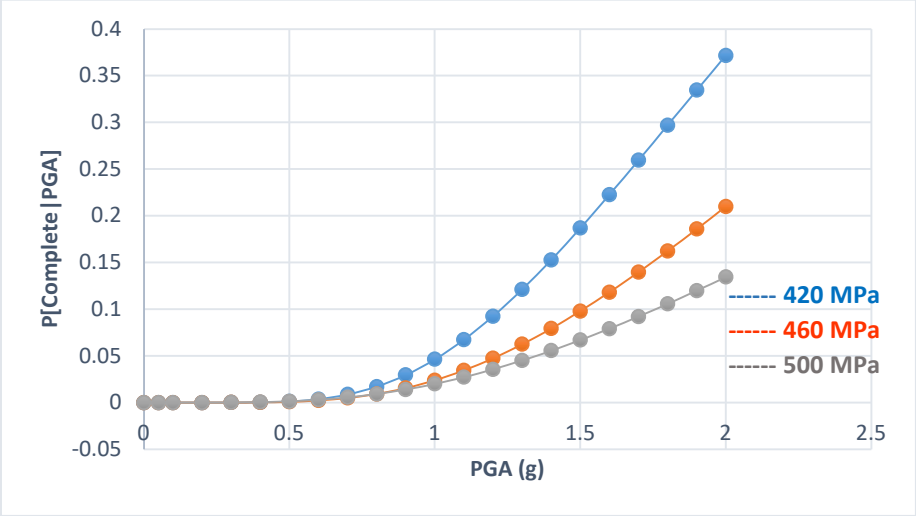


Figure 4-13: fragility curve for complete limit state (LS4)

As we can see from the fragility curve above for the complete limit state (LS4), the 460 MPa reinforcement steel has a lower collapse probability (4.743%) at a PGA value of 1.2 g comparing to the PGA value of 1.2g (9.25%) of 420 MPa reinforcement steel. The 500 MPa reinforcement steel has a lower collapse probability (7.9147%) at a PGA value of 1.6g comparing to the PGA value of 1.6 g (11.8235%) of the 460 MPa bridge pier. Observations from Figure 4-10 to 4-13 (for all four damage states) and median value comparison for all limit states reveal the probability of getting damage decreases with the increasing reinforcement steel strength.

4.8.3. Fragility analysis for bridge slab deck thickness

This section presents the impact of bridge slab deck thickness (superstructure mass) on the prestressed I girder bridge. As we can see from figure 4-14, a comparison of the fragility curves in different damage states has been performed.

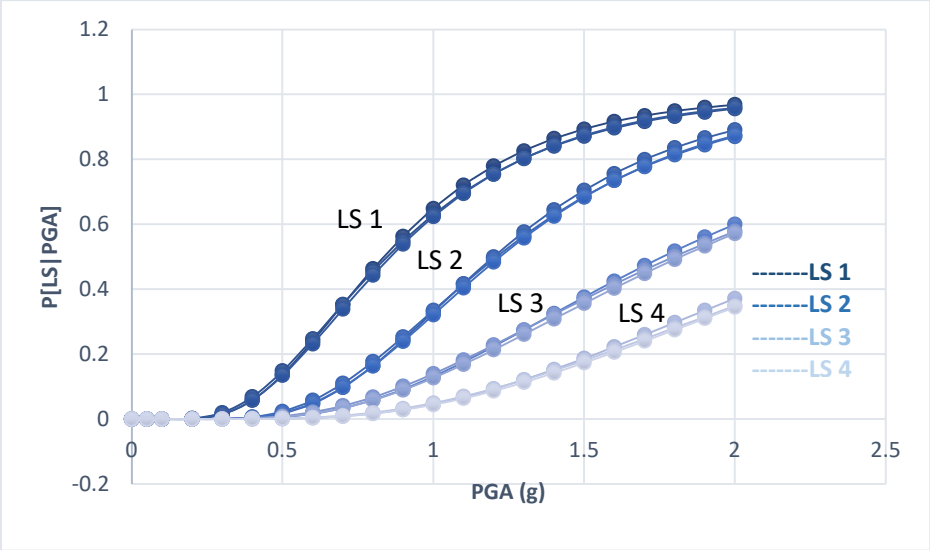


Figure 4-14: Fragility curve comparison for all damage limit states

The results show that bridge deck slab thickness has not a significant impact or does not have a particular effect on the seismic vulnerability of the bridges. There may be a slight reduction in the vulnerability of the bridge as the bridge deck slab thickness (superstructure mass) increases. But the effect on the seismic performance of the bridge is negligible as the change in fragility is very small, as we can see from the graph.

4.8.4.Fragility analysis for Skew angle

In this study, four skew angles of the bridge are selected to investigate their effect on seismic fragilities of the bridge. The skew angles are 0º, 20º, 30º, and 45º. Before conducting the fragility analysis, the impact of the skew angle on the seismic demand of the bridge columns is investigated through the comparison of median PGA values of the bridges with different skew angles, as shown in Table 4-7.

Table 4-7: Median for Skew angle

Median	0º	20º	30º	45º
	λ	λ	λ	λ
LS1	0.8438	0.836	0.8146	0.79
LS2	1.33	1.2	1.1496	1.0898
LS3	2.1546	1.76	1.6564	1.5316
LS4	3.12416	2.364	2.1943	1.99

Table 4-7 compares the median values for bridges with various skew angles. The median fragility value decreases with increasing the skew angle. For LS1 and LS2, when the skew angle increases from 0 to 45°, the median PGA value decreases from 0.8438g to 0.79g (6.376%) and the median PGA value decreases from 1.33g to 1.0898g (18.06%), respectively. And the median for LS3 decreases from 2.1546 to 1.5316g (28.915%) when the skew angle increases from 0^0 to 45º. Figure 4-15 to Figure 4-17 depicts the fragility curves for the skew angle of LS1, LS2, and LS3.

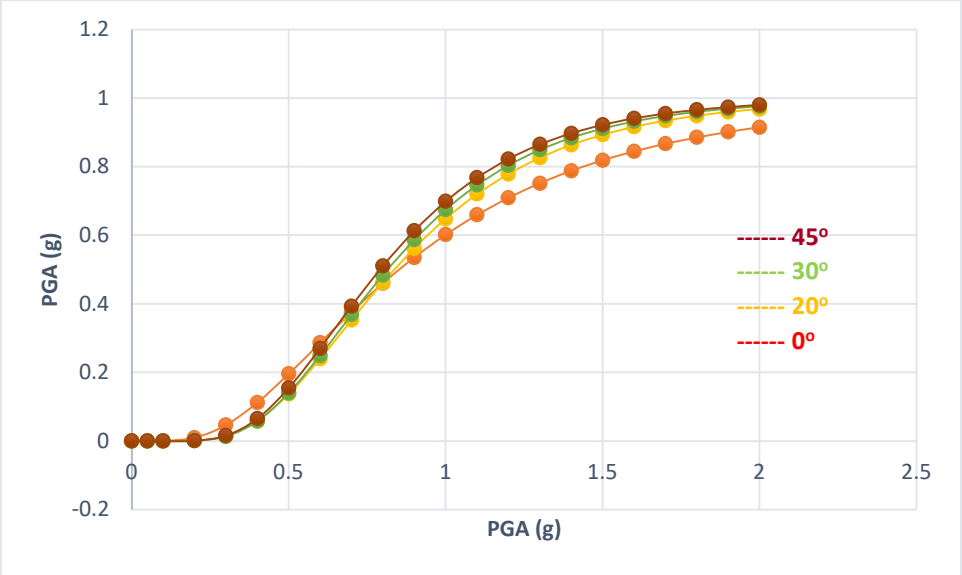


Figure 4-15: fragility curve comparison for slight limit state (LS1)

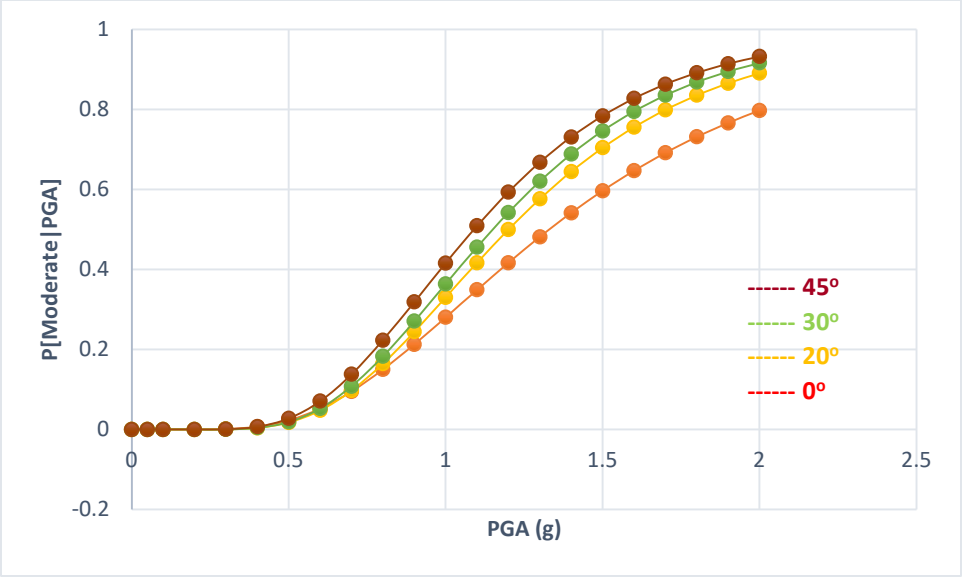


Figure 4-16: fragility curve comparison for moderate limit state (LS2)

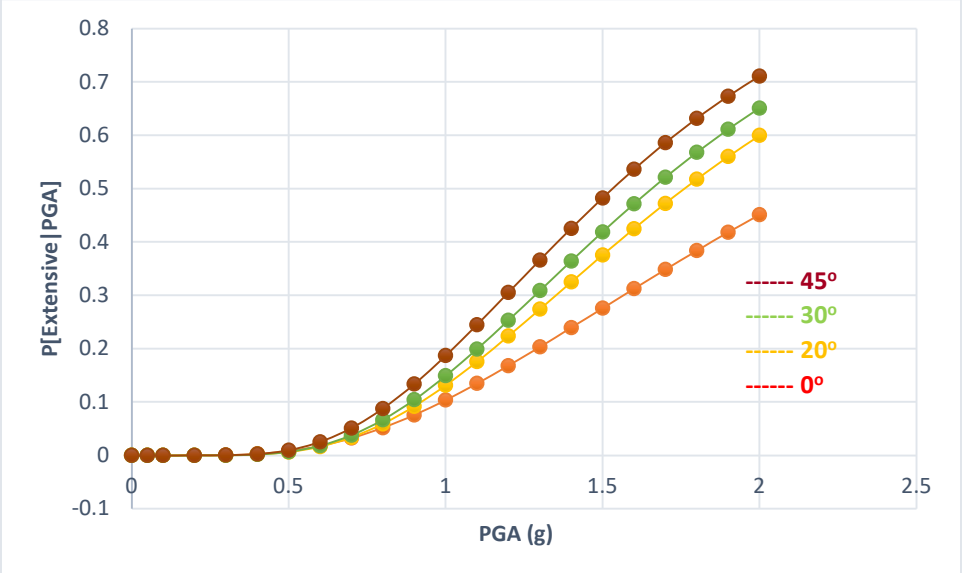


Figure 4-17: fragility curve comparison for extensive limit state (LS3)

The fragility curves for the slight to extensive damage states for 0º, 20º, 30º, and 45º degree skews shown in Figure 4-15 to 4-17. For LS1 (slight damage state), the damage exceedance probability at 0.8 g PGA increased from 45.998% for 0º, to 46.259%, 48.398%, and 51.046% for 20º, 30º, and 45º respectively.

For LS3 (extensive damage state), the same trend as LS1, the damage exceedance probability at 1.2 g PGA increased from 16.818%, for 0º, to 22.387%, 25.326%, and 30.541% for 20º, 30º, and 45º, respectively. From these comparisons, an increase in skew angle makes the bridge more vulnerable, or the probability of getting damage increases with the increasing skew angle.

5. Conclusions and Recommendations

5.1. Conclusion

This thesis studies the effect of geometric and material parameters on the seismic performance of prestressed I girder bridges. The parameters are reinforcement steel yield strength, the compressive strength of concrete, bridge deck slab thickness (superstructure mass), and skew angle of the bridge. The prestressed I girder bridge has two spans, two integral abutments, and a skew angle of 20^{o}. For this bridge, comprehensive nonlinear numerical models are developed and simulated in CSiBridge 20 software for nonlinear time history analysis (NTHA). Then the effects of each parameter on the seismic performance of the bridge are investigated through probabilistic approaches (using fragility curve).

For the probabilistic analysis approach, the fragility analysis is used to assess the effects of the uncertainty of the modeling parameters on the bridge seismic performance. The impacts of each parameter (including various levels of each parameter) on the fragilities are investigated separately by conducting a set of time history analysis of the bridge models to develop fragility curves. Fifteen ground motion records from the earthquakes around the world are utilized in the NTHA. These bridge structures are modeled by accounting for geometrical and material uncertainties to generate the fragility curves of each parameter (for all limit states) via a fragility analysis approach. Based on this parametric study, the following conclusions are made for the prestressed I girder bridges.

- Concrete compressive strength: the concrete compressive strength has a minimum impact on the bridge column vulnerability (seismic performance of the bridge) or the compressive strength of concrete does not have a significant effect on the column fragility.

- Reinforcement steel yield strength: Increasing the reinforcement steel yield strength decreases the column vulnerability for LS1, LS2, LS3, and LS4. In general, we can conclude that from the median and fragility curve comparison for all limit states reveal the probability of getting damage decreases with the increasing reinforcement steel strength.

- Bridge deck slab thickness (Superstructure mass): Results from the median and fragility curve comparison show that varying the bridge deck slab thickness (bridge superstructure) doesn’t affect the vulnerability of the bridge.

- Skew angle: From median and fragility curve comparison for all limit states reveal that the probability of getting damage increases with the increasing the skew angle of the bridge.

In general, from fragility analysis, we can conclude that the yield strength of reinforcement steel had the highest effect on the performance of the bridge. After which the skew angle of the bridge showed a significant impact on the seismic performance of the bridge. Again, the concrete compressive strength and bridge slab deck thickness (superstructure mass) of the bridge had the least effect on the performance of the bridge, with the smallest difference between the original bridge results and the maximum difference.

5.2. Recommendations for further work

Additional research should be done in future work to understand the effect of other parameters and to study the bridge behavior under seismic loads. Some of the areas that should be covered are stated in the following section.

- The fragility of the bridge structures should be further investigated for the effect of different bridge span number. Because this thesis only studies two-span bridges.
- Various types of bridges other than prestressed I girder bridges should be studied for different parameters. Bridges like T girder bridges, box girder bridges, etc., and also steel bridges should be investigated to realize the uncertainty effect of modeling parameters on seismic performance of the bridge.
- Bridge irregularities, such as bridges with different column height, bridges with different cross-sections, and curved bridges, should be considered in the future work of seismic performance analysis of the bridge structure.
- Further studies should be considered on the effect of retrofitting measurements for different types of bridge and should be investigated their effectiveness.

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