

TIME-DEPENDENT SEISMIC FRAGILITY ASSESSMENT OF AGING ORDINARY HIGHWAY BRIDGES SUBJECT TO FLOOD-INDUCED SCOUR

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*“The impediment to action advances action. What stands in the way
becomes the way.”*

Marcus Aurelius



ABSTRACT

Time-dependent seismic vulnerability of bridges is a topic that attracts significant research interest in recent years. This issue has begun to surface itself recently in the United States and Europe, where most of the concrete bridges are older than 70 years of age by today. Seismic performance of bridges may reduce in time due to several aging mechanisms such as chloride-induced corrosion in reinforced concrete members. Because of that, damage and loss estimations which are traditionally being conducted upon the pristine state of a structure may yield inaccurate results regarding future earthquake scenarios. Furthermore, performance of river-crossing bridges may be affected from scouring at pier foundations after a flood event due to the removal of soil around bridge piers. Unexpected increase in expected damage due to such mechanisms may lead to an increased risk for bridge infrastructure systems under the multi-hazard of earthquakes and floods. This study aims to present a stochastic framework to evaluate the time-dependent seismic fragility of highway bridges subject to flood-induced local scour.

In this thesis study, firstly, two different approaches for the development of seismic fragility curves are compared for their applicability and feasibility. Following that, possible aging mechanisms in an ordinary highway bridge are depicted and three-dimensional finite element models of the study bridge for different ages are generated incorporating various levels of scour to develop the time-dependent seismic fragility curves of bridges. Then, a ground motion data set representing the general seismicity of Turkey is assembled. For the combinations of all considered scour depths and bridge ages, nonlinear time-history analyses are performed to obtain the seismic demands. After a binary damage assessment, Method of Maximum Likelihood is employed to estimate the

fragility parameters. Finally, seismic fragility surfaces are developed for different ages of the bridge. The outcomes of this study can be utilized in future efforts towards calculating the lifetime risk of ordinary highway bridges under the multi-hazard of earthquakes and floods.



ÖZETÇE

Köprülerin zamana bağlı deprem kırılganlığı, son zamanlarda ilgi çekici bir araştırma alanı haline gelmiştir. Bu sorun, çoğu altyapı sisteminin 70’li yaşlarda olduğu Birleşik Devletler’de ve Avrupa ülkelerinde kendini göstermeye başlamıştır. Köprülerin deprem performansında, bazı yaşlanma mekanizmalarına binaen zamana bağlı bir düşüş meydana gelir. Köprünün bozulmamış hali ele alınıp yapılan incelemelerde, diğer bir deyişle klasik metodun izlendiği ve yaşlanma etkilerinin göz önüne alınmadığı deprem kaybı hesaplarında isabetsiz sonuçlara ulaşılabilir. Bunun yanında, nehir geçen karayolu köprülerinde, sel afetine bağlı olarak kolon diplerinde bir zemin kaybı yaşanır, ve bu tarz yerel oyulmaların sonucunda deprem performansında bir değişim gözlenir. Yaşlanma ve oyulmaya bağlı olarak hasar seviyelerindeki beklenmedik artışlar deprem riskinin artmasına sebep olur. Bu çalışma, sele bağlı oyulmaya maruz kalan karayolu köprülerinin zamana bağlı deprem kırılganlıklarını hesaplamak için olasılıksal bir çerçeve tanımlamayı amaçlamaktadır.

İlk olarak, çalışmada kullanılacak metodu belirlemek amacıyla, kırılganlık hesaplamalarında sıkça kullanılan iki metodun uygulanabilirlikleri üzerinde bir karşılaştırma çalışması yapılmıştır. Sonrasında, sıradan karayolu köprülerinin maruz kalabileceği bozulma etkileri belirlenmiştir ve çalışma köprüsünün üç boyutlu sonlu eleman modelleri birçok oyulma ve yaşlanma seviyesini göz önüne alınarak oluşturulmuştur. Bunun ardından, Türkiye’nin genel depremsel özelliklerini yansıtacak bir yer hareketi seti oluşturulmuştur. Çeşitli oyulma ve yaşlanma seviyeleri için deprem taleplerini elde edebilmek için doğrusal olmayan zaman-tanım alanında analizler yapılmıştır. İkili bir hasar değerlendirmesi yapıldıktan sonra, kırılganlık parametrelerini

belirleyebilmek amacıyla Maksimum Olabilirlik Yöntemi'ne başvurulmuştur. Son olarak, köprünün farklı yaşları için sismik kırılgenlık yüzeyleri elde edilmiştir. Bu çalışmanın sonuçları, köprülerin deprem ve sel çoklu tehlikesi altındaki riskini hesaplamak amacıyla yapılacak gelecek çalışmalarda kullanılmaya uygundur.



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TABLE OF CONTENTS

ABSTRACT	iv
ÖZETÇE	vi
ACKNOWLEDGMENTS	viii
LIST OF TABLES	xv
LIST OF FIGURES	xvii
I INTRODUCTION	1
1.1 Motivation	1
1.2 Objectives	4
1.3 Methodology	5
1.4 Organization of the Thesis	6
II LITERATURE REVIEW.....	8
2.1 Previous Studies Regarding the Seismic Vulnerability Assessment of Bridges	8
2.2 Previous Studies on Time-Dependent Fragility Assessment of RC Highway Bridges Incorporating Aging Effects	12
2.2.1 Deterioration at Reinforced Concrete Piers.....	13
2.2.2 Deterioration at Bearings.....	16
2.2.3 Consideration of Multiple Aging Mechanisms	17

2.3	Previous Studies Reliability Assessment of Highway Bridges under Multi-Hazards	18
III EVALUATION OF METHODOLOGIES FOR GENERATING ANALYTICAL SEISMIC FRAGILITY CURVES OF BRIDGES		
21		
3.1	General	21
3.2	Bridge modeling	22
3.3	Ground Motion Data Set	24
3.4	Quantification of Uncertainties	26
3.4.1	Uncertainties in Modeling parameters: structural and geotechnical parameters	27
3.5	Fragility curve methodologies.....	28
3.5.1	Method of Maximum Likelihood (Approach-1)	29
3.5.2	Probabilistic Seismic Demand Model (Approach-2)	30
3.5.3	Confidence Intervals.....	32
3.5.4	Results.....	33
3.6	Evaluation of the Results.....	36
IV TIME-DEPENDENT SEISMIC FRAGILITY ASSESSMENT OF BRIDGES SUBJECT TO FLOOD-INDUCED SCOUR.....		
38		
4.1	Study Bridge.....	39
4.1.1	Review of the Bridge Inventory in Turkey.....	39
4.1.2	Characteristic Highway Bridge	40

4.2	Ground Motion Data Set	42
4.3	Deterioration of Bridge Components	48
4.3.1	Chloride-Induced Corrosion at Reinforced Concrete Piers.....	51
4.3.2	Secondary Effects of Corrosion.....	54
4.3.3	Summary	55
4.4	Bridge Modeling.....	56
4.4.1	Modeling of the Superstructure	57
4.4.2	Modeling of the Columns and Extended Pile Shafts.....	58
4.4.3	Modeling of Abutments, Bearings, Shear Keys	60
4.4.4	Summary.....	65
4.5	Seismic Analysis of the Bridges.....	66
4.5.1	Modal Analysis.....	66
4.5.2	Nonlinear Time-History Analysis of the Bridge Models	69
4.5.3	Seismic Response of the Study Bridge for Extreme Aging and Extreme Scour Case	70
4.6	Probabilistic Framework for Generating Time-Dependent Fragility Curves of Bridges	71
4.6.1	Consideration of Uncertainties	72
4.6.2	Seismic Intensity Measure.....	74
4.6.3	Component Limit States and Definition of System Fragility ..	75

4.7	Time-Dependent Seismic Fragility Curves of Bridges subject to Flood-Induced Scour	78
4.7.1	Fragility of Piers	78
4.7.2	Fragility of Bearings.....	80
4.7.3	Fragility of Abutments.....	82
4.7.4	System Fragility Curves for Different Aging and Scour Levels	84
4.7.5	Fragility Surfaces.....	86
V CONCLUSIONS.....		92
5.1	Summary of the Thesis.....	92
5.2	Major Outcomes	94
5.3	Recommendations for Future Studies	96
APPENDIX A - DETAILS OF THE GROUND MOTION DATA SET DEVELOPED FOR EVALUATION OF FRAGILITY METHODOLOGIES		97
APPENDIX B - SENSITIVITY STUDY FOR THE MODELING PARAMETERS PERTINENT TO ORDINARY HIGHWAY BRIDGES.		107
APPENDIX C - BRIDGE SAMPLES AND CONFIDENCE INTERVALS		116

APPENDIX D - DETAILS OF THE GROUND MOTION DATA SET DEVELOPED TO OBTAIN THE TIME-DEPENDENT FRAGILITY CURVES FOR ORDINARY HIGHWAY BRIDGES	118
APPENDIX E - FRAGILITY ANALYSIS RESULTS FOR THE ORDINARY AGING HIGHWAY BRIDGES SUBJECT TO FLOOD-INDUCED SCOUR	122
BIBLIOGRAPHY	126



LIST OF TABLES

Table 3-1: Uncertain parameters that are to be considered in fragility curve comparison study.....	28
Table 3-2: Fragility parameter results for all approaches.	35
Table 4-1: Analysis numbers and descriptions.	39
Table 4-2: Values of the parameters taken for corrosion initiation estimation.	52
Table 4-3: Modeling parameter values for different ages of the bridge.	56
Table 4-4: Modal periods for all considered cases.	68
Table 4-5: Threshold limits for the bridge components.	77
Table A-1: Ground motion properties for the set developed for Chapter 3.....	97
Table B-1: Uncertain parameters compiled from literature.....	107
Table B-2: Input values for the sensitivity study.....	111
Table B-3: Details of the ground motion that is used for sensitivity analysis. Obtained from NGA-West2 database.....	112
Table C-1: Samples generated for the confidence interval study in Chapter 3.	116
Table C-2: Fragility curve median values for the confidence interval study.....	117
Table D-1: Ground motion properties developed for generating the time-dependent multi-hazard fragility curves.	118

Table E-1: Fragility median values for piers.	122
Table E-2: Fragility median values for bearings (in longitudinal dir.).	123
Table E-3: Fragility median values for abutments, for both longitudinal and transverse directions.....	124
Table E-4: Fragility median values for bridge system level.	125



LIST OF FIGURES

Figure 1-1: Deteriorated highway bridge columns (a) Yıldırım and Sümer [3] (b) ElSafty et al. [4].....	2
Figure 1-2: A case of scour in the foundation of Ryde Bridge in Sydney, Australia [7].	3
Figure 1-3: (a) Seismic hazard map of Turkey [8] and (b) Known flood events between 1955 and 2008 in Turkey [9].	4
Figure 1-4: Flowchart for this thesis.....	6
Figure 3-1: (a) Schematic drawing of the bridge pier in transverse direction and (b) fundamental mode shape of the bridge pier.....	23
Figure 3-2: (a) Magnitude, (b) PGA, (c) Arias Intensity, (d) shear wave velocity and (e) rupture distance details of the ground motion set that is employed for Chapter 3.	25
Figure 3-3: Acceleration response spectra of the records used in Chapter 3.....	26
Figure 3-4: PSDM results (Approach-2).	33
Figure 3-5: Fragility curves obtained from Approach-1 and Approach-2.....	34
Figure 4-1: Geometric details of the study bridge in the longitudinal direction and section details of the box-girder.....	41
Figure 4-2: Conditional mean spectrum and simulated spectra for all considered earthquake scenarios.....	45
Figure 4-3: Response spectra of the selected ground motions for Chapter 4. Scaled ground motions are included.....	46
Figure 4-4: Flowchart for the ground motion selection process via Conditional Mean Spectrum method.....	46

Figure 4-5: Scatter of the features of the ground motion records in the data set developed for Chapter 4.	47
Figure 4-6: Schematical representation of deterioration on the reinforced concrete section.	55
Figure 4-7: Geometric properties of the box-girder.....	58
Figure 4-8: Section details of the columns and extended pile shafts.....	59
Figure 4-9: Validation of modeling fiber sections.....	60
Figure 4-10: Backbone curve of the abutment in longitudinal direction for seat-type abutments (elasticPPGap).	61
Figure 4-11: Backbone curve for the abutment response in transverse direction (elasticPP).	62
Figure 4-12: Backbone curve for the response of the elastomeric bearing systems (elasticPP).	64
Figure 4-13: Backbone curve for the response in shear keys (Steel01).	65
Figure 4-14: Modeling details of the study bridge.	65
Figure 4-15: Mode shapes for the (a) longitudinal, (b) transverse and (c) torsional modes of the study bridge.	67
Figure 4-16: Flowchart for the NLTHA.	70
Figure 4-17: Column top displacement response history for a ground motion record. ..	71
Figure 4-18: PSDMs for (a) PGA (in g) and (b) $S_a(T=1s)$ (in g).....	75
Figure 4-19: Fragility curves of piers for different ages of the bridge (for scour depth = 0 m).	79
Figure 4-20: Fragility curves of piers for different scour depths (when bridge age = 0-yr)	80

Figure 4-21: Fragility curves of bearings for different ages of the bridge (for scour depth = 0 m).....	81
Figure 4-22: Fragility curves of bearings for different scour depths (when bridge age = 0-yr).....	82
Figure 4-23: Fragility curves of the abutments in longitudinal direction for different ages of the bridge (for scour depth = 0 m).....	83
Figure 4-24: Fragility curves of the abutments in transverse direction for different ages of the bridge (for scour depth = 0 m).	83
Figure 4-25: Fragility curves of the abutments in longitudinal direction for different scour depths (when bridge age = 0-yr).	84
Figure 4-26: Fragility curves of the abutments in transverse direction for different scour depths (when bridge age = 0-yr).	84
Figure 4-27: System fragility curves for different ages of the bridge (for scour depth = 0 m).	85
Figure 4-28: System fragility for different scour depths (when bridge age = 0-yr).	86
Figure 4-29: Multi-hazard fragility surfaces for the pristine (0-yr) state of the bridge. .	87
Figure 4-30: Multi-hazard fragility surfaces for the 25-yr old bridge.	88
Figure 4-31: Multi-hazard fragility surfaces for the 50-yr old bridge.	89
Figure 4-32: Multi-hazard fragility surfaces for the 75-yr old bridge.	90
Figure 4-33: Multi-hazard fragility surfaces for the 100-yr old bridge.	91
 Figure B-1: Probability distributions for (a) corrosion initiation time (lognormal) and (b) scour depth (normal).....	 110
Figure B-2: Tornado diagrams for (a) No Aging / No Scour, (b) Aging / No Scour, (c) No Aging / Scour and (d) Aging / Scour cases.....	115

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CHAPTER I

INTRODUCTION

1.1 Motivation

Past earthquake events have demonstrated that highway bridges are the most vulnerable elements in highway networks. 1.7 and 6.5 billion dollars have been spent to repair the damaged bridges in 1989 Loma Prieta and 1995 Kobe alone, respectively [1]. Observing the high figures in costs and studying the consequences, government agencies and decision makers urged upon the importance of quantifying the seismic risk, which prompted the research community to investigate the damage characteristics and the factors that would influence the seismic vulnerability of bridges. This thesis aims to build upon the scientific knowledge that have been accumulated thus far in this area by investigating the effects of aging and flood-induced scour on the time-dependent seismic fragility of ordinary highway bridges.

20th century have been a witness to one of the biggest infrastructure investments in history due to the economic boom and the improvements in concrete technology. Turkey is no exception, as 63% of the overall bridges are constructed before 2002 and vast majority of them are mainly built with reinforced concrete (RC) [2]. However, it is known that the concrete is a material that is a subject to various environmental degradation mechanisms (see Figure 1-1), thus there can be an alteration in the structure performance. Because of that, seismic vulnerability and the associated risk of the highway bridges increase over time. Damage estimations which have been traditionally conducted

on the pristine state of a structure up to the considered time instant may yield unconservative results regarding future earthquake scenarios. Assessing the vulnerability for certain ages while implementing the corresponding deterioration to the bridge models would draw a clearer picture about the lifetime seismic risk.



(a)



(b)

Figure 1-1: Deteriorated highway bridge columns (a) Yıldırım and Sümer [3] (b) ElSafty et al. [4]

Seismic performance and risk may also be affected by scouring, which is induced during a flood event in river-crossing bridges due to the removal of soils around bridge piers [5], as illustrated in Figure 1-2. Lateral stiffness and accordingly the dynamic characteristics of a bridge can be heavily influenced with the decrease in soil-embedded foundation (pile) length. Even though scour holes may heal over time as a result of the material brought by the river flows, the probability of observing a maximum flood event

and a successive earthquake cannot be overruled in regions subject to multiple hazards. For example, an earthquake with a magnitude of 4.5 occurred in the state of Washington in 2009, just three weeks after a major flood event [6]. Also, a certain portion of the scour depth occurred in a past flood event may still exist when an earthquake hits the bridge, and this condition necessitates a detailed performance assessment of bridges for varying scour depths at pier foundations.



Figure 1-2: A case of scour in the foundation of Ryde Bridge in Sydney, Australia [7].

Figure 1-3(a) and Figure 1-3(b) present respectively the seismic hazard map and the schematic representation of known flood events between 1955 and 2008 in Turkey. It can be observed from these maps that there is a vast area of conjunction in the “red zones”, hence the river-crossing bridges located in these regions are under a certain risk of experiencing multiple hazard conditions. Oversight on the shift of seismic behavior and increase in earthquake damage due to aging and scouring might weigh heavily on Turkish taxpayers, as allocated resources might not be sufficient to deflect the unexpected increase in the socio-economic toll of an earthquake scenario. Hence, the motivation of

this thesis is to evaluate the combined effects of aging and scouring on the seismic vulnerability of the ordinary highway bridges.

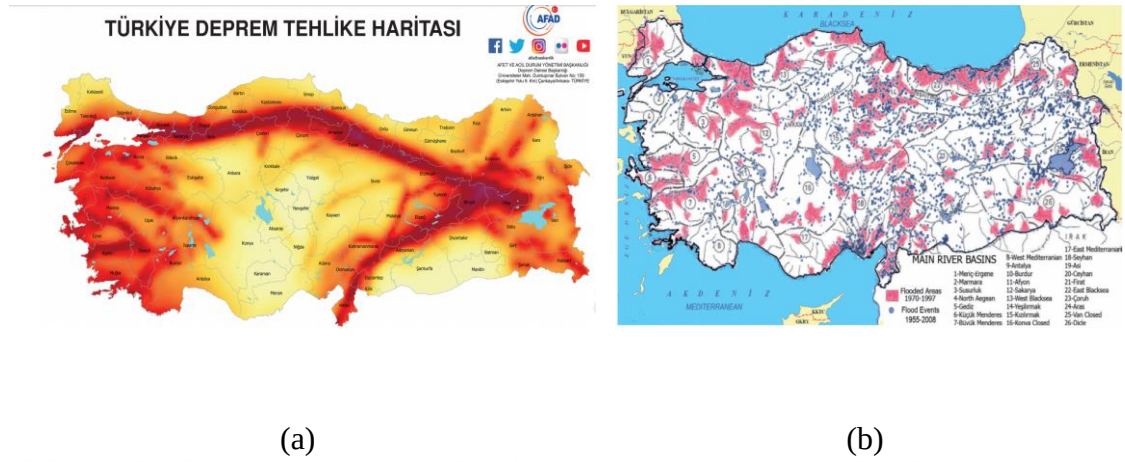


Figure 1-3: (a) Seismic hazard map of Turkey [8] and (b) Known flood events between 1955 and 2008 in Turkey [9].

1.2 Objectives

The main goal of this study is to assess the time-dependent seismic vulnerability of ordinary river-crossing highway bridges, considering the deterioration in the reinforced concrete piers and the scouring in riverbed caused by flood events. Up to this point, research has revolved around the singular effects of these phenomena, and the aging effects are not yet sufficiently covered in studies regarding multiple hazard condition of earthquake and flood-induced scour. Thus, there is still a need for a comprehensive multiple hazard risk framework which would consider the subsequent occurrence of earthquake, flood-induced scour and aging. This thesis aims to provide explanations and insights into these issues with the following objectives:

1. Generate the time-dependent seismic fragility curves for various pairs of bridge age and scour depth. These curves can be employed in future risk assessment

studies for aging bridges subject to the multiple hazard condition of earthquake and flood-induced scour.

2. Evaluate the current approaches used in estimation of fragility parameters and by doing so, determine the optimal methodology to be applied for generating the seismic fragility curves in this thesis.

1.3 Methodology

The general methodology that is followed in this study is presented in the flowchart in Figure 1-4. In the first part of the study, the relative importance of different aging mechanisms is evaluated from a preliminary seismic fragility analysis on a bridge, and the aging mechanisms that is to be considered in this thesis are determined. In the second part, different methodologies in determining the fragility parameters are compared for their conservativeness and applicability. Results of this sub-study is used to depict the reliability approach that is to be employed to generate the fragility curves in this thesis. A distinct third part is allocated to develop a ground motion set that would represent the mid-to-high seismicity of Turkey. In the last part, all of the previous are combined; depicted aging mechanism and its effect are implemented to FE models for different ages of the bridges, seismic analyses are conducted with the ground motion set that was developed, and the seismic fragility curves are generated for various scour depths and ages of the bridge with the chosen fragility curve methodology. These fragility curves are developed by considering five different scour depths (0m, 1m, 2m, 3m, 4m) and five different ages (0-year, 25-year, 50-year, 75-year, 100-year). Finally, the produced fragility curves are also plotted in the form of multi-hazard fragility surfaces for different ages of the bridge.

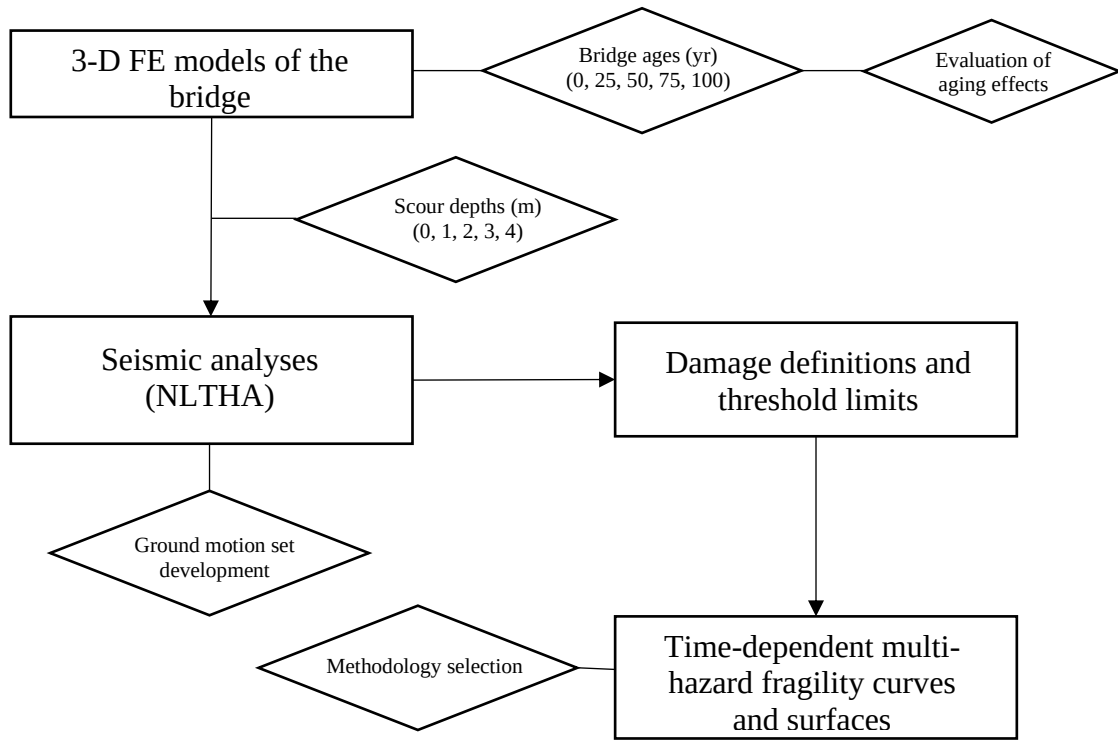


Figure 1-4: Flowchart for this thesis.

1.4 Organization of the Thesis

This thesis is comprised of five chapters.

Chapter 2 presents a literature review about seismic vulnerability analysis of bridges including the considerations of aging and multiple hazard conditions.

Chapter 3 hosts a discussion to make a comparison between two different fragility parameter estimation methods. The sole aim of this chapter is to specify the methodology that will be used in generating the time-dependent fragility curves in Chapter 4.

Chapter 4 presents the framework that has been built to obtain the objectives of this thesis and shows the results of this study in terms of time-dependent multi-hazard fragility curves of bridges. The selected study bridge, the details on bridge modeling, the

considered aging effects and the procedure for constituting the ground motion data set are also discussed in this chapter.

Chapter 5 comments on the key findings of this study. Contributions of this thesis is exhibited and the room for future improvements are argued.



CHAPTER II

LITERATURE REVIEW

In this chapter, the current literature regarding seismic vulnerability of bridges including the studies that considered aging effects and multiple hazard conditions are reviewed. This section is structured into three separate parts: (i) The history of fragility curves is presented, the notion of analytical fragility curves is expressed and the most employed methods are discussed, (ii) The aging mechanisms affecting the seismic performance of bridges are identified, regular research paths are shown and the relevant studies are examined, (iii) Multi-hazard concept is addressed and the studies regarding the joint effect of earthquake and flood-induced scour are surveyed.

2.1 Previous Studies Regarding the Seismic Vulnerability Assessment of Bridges

1994 Northridge and 1995 Kobe earthquakes showed that bridges are one of the most susceptible structures to earthquake damage in a highway network. Socioeconomic losses and the consequential interruptions of service makes it essential to assess the seismic vulnerability of bridges. Over the years, fragility curves are evolved as an efficient tool to quantify the seismic vulnerability and risks associated with highway bridges. Fragility curves demonstrate the probability of exceeding a damage state for a certain level of seismic intensity. It can be represented as a two-parameter lognormal distribution function [10]. Fragility curves can be developed for existing bridges [11] [12] [13] and also to assess the effect of retrofitting on seismic vulnerability of bridges [14] [15].

Various factors were reported to affect the seismic fragility of bridges such as substructure length [16], superstructure length [17], bridge skew angle [18], aging mechanisms [19] [20], soil conditions and foundation type [13].

Fragility curves for bridges were traditionally generated with expert opinion or with the real data pertaining to post-earthquake reconnaissance of researchers. In the latter type, the usage of earthquake performance data can unveil a more realistic evaluation of bridge vulnerability. However, it is not possible to develop empirical fragility curves for the locations where earthquake damage data is unavailable. Even if such information is present, inconsistencies may arise between different researchers due to the subjectivity in evaluation of observed performances. Billah and Alam [21] showed that the empirical fragility curves developed by Yamazaki et al. [22] and Shinozuka et al. [10] for 1995 Kobe earthquake Hanshin Expressway region is very different from each other due to the discrepancies in evaluation. To solve such problems and generate damage data, in the light of recent developments in computer technologies, researchers have started to adopt analytical frameworks and conduct comprehensive seismic simulations [11].

Analytical fragility curve methodology has three components. First, a nominal finite element (FE) model representing the study bridge is needed. The reliability of FE analysis results depend upon the assumptions that are undertaken by the researcher. In that sense, full-fledged three-dimensional FE models would distinguish themselves from less detailed counterparts. However, it is not practical to build comprehensive models except it is needed, since analysis time would exponentially increase with more detailing in the FE model. According to Alam et al. [23], idealized two-dimensional models are shown to yield satisfactory results, given that the bridge is straight in its longitudinal direction. This study further refers to Zhang and Huo [24], where a comparison between a three-dimensional bridge model and a two-dimensional transverse model is drawn.

These models were generated so that the fundamental period of the two-dimensional model matches with the transverse mode period of the three-dimensional model. Results of this study showed that the responses recorded from both cases are in close agreement. Nevertheless, there is a tendency to develop detailed three-dimensional bridge models in literature in order to record the damage in abutments and other secondary components [15] [25]. Secondly, an analysis scheme should be depicted. Seismic events result in nonlinear behavior in several bridge components such as piers, abutments and shear keys. Furthermore, seismic ground motions contain randomness in intensity and direction that may cause different dynamic load effects for two consecutive instances. For this reason, dynamic analysis methods such as nonlinear time-history analysis (NLTHA) scheme is deemed as the most robust and dependable technique in this endeavor [10]. For this method to yield reliable results, an adequate set of ground motions that represents an appropriate seismicity is a prerequisite. However, generally there are not enough distinct ground motions to employ for a specific region. Over the years, different ways have been presented to solve this issue in literature. One solution that researchers suggest is to include ground motions from different regions having similar seismicity in their set. For example, Avsar et al. [18] assembled a ground motion set that represents the seismicity of Turkey to generate the fragility curves for highway bridges. In that study, ground motion records from different strike-slip faulting regions are employed in order to complement the records from the North Anatolian fault line. Also, researchers might employ generated synthetic sets [11] or pre-determined set of real ground motions [26] for the region that they are interested in. One other possible solution might be to search for ground motions that match a target response spectrum. Baker et al. [27] proposes the Conditional Mean Spectrum (CMS) method for ground motion selection procedures. This study was presented as an improvement towards the frequently used Uniform Hazard

Spectrum (UHS) approach. It is argued that ground motions selected from this methodology would be more probabilistically consistent, as it considers the probability of exceeding all spectral values at the same time. Apart from those, this problem might be solved by employing the incremental dynamic analysis (IDA) scheme. This method is originally proposed by Vamvatsikos and Cornell [28], widely used ever since. Basically, it is an approach which some number of ground motions are scaled up to certain point, and each scaled record is used as a distinct ground motion. Precision of this framework is highly dependent on the selected ground motions and depicted scale factors [29]. It is generally used for collapse fragility assessment [21]. Nevertheless, there are some studies where nonlinear static procedures are implemented in literature. Banerjee and Shinozuka [30] compared the fragility curves developed from Capacity Spectrum Method (CSM) and NLTHA, showing that the results are consistent with each other in both cases. As a result, that study proposed the usage of the less computationally expensive CSM as a substitute of NLTHA. Thirdly, a reliability method needs to be depicted to estimate the fragility parameters. Two most common approaches are *Method of Maximum Likelihood* (MML), where the parameters are calculated by an algorithm following a binary damage assessment [10] and *Probabilistic Seismic Demand Model* (PSDM), where the parameters of demand is calculated from normalized results and processed with the capacity parameters to construct the fragility curve [31]. A preliminary study conducted by the author revealed that both of those methods yield similar results in lower limit states. However, they start to deviate from each other, as it goes from lower limit states to higher limit states. MML is observed to yield more conservative results compared to PSDM. That is attributed to the nature of PSDM approach, where a linear line is fit to the normalized results for each unique ground motion. This linearization tends to

underestimate the demands in higher intensity measures, thus yields higher median values. Results are further illustrated in Chapter 3.

2.2 Previous Studies on Time-Dependent Fragility Assessment of RC Highway Bridges Incorporating Aging Effects

In 2003, United States Department of Transportation disclosed that over 27% of the highway bridges in the U.S. were worn out and defective [32]. Such statistics prompted researchers towards quantifying the effects of aging on the seismic performance and vulnerability of highway bridges. In this section, possible aging mechanisms and their adverse effects on seismic vulnerability of bridges are discussed and relevant studies are surveyed. This section is structured into three sub-sections: In the first one, chloride-induced corrosion in reinforced concrete piers are investigated separately, as it is deemed as one of the most influential aging mechanisms. Second one is the review of the aging effects in bearings. Third one is the review of the studies in which multiple aging mechanisms are considered.

A prelude is needed to clarify the orientation of this section. The effects of aging are projected on bridge FE models through the use of deterioration models. As Wan-Wendner [33] points out, even a relatively small mistake in a deterioration model would accumulate into significant miscalculations in estimating the future state of the structure. While there are many studies for each of the below mentioned deterioration mechanisms, to the best of author's knowledge, there is no complete theoretical models to incorporate into FE analyses except for chloride-induced corrosion in piers. Apart from that, the second most prominent aging mechanism is associated with the time-dependent properties of elastomeric and steel bearings. Thus, this section is structured around pier and bearing deterioration. Nevertheless, possible aging effects in RC elements other than

those are further detailed in Section 4.3 for clarification and to present the research opportunities, given that a reliable analytical material degradation model can be found in the future.

A valuable note may be about the retrofitting philosophy regarding the obsolete highway bridges. FHWA [34] basically recommends abstaining from any effort of retrofitting after 50 years of age in a highway bridge. In that manner, AASHTO [35] proposed a framework to evaluate the condition of the highway bridges and to propose maintenance strategies. 50-year mark is further underlined in this document. Following these studies, Ocak and Caner [36] proposed condition factors for the seismic performance of highway bridges. Readers are further referred to this study for valuable insights in this matter.

2.2.1 Deterioration at Reinforced Concrete Piers

Bridge piers are pivotal in resisting the seismic shocks by sustaining large lateral displacements in an earthquake event. Because of that, deterioration at piers is one of the most influential aging mechanisms that negatively affect the seismic performance of reinforced concrete bridges. As established before, the main aging mechanism that governs the vulnerability of RC elements is the chloride-induced corrosion of steel reinforcement. Primarily, corrosion results in area loss in longitudinal and transverse reinforcements in a RC section. In recent years, corrosion with its primary and secondary effects to the bridge elements are issued extensively in literature. Because of its diverse and uncertain nature, lots of research opportunities are present and its effects on seismic vulnerability is still being investigated.

There are several focal points of research around corrosion at RC piers, revolving around the uncertainties related with corrosion. Firstly, there are regional uncertainties. Highways that are under harsh winter conditions may be of subject to deicing salts, mainly

applied by government agencies to prevent accidents related with icing. When melt, they are swept or spattered towards to the substructure of bridges, initiating corrosion. Some of the bridges are directly in contact with chloride ions due to being in a marine environment. In this situation, chloride ions ingress through clear cover by salty water or air. Initiation times and corrosion rates differ for each unique regional exposure condition and location of the bridge. Secondly, there is a debate related with the location of corrosion along the rebar. It is established that chloride-induced corrosion causes local cavities, i.e. pitting corrosion. At specific locations, chloride penetration to local cavities might be from 3 to 16 times more than average penetration along the bar [37]. Recently, Shekhar et al. [20] investigated the change in vulnerability arising from uniform corrosion and pitting corrosion. It is concluded that vulnerability increase in pitting corrosion was more significant compared to uniform corrosion. However, it is hard to predict the location of such localizations in a corrosion scenario with precision and it is challenging to incorporate it into FE models. Because of that, many researchers assumed a uniform corrosion along the rebar [19] [38] [39]. Lastly, there is a question whether the corrosion rate should be assumed constant or not. Thoft-Christensen [40] proposes a uniform average corrosion rate formula, and while this were being employed in studies like Enright and Frangopol [41], Liu and Weyers [42] proposed a formula which depicts corrosion rate as a function of chloride content, temperature, ohmic resistance of the concrete cover and corrosion time is the best fit to their experimental data. Considering the other parameters in their annual average values, it becomes a function of time. Vu and Stewart [43] state that as the iron in reinforcement is gradually replaced by rust, which reduces the diffusion of chloride ions, thus leading to a decreased corrosion rate over time. This study further recommends the time-dependent corrosion rate formula that is suggested by Liu and Weyers [42]. Ghosh and Sood [44] state that uniform area loss

assumption leads to considerable overestimation of seismic vulnerability in bridges, thus it is advised to consider a time-dependent corrosion rate for such studies.

As stated above, primary effect of corrosion is the reinforcing steel area loss, which ultimately leads to increase in vulnerability. There are many studies in literature in which this adversity is implemented. Choe et al. [45] investigated the degraded capacity models in terms of drift and shear capacity for RC columns subject to uniform corrosion due to marine submerged exposure condition, assuming a time-dependent corrosion rate. This study was further expanded to estimate the seismic fragility of California bridges with RC columns in Choe et al. [38]. Ghosh and Sood [44] examined the deteriorated column curvature ductility capacity for different ages of a bridge that is subject to marine exposure conditions with pitting corrosion.

Recently, secondary effects of corrosion are being implemented, as they are observed to be influential towards time-dependent seismic vulnerability. Cracks can develop in clear cover as a by-product of corrosion causing a strength loss in cover concrete fibers of a RC section [46]. Furthermore, since the transverse reinforcement is affected by corrosion, there is a loss in confinement which first and foremost affects the core concrete strength. Such confinement loss can be implemented in the FE models by defining the reduced amount of transverse steel for the calculation of core concrete strength per concrete material model of Mander et al. [47]. Also, as the corrosion progresses, iron material in reinforcement is gradually replaced by rust, thus there is a decrease in load bearing capacity [48]. Considering such effects, Alipour et al. [39] assessed the deteriorated capacity and life-cycle cost of aging bridges. In that study, cover concrete was removed from FE models when crack width reaches a certain level and mechanical loss in reinforcement was considered as suggested in Du et al. [48]. Subsequently, Alipour et al. [49] presented a deterministic methodology to update the

aging parameters as the bridge deteriorates over time, considering a chloride-induced corrosion caused by marine environment. After that, Alipour and Shafei [50] explored the effect of chloride-induced corrosion of piers to the mesoscale and microscale of transportation networks. Panchireddi and Ghosh [51] investigated the cumulative effect of recurrent seismic shocks and marine exposure to chloride-induced corrosion to the overall seismic vulnerability of a case study California RC bridge. The difference of this study is that the time-dependent cover concrete strength is assessed as in Coronelli and Gambarova [52] and as later suggested by Rao et al. [46]. Likewise, Shekhar et al. [20] studied the life-cycle cost of a typical CSUS highway bridge, considering different exposure conditions and aforementioned effects including a comparison between uniform and pitting corrosion.

2.2.2 Deterioration at Bearings

Bearing systems are used in bridges to facilitate the movement of the superstructure. They are considered as one of the main components of highway bridges. Their contribution towards the seismic behavior of a bridge is significant due to the isolation effect they induce between the substructure and superstructure for non-monolithic connections (e.g. at seat-type abutments). Steel and elastomeric pad configurations are two of the most used bearing types in RC highway bridges [32]. Steel bearings are mainly subject to chloride-induced corrosion, which would decrease their ultimate lateral strength [53] and cause a debris accumulation in the system [54]. Mander et al. [55] stated that the coefficient of friction may increase up to 0.12 in bridge steel bearings, which would cause interlocking and significantly change the bearing behavior. Elastomeric pad bearings are subject to thermal oxidation, which increases the lateral stiffness due to the hardening of the material [56]. There are various studies concerning such phenomenon, as listed below.

2.2.3 Consideration of Multiple Aging Mechanisms

Ghosh and Padgett [19] examined the increased fragility due the multiple deterioration of RC bridge piers and steel bearings, considering chloride-induced corrosion on both. In this study, the change in the coefficient of friction of bearings due to rust accumulation per Mander et al. [55] and the degradation in the ultimate lateral strength due to anchor bolt corrosion per Hoeke et al. [53] was considered. Contrary to what is expected, there is a decrease in longitudinal demand of fixed bearings due to rust accumulation in the system, which makes up for the capacity loss caused by anchor bolt corrosion. Rokneddin et al. [57] investigated the network reliability of 515 case study bridges located in South Carolina, considering corrosion in bridge piers and thermal oxidation in elastomeric bearing pads. Along with stiffness increase due thermal oxidation, doweling loss of bearings due corrosion is also considered in that study. The effect of stiffening in bearing pads are found to reduce the seismic vulnerability of bridges. Yılmaz and Aygün [58] investigated the combined effects of chloride-induced corrosion at piers and the stiffening of elastomeric bearings due to thermal oxidation on the seismic vulnerability of a regular highway bridge. It was shown that in the early ages of the bridge, there is a slight decrease in vulnerability at low damage states due to bearing stiffening, whereas towards the end of its service life the increase in vulnerability is well-pronounced due to chloride-induced corrosion. Another conclusion drawn in this study is that deterioration of elastomeric bearings has a negligible impact on seismic fragility of the bridge compared to the chloride-induced corrosion effect at RC piers.

2.3 Previous Studies on Reliability Assessment of Highway Bridges under Multi-Hazards

Bridges are subject to various natural and manmade hazards during their service life such as earthquakes, floods, vessel collisions, winds, hurricanes, truck loads etc. In design, the combined effect of these hazards is accounted through load combinations. If a bridge is designed following a load combination consisting of all possible extreme events, the resulting sections would be impractical and uneconomical, as the chance of observing two or more events at the same time is extremely low [59]. However, in a risk framework, the consequences arising from a case is just as important as its probability of occurrence. Thus, extents of damage caused by certain multiple hazard scenarios should be investigated to quantify the resultant vulnerability and its consequences.

Providing access over a waterway is one of the chief reasons of which a bridge is built. Furthermore, significant number of such bridges are located in seismically active regions, hence, these structures are frequently subjected to both earthquakes and flooding. The statement noted above applies for this case too, the chance of observing an earthquake and a flood event at the same time is slim. However, scour holes in riverbed may occur around bridge foundations collaterally due to a flood event before an earthquake hits the bridge, which might later affect the seismic performance. There are various scour types, and among them, local scour is the most prominent [60]. Local scour is caused by a vortex around bridge piers as a result of flood water flowing around them, which removes soils up to a certain depth [5]. These scour holes are eventually get filled with riverbed material over time, hence the probability of observing a virgin scour hole and an earthquake event is low. However, an unhealed scour hole with maximum depth on a given current discharge is generally considered with the purpose of investigating the severity of consequences. This assumption is widely applied in literature [13] [17] [61] [62] [63].

Decó and Frangopol [61] studied the combined effects of earthquake and scouring along with aging and traffic load on the lifetime risk of a bridge. Although the multi-hazard risk level could be predicted by the proposed framework, the results would be valid in an environment where risks arising from scouring are more dominant compared to seismic risks, because a comprehensive seismic analysis was lacking. This deficiency was addressed in Dong et al. [62]’s study, where the sustainability and vulnerability of a bridge structure was investigated under the combined effects of earthquake, flood-induced scour and aging. In this study, a bridge bent model was used and a single flood scenario was considered. Banerjee and Prasad [64] argued that ignoring the scour’s effect on seismic response leads to significant underestimation of risk and suggested this multiple hazard scenario to be considered in analyses. Foundations with larger shaft diameters were found to limit the effects of scouring on seismic performance. Following this, Prasad and Banerjee [17] investigated the risk characteristics of four highway bridges with different numbers of spans that are exposed to seismicity and flood-induced scour. It was concluded that overall seismic risk increases in a decreasing rate with scour depth, thus, the increase is halted after a certain extent. Furthermore, it was stated that the fragility median values in bridges with high number of spans were observed to be higher compared to the bridges with less number of spans, which ultimately led to the conclusion that longer bridges are more vulnerable to the joint effect of earthquake and flood-induced scouring. Alipour et al. [60] presented modified load and resistance factors to offer a design aid for bridges located in extreme flood and earthquake regions. In this study, joint probability matrices of scour and earthquake were employed, however, the probability of singular hazards were neglected. Wang et al. [63] investigated the seismic performance of highway bridges subjected to earthquake and scouring while considering foundation damage and deck unseating limit states. The results of this study showed that the

foundation damage can be reduced by increasing the stiffness. Furthermore, it was concluded that increasing the foundation depth has a limited effect on seismic performance. Subsequently, Wang et al. [65] proposed scour load and resistance factors for bridges as well. The main improvement compared to Alipour et al. [60]'s study was the addition of the probabilities of earthquake-alone events to the hazard matrix.

Yilmaz et al. [13] presented a multi-hazard risk framework to generate fragility curves and surfaces for two real California bridges considering earthquake and flood-induced scour. This study examined two existing bridges, so reflected the importance of regionality in defining hazards and improved upon the simpler modeling aspect of past studies, hence, introduced a more realistic approach. The results of this study indicated that the bridge with large diameter extended pile shafts can be more beneficial against flood-induced scour effects, compared to its more conventional counterparts (i.e. pile group foundations). Afterwards, Yilmaz et al. [66] investigated the confidence levels of seismic fragility curves considering flood-induced scouring. In that study, the random variables that make the seismic response the most sensitive were demonstrated through two sensitivity analysis methods. These random variables were further employed in an uncertainty analysis where the confidence intervals of the seismic fragility curves were generated to quantify the related uncertainties. It was concluded that uncertainties in bridge risk increase due to the consideration of flooding, while the mean risk is observed to be not affected from different flood levels. Then, Yilmaz and Banerjee [67] presented a spectrum of impact of scouring on the seismic fragility of characteristic bridges with different types of pile foundations. This study recommended the usage of extended pile shaft foundations due to its insensitivity towards local scouring in flood-prone regions. Designers were further suggested to be more careful about superstructure damage in such a scenario.

CHAPTER III

EVALUATION OF METHODOLOGIES FOR GENERATING ANALYTICAL SEISMIC FRAGILITY CURVES OF BRIDGES

Fragility curves can be expressed with two-parameter lognormal distributions that represent the probability of exceeding a damage state under a certain intensity level [10]:

$$F(x_j, c_k, \zeta_k) = \Phi \left[\frac{\ln \left(\frac{x_j}{c_k} \right)}{\zeta_k} \right] \quad (3-1)$$

where the $F()$ denotes the probability of exceeding a damage state k for a bridge component (or bridge system) under the ground motion j with the intensity measure x_j . c_k denotes the lognormal mean (i.e. median), and ζ_k represents the lognormal standard deviation (i.e. dispersion) of the fragility curve at k^{th} damage state respectively.

There are various reliability methods to obtain the “fragility parameters”, i.e. median and dispersion of the lognormal distribution. This chapter is dedicated to review and compare the methods for estimating the fragility parameters. At the end of this sub-study, the methodology that is to be used in Chapter 4 is depicted.

3.1 General

For this chapter, a bridge bent is modeled in OpenSEES [68] to obtain the seismic response in the transverse direction of the bridge. A ground motion set is developed and NLTHA is performed on the bridge models. Results are processed with two different fragility parameter estimation methods. In the first one (*Approach-1*), a model is

developed with expected (mean) values of modeling parameters and subjected to the entire ground motion set. After a binary damage assessment, fragility curve median value is determined by using MML. The dispersion value is assumed as 0.6 for all damage states, following the recommendations of HAZUS [69]. This method is first proposed by Shinozuka et al. [10] and later used widely in several studies dealing with seismic fragility assessment of bridges [13] [64]. In the second method (*Approach-2*), several “nominally identical but statistically significant” bridge models are developed with Latin Hypercube Sampling (LHS) by utilizing the statistical distributions of the modeling parameters. Then, these models are randomly paired with ground motion records. PSDM is developed by normalizing the demand versus seismic intensity measure, as shown in Cornell et al. [31]. Then, a linear regression is conducted on normalized data to obtain the demand regression parameters. Combining the obtained demand parameters and ready-to-use capacity distributions, the basic reliability theory is employed to estimate the fragility median and dispersion for all damage states. This method is also frequently used in literature in several past studies [11] [19] [51].

For both of the abovementioned approaches, fragility parameters are determined and accordingly fragility curves are plotted. In addition to these curves, an uncertainty analysis is further conducted to estimate the confidence levels of the fragility curves by repeating the same procedure applied in *Approach-1* for a number of models created from the parameter sample sets generated via LHS.

3.2 Bridge modeling

Two-dimensional models of a generic bridge bent as shown in Figure 3-1a are generated in FE analysis software OpenSEES [68]. This bent is adopted from a

characteristic bridge that represents the Western US bridges, which was previously generated by Yilmaz and Banerjee [67]. The pier consists of two circular reinforced concrete (RC) columns that extend below ground as pile shafts. The columns are monolithically connected to the bridge girder at the top, thus a rigid link element is used to connect the top joints of both columns. Tributary mass and weight transferred from the adjacent spans of the pier are distributed onto the rigid link nodes. Displacement-based fiber elements are employed to model the nonlinear response of the columns and the extended shafts. RC section definitions are made with assigning the uniaxial materials of *Concrete07* and *Steel02* on concrete and longitudinal steel fibers, respectively. Concrete model of Mander et al. [47] is used to capture the stress-strain relationship of unconfined and confined concrete. The lateral support of soil in horizontal direction is modeled with zero-length *p-y* springs which are assigned with the uniaxial material *pySimple1*. The pier is assumed to be sitting on uniform medium dense sand layer and the properties of the soil springs are determined as recommended in API [70]. The bottom of the piles are taken as fixed, assuming that they are fixed to firm soil at their bottoms. The readers are further referred to Section 4.4 for detailed description of the modeling philosophy. The period of the fundamental mode (as shown in Figure 3-1b) is 0.71s for the model in which all parameters are taken at their mean values.

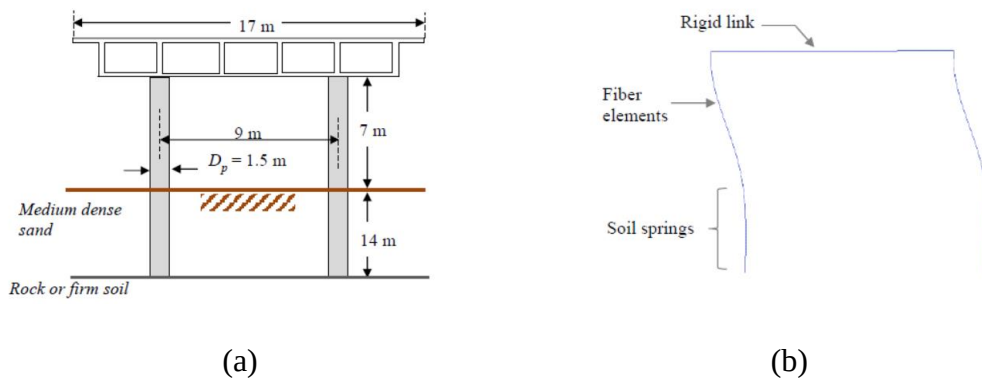
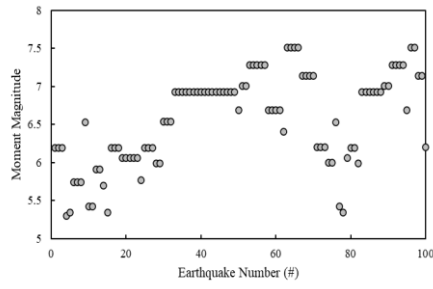


Figure 3-1: (a) Schematic drawing of the bridge pier in transverse direction and (b) fundamental mode shape of the bridge pier.

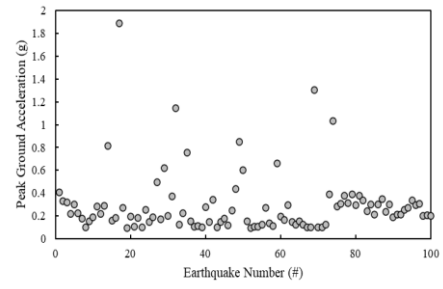
3.3 Ground Motion Data Set

Ground motion set is of paramount importance in seismic vulnerability studies. It should cover a broad range of seismic hazard to yield various demand levels in results. Ground motion set might be selected to reflect the seismicity of a region [18] [26]. However, the aim of this sub-study is to compare fragility methods. Thus, a “generic” ground motion set with moderate-to-high seismicity is assembled. Ground motions are taken from the *NGA-West2* database [71]. A total of 75 unique ground motion records are acquired and 25 of them are scaled with a factor of two to increase the number of ground motions with higher intensities. Since the time-history analyses are conducted only along transverse direction of the bridge, each horizontal component of the available 100 records are employed, resulting in a total of 200 analyses at the end of this procedure. The details of the ground motions that are used for this chapter are presented in Appendix A.

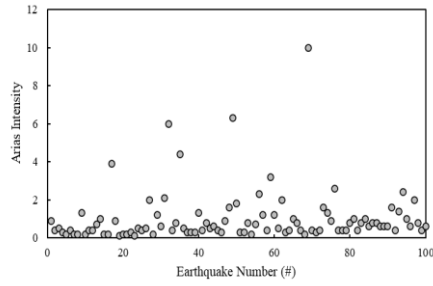
The selected ground motions are recorded from the earthquakes with the magnitudes ranging between 5.0 and 7.5, having epicentral distances up to 150 km and shear wave velocities between 200 and 750 m/s. Shear wave velocity interval is chosen to be in conformity with “medium-dense sand” local soil condition adopted for the study bridge. The physical details of the earthquakes associated with the selected ground motion records are summarized in dot plots as presented in Figure 3-2.



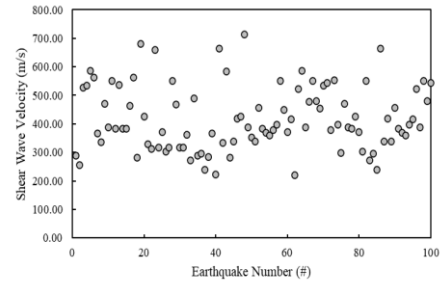
(a)



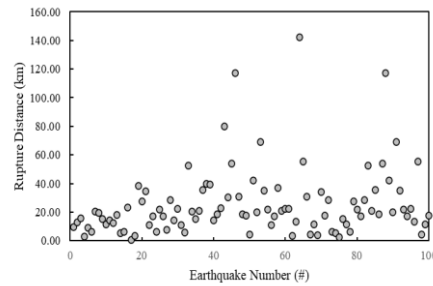
(b)



(c)



(d)



(e)

Figure 3-2: (a) Magnitude, (b) PGA, (c) Arias Intensity, (d) shear wave velocity and (e) rupture distance details of the ground motion set that is employed for Chapter 3.

The acceleration response spectra of all ground motion records in the data set are generated for 5% damping and presented in Figure 3-3. Each horizontal component of a record is treated as a unique ground motion. The plot given in this figure also includes the scaled records.

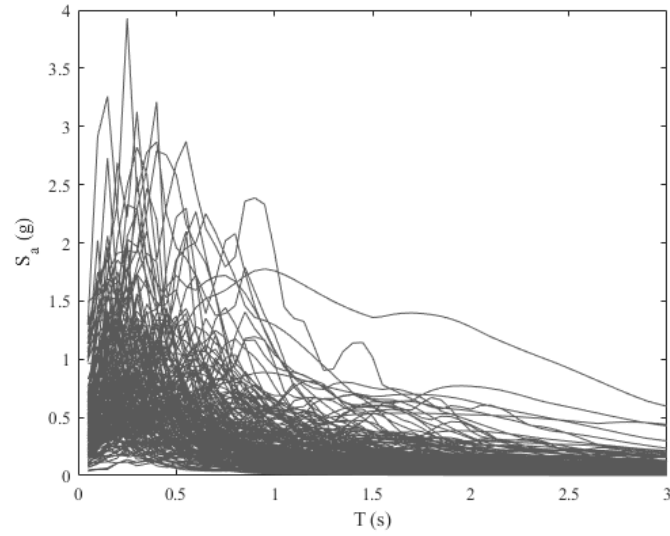


Figure 3-3: Acceleration response spectra of the records used in Chapter 3.

3.4 Quantification of Uncertainties

The methodologies to be evaluated in this study are differentiating from each other in treating the uncertainties related with the bridge models. The first method uses the deterministic (expected) values for all parameters, whereas the second method employs LHS to use samples from the statistical distributions of each considered uncertain parameter. Therefore, it is necessary to list and quantify these uncertainty sources. There are several aleatoric and epistemic uncertainties pertaining to highway bridges that can affect the seismic response. First and foremost, there is an uncertainty about the “modeling parameters”. These include structural parameters such as compressive strength of concrete and yield strength of the steel reinforcement, geometric parameters such as span length and column diameter, and geotechnical parameters such as internal friction angle and unit weight of soil. There is a further uncertainty in the direction of the seismic loading. This sub-chapter is dedicated to address the issue of uncertainties. Please note that geometric uncertainties are not considered in this study, as the objective specified in

this study (i.e. comparison of fragility parameter estimation methods) can be achieved without considering various alternatives of geometric parameters.

3.4.1 Uncertainties in Modeling parameters: structural and geotechnical parameters

For this study, possible uncertain structural and geotechnical parameters are compiled from literature in a holistic manner to account for the modeling uncertainties. The compiled parameters are summarized in Appendix B. However, there are some drawbacks in putting all these 16 parameters into the sampling process all together. First of all, a lot of samples need to be taken to obtain a logical set that would represent the combined distribution of randomness for all input parameters. The number of sample combinations are limited by the extent of our ground motion set, so that rules out this possibility. Furthermore, a large sample set would increase the computational expenses coming from the NLTH analyses exponentially.

In order to solve these problems, a sensitivity study is conducted to reduce the number of uncertain parameters in the sampling process. Tornado diagrams are constructed to observe the “swings” in response with the changes in the parameters, and are shown in Appendix B. The parameters which are found to be insensitive to the seismic response are taken at their expected values. Uncertain modeling parameters that are to be subjected to LHS are summarized below in Table 3-1.

Table 3-1: Uncertain parameters that are to be considered in fragility curve comparison study.

Uncertain parameters		Unit	Dist.	Distribution Parameters		References
Unit mass of concrete	γ_{conc}	t/m ³	Normal	μ : 2.4	δ : 0.05	Lee et al. [72]
Unit weight of wearing surface	γ_{ws}	t/m ³	Normal	μ : 2.2	δ : 0.25	Nowak [73]
Compressive strength of concrete	f_{ce}	MPa	Normal	μ : 32.5	δ : 0.125	Choi [74]
Yield strength of reinforcing steel	f_{ye}	MPa	Lognormal	Λ : 6.16	ζ : 0.08	Ellingwood and Hwang [75]
Longitudinal reinforcement ratio	ρ_{long}	%	Uniform	Lower: 1.0	Upper: 4.0	KGM [76]
Damping ratio	ζ	-	Normal	μ : 0.045	δ : 0.278	Bavirisetty et al. [77]

The directionality of seismic shocks is highly uncertain and a challenge to predict in earthquake engineering. In this study, selected ground motions are applied only in the horizontal direction, as it is the common practice in literature [13] [18], and are assumed to act along the transverse direction of the bridge owing to the analyzed bridge bent model. In the meantime, the variability of occurrence of earthquakes with different intensity and frequency content are already accounted for by the large number of records utilized in the ground motion data set.

3.5 Fragility curve methodologies

In this sub-section, two prominent fragility methods in the literature are introduced. First method (*Approach-1*) employs MML to estimate the median of the fragility curves after a binary damage assessment, per Shinozuka et al. [10]. The second method follows the seminal study of Cornell et al. [31] to estimate the fragility parameters, using the PSDM methodology (*Approach-2*).

For both methods, “maximum curvature ductility demand” (μ_k) is accepted as the engineering demand parameter (EDP) for defining the flexural damage of the pier

(accounting for both the column and the shaft). Threshold limits associated with all the damage states are adopted from the limits defined by Ramanathan [26] for the columns of post-1990 multi-span bridges, as detailed in Section 4.5.3. The limiting median values for curvature ductility are $\mu_k = 1, 4, 8,$ and 12 for minor damage, moderate damage, major (or extensive) damage and collapse state (or complete damage), respectively. For both approaches, the fragility curves of the bridge pier are developed for the seismic intensity measure of spectral acceleration at the period of 1.0 sec, $S_a(T=1.0 \text{ s})$. This is because the corresponding best fit line in the linear regression analysis conducted for PSDM in *Approach-2* yields the greatest R^2 value when $S_a(T=1.0 \text{ s})$ is considered, indicating the best correlation between the intensity measure and the maximum curvature ductility demand. Detailed discussion can be found in 4.6.2 and in Ramanathan [26].

3.5.1 Method of Maximum Likelihood (Approach-1)

In *Approach-1*; a single FE model of the pier (bridge bent) is developed using the expected values of the input parameters listed in Table 3-1. Then, this model is subjected to the complete set of ground motion records, thus $N = 200$ nonlinear time-history analyses are conducted. The resulting structural demands are compared against the median threshold limits of each damage state, and the binary results whether the considered damage state is exceeded or not under each ground motion analysis are processed by using the MML method, which utilizes the likelihood function to obtain the median value for the fragility curve. The likelihood function L is defined as [10]:

$$L = \prod_{j=1}^N [F(x_j; c_k; \zeta_k)]^{r_j} [1 - F(x_j; c_k; \zeta_k)]^{1-r_j} \quad (3-2)$$

in which r_j is the indicator of damage, and F is the fragility function presented in Equation 3-1. r_j takes the value of 1 or 0 depending on whether the damage state k is exceeded or not, respectively. Dispersion value ζ_k is assumed as 0.6 for all damage states

per HAZUS [69]. The median value c_k is determined after an iterative procedure that yields the value of c_k maximizing the likelihood function given in Equation 3-2.

3.5.2 Probabilistic Seismic Demand Model (Approach-2)

Approach-2 is based on the concept of PSDM proposed by Cornell et al. [31]. The procedure below is explained as presented in Ramanathan [26]. PSDM primarily establishes the conditional relationship between seismic demand (D) and seismic intensity measure (IM) in terms of a two-parameter lognormal distribution as:

$$P[D \geq d|IM] = 1 - \Phi\left(\frac{\ln(d) - \ln(S_D)}{\beta_D}\right) \quad (3-3)$$

where S_D is the median demand, β_D is the lognormal standard deviation (i.e. dispersion) of the demand and $\Phi()$ is the standard normal cumulative distribution function. The relationship between the median demand and intensity measure is given as:

$$S_D = a(IM)^b \quad (3-4)$$

where the parameters a and b can be obtained from the linear regression analysis applied on the plotted relationship between the peak seismic demands resulting from the time-history analyses and the corresponding seismic intensity IM , otherwise called the PSDM. In order to conduct the regression analysis, Equation 3-4 is transformed into the lognormal space as:

$$\ln(S_D) = \ln(a) + b \ln(IM) \quad (3-5)$$

where $\ln(a)$ is the vertical intercept and the parameter b is the slope of the best fit line. Dispersion of the seismic demand (β_D) can be obtained from:

$$\beta_D \cong \sqrt{\frac{\sum (\ln(d_j) - \ln(aIM_j^b))^2}{N-2}} \quad (3-6)$$

where d_j represents the resultant peak demand from the j^{th} time history analysis, IM_j is the intensity measure of the j^{th} ground motion record and N is the total number of conducted time history analyses. In this approach, both the demand and the capacity are treated as lognormally varied normal variables, which are described with their median and dispersion values (S_D and β_D for demand and S_C and β_C respectively). Using the classical reliability theory, the fragility curve of a bridge component can be obtained as follows:

$$\begin{aligned} P[D > C | IM] &= \Phi \left(\frac{\ln(S_D) - \ln(S_C)}{\sqrt{(\beta_D)^2 + (\beta_C)^2}} \right) \\ &= \Phi \left(\frac{\ln(a(IM)^b) - \ln(S_C)}{\sqrt{(\beta_D)^2 + (\beta_C)^2}} \right) \\ &= \Phi \left(\frac{\ln(IM) - \left(\frac{\ln(S_C) - \ln(a)}{b} \right)}{\frac{\sqrt{(\beta_D)^2 + (\beta_C)^2}}{b}} \right) \end{aligned} \quad (3-7)$$

The last term of Equation 3-7 is equal to Equation 3-3, so the fragility parameters λ_k and ζ_k can be written as:

$$\lambda_k = e^{\frac{\ln(S_{C,k}) - \ln(a)}{b}} \quad (3-8)$$

$$\zeta_k = \frac{\sqrt{(\beta_D)^2 + (\beta_{C,k})^2}}{b} \quad (3-9)$$

where $S_{C,k}$ and $\beta_{C,k}$ are the median and lognormal standard deviation of the capacity associated with the k^{th} damage state, respectively. The limiting median ($S_{C,k}$)

values for each damage state is taken as considered in *Approach-1*, while the $\beta_{C,k}$ is taken as 0.35 for all damage states as recommended by Ramanathan [26].

In this approach, 200 sample combinations (which is equal to the number of ground motion records in the dataset) of input parameters are generated by using LHS and each structural model with a sample combination is randomly paired with a ground motion record. Using the seismic demands resulting from the nonlinear time-history analysis of each model under the assigned ground motion record, linear regression analysis is conducted to calculate the parameters a , b and β_D as described in Equation 3-5 and Equation 3-6, and then the fragility parameters are determined as presented in Equation 3-8 and Equation 3-9. Prior to each nonlinear time-history analysis, a model analysis is performed to get the Rayleigh damping coefficients under the applied set of input parameters.

3.5.3 Confidence Intervals

To quantify the variability of the seismic fragility curves due to the uncertainties in the modelling parameters, *Approach-1* is repeated under 30 sample combinations of uncertain parameters which are obtained through LHS. Each of the samples are subjected to 200 ground motions and the resulting median values of the fragility curves obtained from each sample combinations are used to develop the confidence bounds (5% and 95% confidence levels) of the fragility curves. In *Approach-2*, such uncertainty quantification is inherently handled owing to the nature of PSDM. As a result of this part, a rigorous comparison would be made between the confidence levels of the fragility curves and the singular fragility curves obtained from both approaches.

3.5.4 Results

The linear regression analysis employed as a part of PSDM (in *Approach-2*) is presented in Figure 3-4.

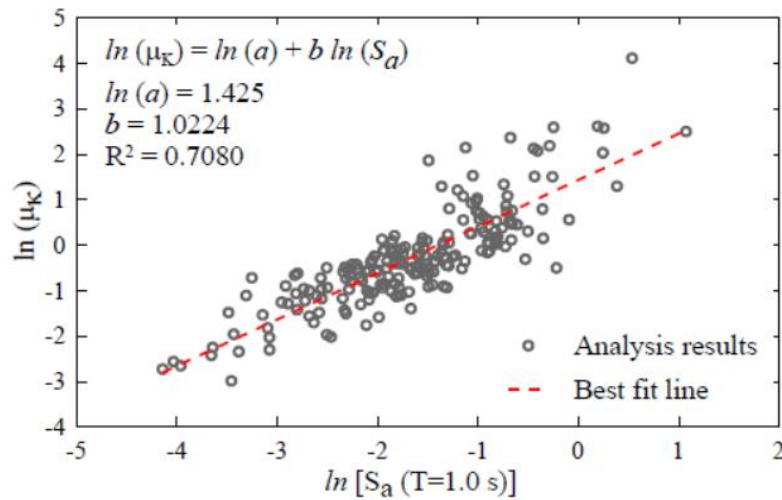


Figure 3-4: PSDM results (*Approach-2*).

The seismic fragility curves developed for each damage state are presented in Figure 3-5 and the corresponding fragility curve parameters are summarized in Table 3-2. Each plot in Figure 3-5 consists of the following curves which are denoted as C1-C4:

C1: Deterministic fragility curve obtained by using *Approach-1*.

C2: Fragility curve obtained by using *Approach-2*.

C3: The set of fragility curves resulting from the uncertainty analysis of 30 random sample combinations by using *Approach-1*,

C4: 5% and 95% confidence level of the curves based on the results of uncertainty analysis (by using the curves of C3).

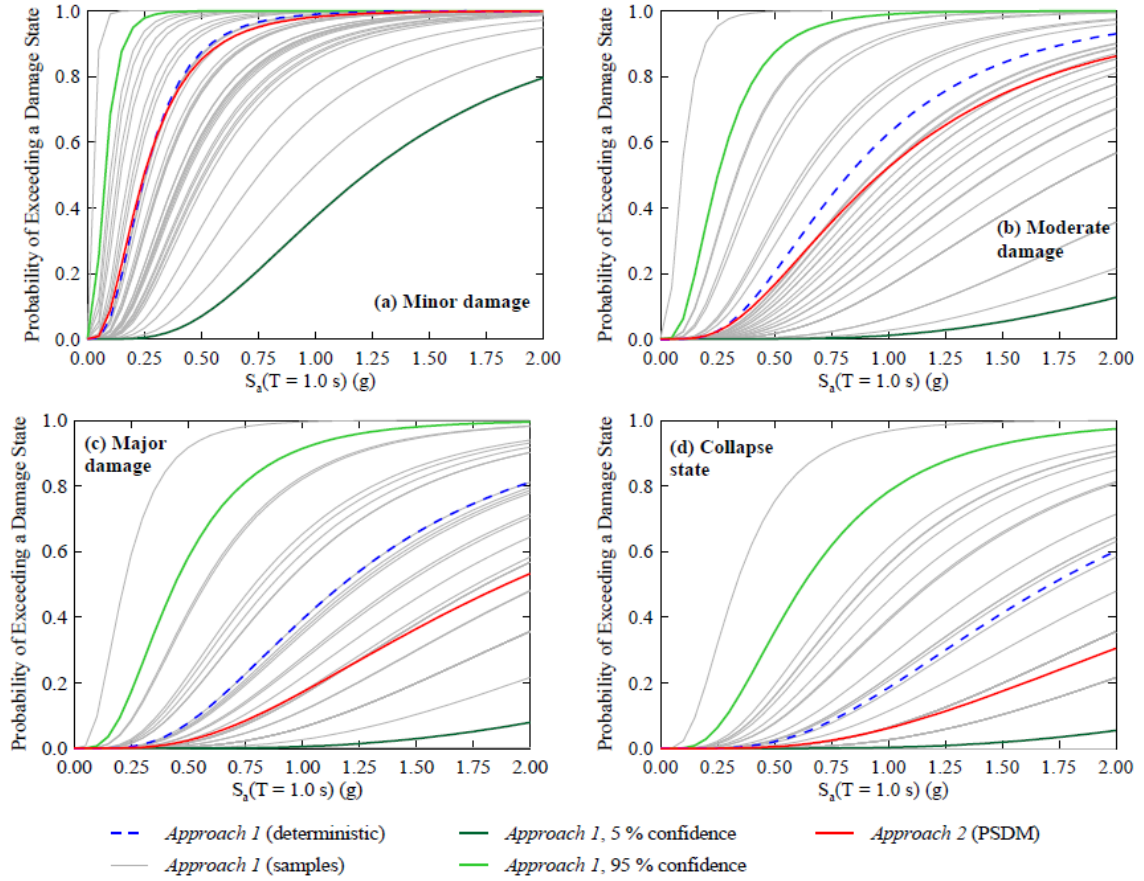


Figure 3-5: Fragility curves obtained from *Approach-1* and *Approach-2*.

In regard to the comparison between the deterministic fragility curve (C1) and the fragility curve obtained from *Approach-2* (C2), it is observed that both approaches yield similar outcomes at lower damage states (minor and moderate damage), while *Approach-1* gives more conservative results than *Approach-2* at higher damage states. This may be attributed to the nature of PSDM, where the best fit line in the linear regression analysis can underestimate some demands especially at high seismic intensity levels, as can be observed from the PSDM in Figure 3-4. The accuracy of both approaches at high damage states can be improved by expanding the ground motion dataset to include more ground motion records with higher seismic intensities. However, increasing the number of ground motions would increase the computational expenses substantially as well. It is

also found that the dispersion (ζ) value adopted from HAZUS [69] for *Approach-1* is in close agreement with the dispersion value resulting from the use of *Approach-2*.

Table 3-2: Fragility parameter results for all approaches.

	Used Approach	Fragility curve	Fragility Curve Parameters				
			Median, λ_k ($IM = S_a(T = 1.0 \text{ s})$ (in g))				ζ
			Minor	Moderate	Major	Collapse	
C1	<i>Approach-1</i>	Deterministic fragility curve	0.252	0.826	1.180	1.716	0.6
C2	<i>Approach-2</i>	PSDM is used	0.248	0.963	1.897	2.820	0.672
C3	<i>Approach-1</i>	50 % confidence level	0.302	1.002	1.438	1.812	0.6
C4		95 % confidence level	0.075	0.253	0.443	0.627	0.6
C4		5 % confidence level	1.218	3.791	4.674	5.233	0.6

Table 3-2 presents the median values of the fragility curves with 50% confidence level in addition to the ones corresponding to 5% and 95% confidence levels (C4). These confidence levels are obtained through the fitted lognormal distribution to the median values resulting from the uncertainty analysis performed under the use of *Approach-1* (C3). Table 3-2 further reveals that both the deterministic fragility curve (C1) and the curve obtained from *Approach-2* (C2) are within the 90% confidence interval, but the C2 curve sways towards the 5% confidence level bound as damage level gets higher. Another outcome is that the median values corresponding to the fragility curve with 50% confidence level are close to the ones for the deterministic fragility curve (C1), which is similar to the outcome obtained by Yilmaz et al. [66].

3.6 Evaluation of the Results

This sub-section presents the evaluation of two different methodologies, referred to as *Approach-1* and *Approach-2*, on developing the seismic fragility curves of highway bridges. For this purpose, FE models of a typical bridge pier is generated and nonlinear time-history analyses are carried out to determine the seismic response of the pier. The fundamental difference between these two methodologies is that in *Approach-1* the fragility curve median values are estimated by using MML, while in *Approach-2* the fragility curve parameters (median and dispersion) are computed through the use of PSDM. The uncertainties in input parameters affecting the seismic response are managed inherently in *Approach-2* owing to the nature of PSDM. On the other hand, when *Approach-1* is used, the impact of the uncertainties is quantified by conducting an uncertainty analysis from which the results are utilized to compute the 90% confidence level fragility curves. Based on the obtained results, the following conclusions can be made:

- *Approach-1*, which is used to compute the deterministic fragility curves, yields a more conservative result than *Approach-2* in which uncertainties are covered through the use of PSDM. On the other hand, this difference is almost diminishes at low damage states.
- The fragility curve dispersion value adopted for *Approach-1* is a reasonable assumption, as it is in close conformity with the dispersion value resulting from *Approach-2*.
- The 90% confidence interval of the fragility curves revealed that there is a large variability in seismic fragilities as a result of parameter uncertainties. However, it should be noted that this outcome is restricted to the condition applied herein due

to the use of a 2D model for the bridge pier, a relatively small number of sample combinations, and large variations of the uncertain input parameters.

The outcomes of this sub-study are believed to be beneficial for the future studies on selection and application of the methodologies for generating seismic fragility curves of bridges or other types of structures. As a result, it is decided that the MML (Method of Maximum Likelihood) is to be used for the course of this thesis, due to its conservative results and its straight-forward demeanor.



CHAPTER IV

TIME-DEPENDENT SEISMIC FRAGILITY ASSESSMENT OF BRIDGES SUBJECT TO FLOOD-INDUCED SCOUR

In this chapter, time-dependent multi-hazard fragility curves (and fragility surfaces thereafter) for ordinary highway bridges are presented. Fragility curves are generated via MML, as argued in Chapter 3. The study bridge that is being investigated is presented and the idea behind the selection process is introduced. For all modeling parameters, the expected values of the parameters as listed in Appendix B are used. The ground motion set and the methodology behind the selection of records are mentioned. The possible deteriorations that might be observed in a river-crossing bridge are discussed, and the analytical models for the most influential mechanisms are given. Modeling philosophy and the details on modeling are featured. Modal analysis results, the process of NLTHA, and typical seismic responses of bridge piers are exhibited. The probabilistic framework that is employed in generating fragility curves is presented, along with a discussion about the consideration of uncertainties and the optimal seismic intensity measure. At the end of the chapter, resultant fragility curves and surfaces are presented. For the analyses, 5 cases for aging and scour are considered, with 25 possible combinations as displayed in Table 4-1.

Table 4-1: Analysis numbers and descriptions.

Analysis No	Scour Depth	Bridge Age
1	0 m (intact)	0-yr
2	0 m (intact)	25-yr
3	0 m (intact)	50-yr
4	0 m (intact)	75-yr
5	0 m (intact)	100-yr
6	1 m	0-yr
7	1 m	25-yr
8	1 m	50-yr
9	1 m	75-yr
10	1 m	100-yr
11	2 m	0-yr
12	2 m	25-yr
13	2 m	50-yr
14	2 m	75-yr
15	2 m	100-yr
16	3 m	0-yr
17	3 m	25-yr
18	3 m	50-yr
19	3 m	75-yr
20	3 m	100-yr
21	4 m	0-yr
22	4 m	25-yr
23	4 m	50-yr
24	4 m	75-yr
25	4 m	100-yr

4.1 Study Bridge

In this sub-section, the study bridge is introduced. First, an abstract review of the bridge inventory in Turkey is presented in 4.1.1, and then the characteristic highway bridge adopted in this study is specified in 4.1.2

4.1.1 Review of the Bridge Inventory in Turkey

There are more than 7000 highway bridges in Turkey, scattered around the country along the highways that interconnect its provinces. The majority of them have been built with concrete. The recent trend in Turkey indicates to a combination of reinforced concrete substructures with superstructures composed of prestressed concrete

girders [2]. Avşar [78] presented a detailed review of the bridge inventory of Turkey. According to that study, approximately 70% of the bridges are non-skewed, most of them have 2-3 spans, total length varying ranging from 40 m to 200 m, individual span lengths varying between 10-50 m, column heights are mostly 6 m or 7 m, and most of them are built with two-column bents. In the light of that study, a characteristic highway bridge is generated as explained below.

4.1.2 Characteristic Highway Bridge

A characteristic structure refers to an *ordinary* and most observed type of structure. Ordinary bridges are generally constructed with a sound geometry on convenient local soil conditions with the most conventional foundations, and sometimes with slight modifications to its geometric and structural aspects. Performance criterion is the ultimate distinguisher between the ordinary and non-ordinary bridges: While ordinary bridges are expected to experience high damage levels and not to be in service after a major earthquake event, non-ordinary bridges are required to be shortly in service after a quick repair or be in full service depending on its importance level. Following such recommendations alongside with the bridge inventory details of Turkey as listed above, the characteristic bridge employed in this study is chosen to be in conformity with the following definition of Caltrans [79] for “ordinary standard bridges”:

- All span lengths are less than 300ft ($\approx$ 90m),
- Box-girder, slab on girder, or slab-girder type superstructures,
- Solid column-type or bent foundations,
- Fundamental period of the structure is 0.700 sec. or greater, both in longitudinal and transverse directions,
- Bridge is farther than 300 ft away from a fault.

The study bridge is a three-span continuous box-girder with seat-type abutments and extended pile shafts. The geometric and structural details of the bridge is transcribed from Yilmaz and Banerjee [67]. The gap between the abutment and the superstructure is taken as 5 cm. Bents are consisted of two circular reinforced concrete columns with a diameter of 1.50 m. Columns are raised 7 m from the ground (intact river bed level). The superstructure width is taken as 17 m, and the distance between the two columns are taken as 9 m. Foundation (pile) depth below ground level is 14 m. Soil is medium-dense sand, and the pile shafts are assumed to extend to a firm rock in the bases. Two approach spans are assumed to be 35 m long, and the middle span length is taken as 45 m, which accounts to a total bridge length of 115 m. The geometric features of the study bridge in the longitudinal direction and the box-girder are displayed in Figure 4-1. The elevation view of the bridge in the transverse direction is the same as demonstrated in Figure 3-1a. All modeling parameters are taken at their expected values as displayed in Appendix B.

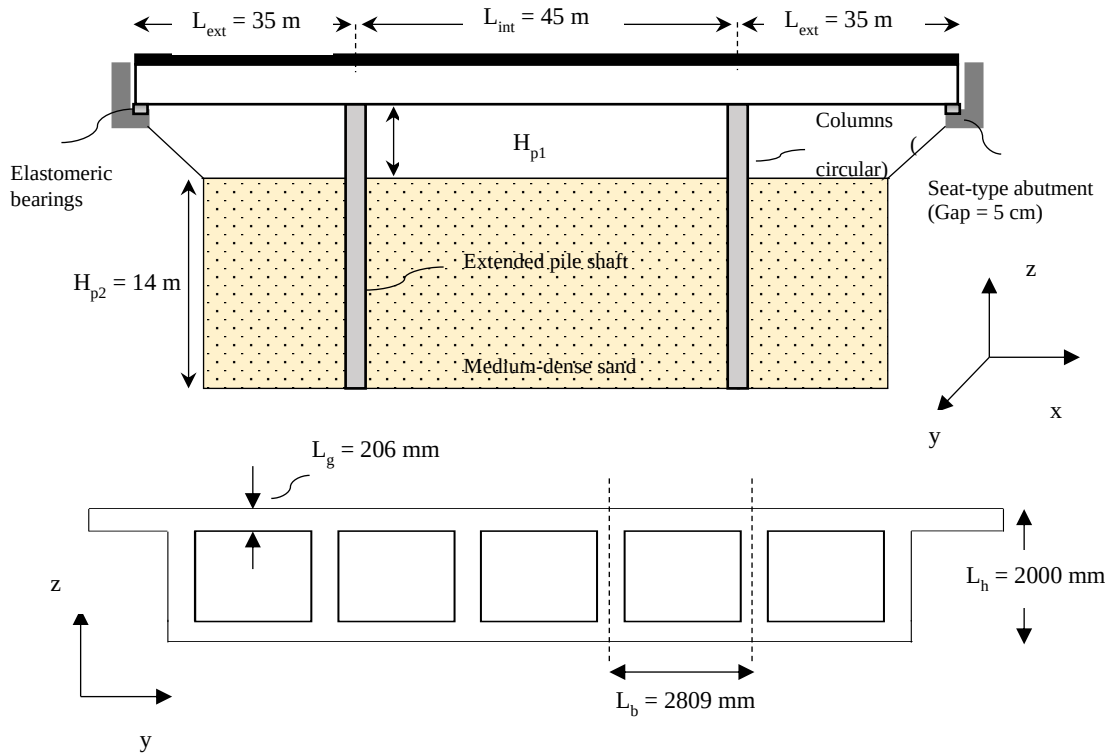


Figure 4-1: Geometric details of the study bridge in the longitudinal direction and section details of the box-girder.

4.2 Ground Motion Data Set

Ground motion (GM) data set constitutes one of the most important parts of the seismic risk studies. The data set should be convenient enough to reflect a broad hazard scenario that is independent from a specific site location. GM data sets may be devised in several ways. First of all, there are various available sets that are compiled with real ground motions in the literature, using a stochastic selection methodology. Baker [80] is one example of this: It is developed with the *Conditional Mean Spectrum* (CMS) methodology as it is specified in Jayaram et al. [81]. Also, there are “synthetic sets” that are being consisted of simulated ground motions. One example to it would be Rix and Fernandez [82], which was previously employed in studies such as Ghosh and Padgett [19]. On top of these, it is also possible to put together a set “manually” by picking ground motions with certain characteristics. In other words, these kinds of sets are developed by searching ground motions in databases by specifying the intervals of magnitude, shear wave velocity, distance to rupture and fault type. Avşar et al. [18] and Yilmaz et al. [13] sets were developed in that manner.

Among the alternatives, CMS method is employed to pick the ground motions that are to be used for Chapter 4. The methodology employed herein is expressed as presented in Baker [80], in which the standard data sets for PEER Transportation Research Program were developed. These state-of-the-arts data sets are deemed to be practical for seismic vulnerability studies and widely employed to construct the fragility curves for bridges [26]. Even though the Baker [80] data sets are deemed as readily available for regions that are mid-to-high seismicity, the procedure is nevertheless repeated in this study to reach a tailor-fit set that would reflect the unique features of the Turkey region.

The flowchart for this method is presented in Figure 4-4. In this methodology, conditional mean response spectrum for a case is represented with a multivariate normal

random variable $R(\mu, \Sigma)$. Mean matrix “ μ ” and the covariance matrix “ Σ ” are further utilized to generate “response spectrum samples”, which can be later used to find a ground motion that has a matching spectrum. The process is as follows: Initially, certain magnitude (M), distance (R) and $\varepsilon(T_{int})$ values are designated to define the backbone of the attenuation. Baker et al. [27] considered various scenarios varying in magnitude (M) and distance (R) in his study. Likewise, 6 cases are selected for this thesis study: (i) M = 6.5, R = 10 km; (ii) M = 7.0, R = 10 km; (iii) M = 7.5, R = 10 km; (iv) M = 6.5, R = 20 km; (v) M = 7.0, R = 20 km; (vi) M = 7.5, R = 20 km. The reason for considering several cases is to represent a broad range of hazard scenarios in the data set, which is independent from a specific site location.

“ ε ” is the number of standard deviations which a given $\ln S_a$ (lognormal of spectral acceleration) differs from the mean prediction as attained from the ground motion prediction model for a period of interest T , which is readily integrated to the modern attenuation relationships. $\varepsilon(T_{int})$ may be assumed as “2”, as argued in Baker [80]. T_{int} is the initial period of interest, typically the fundamental period of the structure or simply $T = 1$ sec. This $\varepsilon(T_{int})$ is utilized as the backbone to find the ε values in all periods. In other words, $\varepsilon(T_{int})$ is used to find the $\varepsilon(T_n)$, for all the values of T that constitute the response spectrum. This can be evaluated as [81]:

$$\varepsilon(T_n) = \rho(T_n, T_{int}) * \varepsilon(T_{int}) \quad (4-1)$$

where $\rho(T_n, T_{int})$ is the correlation between the period of interest and the initial period T_{int} . Baker and Jayaram [83]’s algorithm for the correlation relationship is used in this study to determine $\rho(T, T_{int})$ values for all T .

After obtaining $\varepsilon(T)$ for all T ’s, an attenuation relationship is employed to predict the spectral acceleration values and to obtain the conditional mean spectra for all 6 cases.

For this study, GMPEs that are presented for Middle East and Europe region by Akkar and Bommer [84] are employed. Mean matrix μ for a case is determined as [81]:

$$\mu = \begin{bmatrix} \mu_{lnSa(T1)} + \rho(T_1, T_{int})\varepsilon_{Tint}\sigma_{lnSa(T1)} \\ \mu_{lnSa(T2)} + \rho(T_2, T_{int})\varepsilon_{Tint}\sigma_{lnSa(T2)} \\ \vdots \\ \mu_{lnSa(Tn)} + \rho(T_n, T_{int})\varepsilon_{Tint}\sigma_{lnSa(Tn)} \end{bmatrix} \quad (4-2)$$

where $\mu_{lnSa(Tn)}$ is the mean and $\sigma_{lnSa(Tn)}$ is the standard deviation of the spectral acceleration that are computed from the GMPEs for the n^{th} period of interest T_n . Covariance matrix for a case is determined as [81]:

$$\Sigma = \Sigma_0 - \frac{1}{\sigma_{lnSa(Tint)}^2} \Sigma_1 \Sigma_1^T \quad (4-3)$$

$$\Sigma_0 = \begin{bmatrix} \sigma_{lnSa(T1)}^2 & \cdots & \cdots & \rho(T_1, T_n)\sigma_{lnSa(T1)}\sigma_{lnSa(Tn)} \\ \rho(T_2, T_1)\sigma_{lnSa(T2)}\sigma_{lnSa(T1)} & \cdots & \cdots & \rho(T_2, T_n)\sigma_{lnSa(T2)}\sigma_{lnSa(Tn)} \\ \vdots & \cdots & \cdots & \vdots \\ \rho(T_n, T_1)\sigma_{lnSa(Tn)}\sigma_{lnSa(T1)} & \cdots & \cdots & \sigma_{lnSa(Tn)}^2 \end{bmatrix} \quad (4-4)$$

$$\Sigma_1 = \begin{bmatrix} \rho(T_1, T_{int})\sigma_{lnSa(T1)}\sigma_{lnSa(Tint)} \\ \rho(T_2, T_{int})\sigma_{lnSa(T2)}\sigma_{lnSa(Tint)} \\ \vdots \\ \rho(T_n, T_{int})\sigma_{lnSa(Tn)}\sigma_{lnSa(Tint)} \end{bmatrix} \quad (4-5)$$

For all 6 cases, corresponding μ and Σ matrixes are computed, and 15 samples are taken via Multivariate Monte Carlo Simulation. Conditional mean spectrum and simulated spectra for all cases are shown in Figure 4-2.

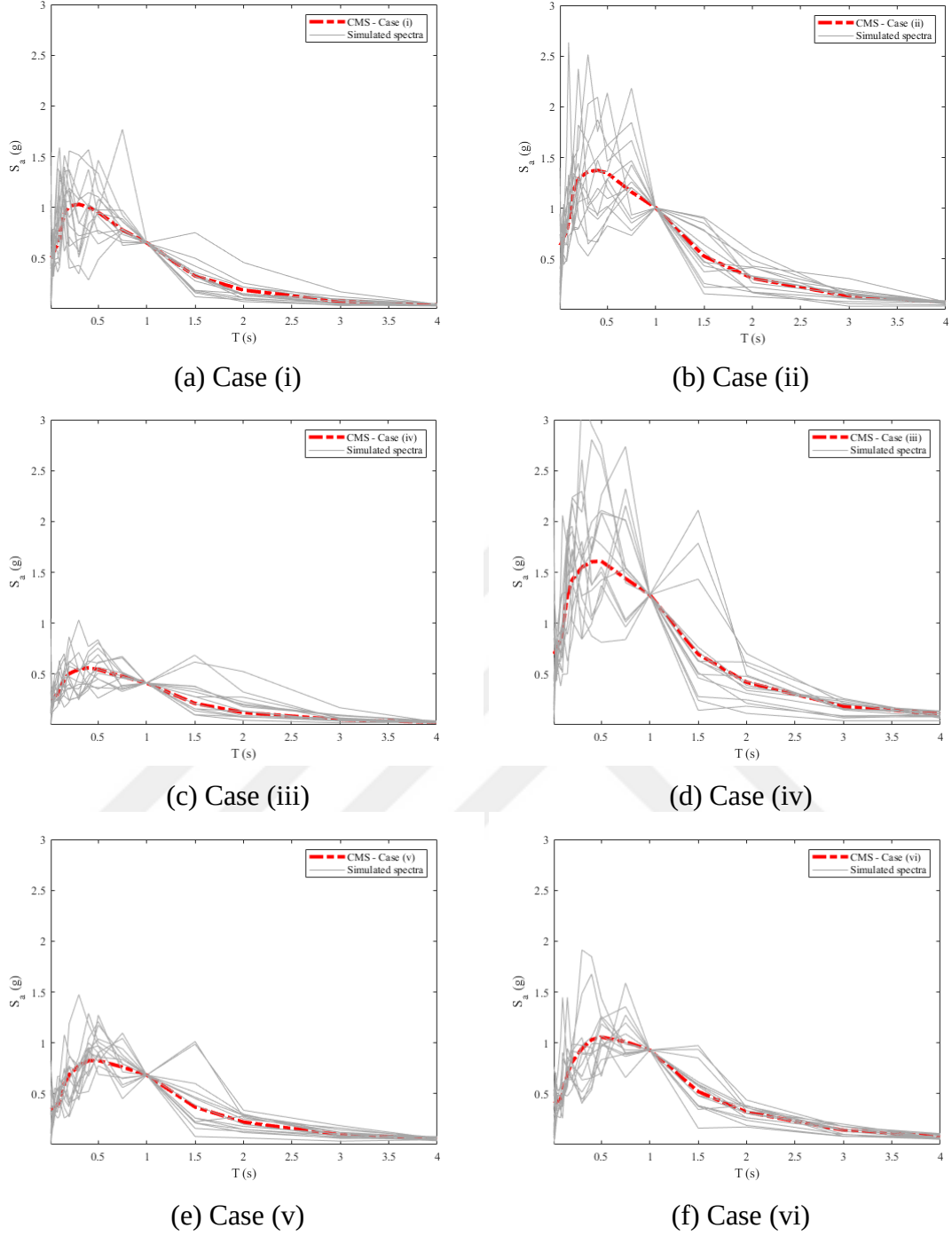


Figure 4-2: Conditional mean spectrum and simulated spectra for all considered earthquake scenarios.

In total, 90 unique ground motions that match the simulated spectra from 6 cases are selected from NGA-West2 database [71]. 20 of the selected ground motions were relegated later on to cut down the computational expenses. From the remaining 70, 10 of

them are further scaled with 2 to increase the dataset's ability of inflicting higher levels damage to the bridge structure. So, the final set is consisted of 80 ground motions. Ground motion names, components and their intensity measures are detailed in Appendix D.

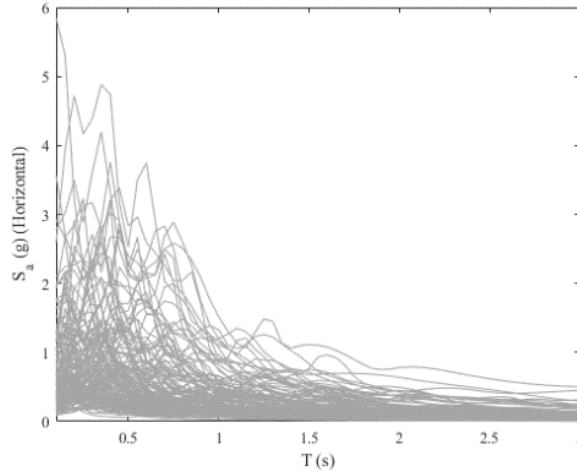


Figure 4-3: Response spectra of the selected ground motions for Chapter 4. Scaled ground motions are included.

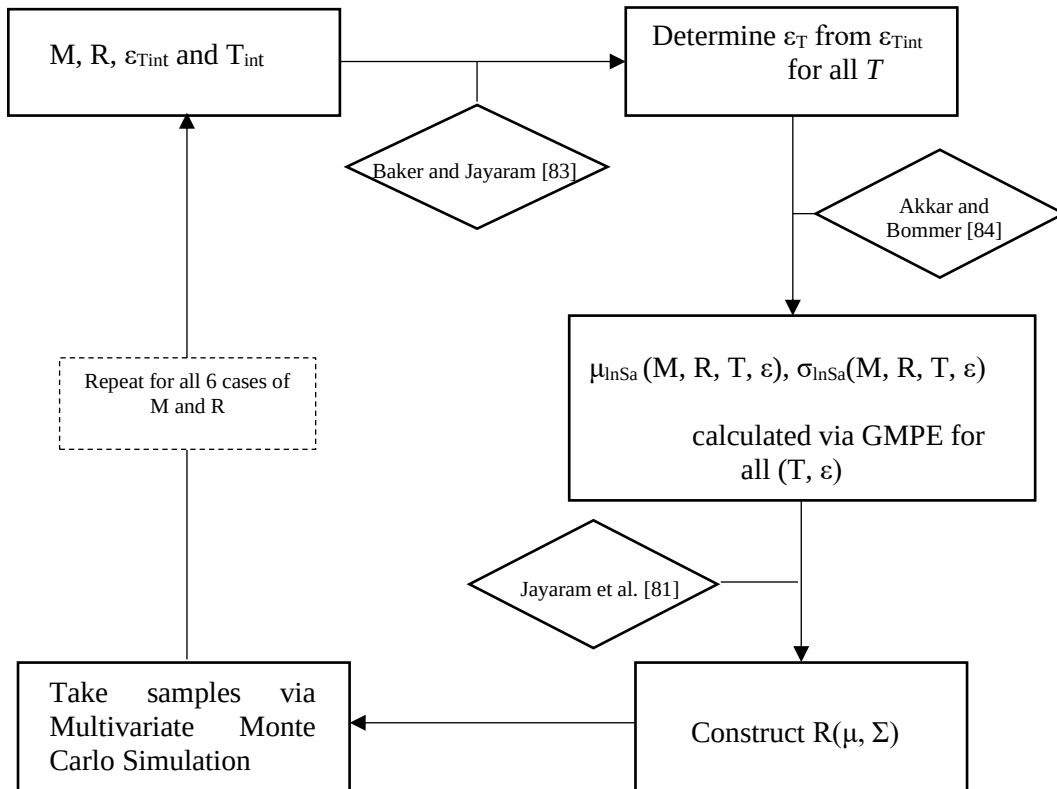
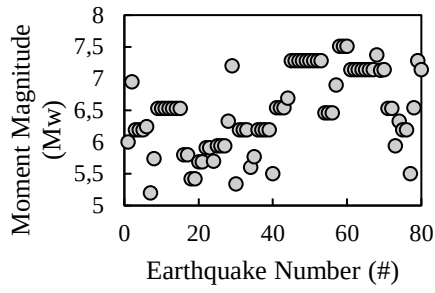
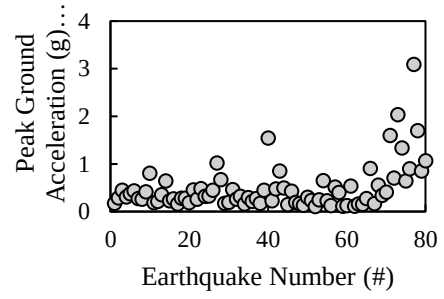


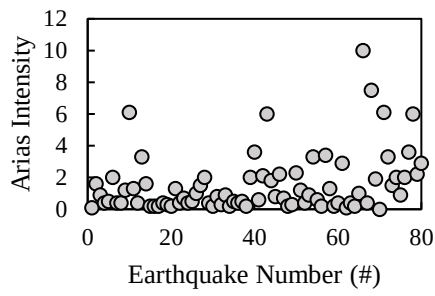
Figure 4-4: Flowchart for the ground motion selection process via Conditional Mean Spectrum method.



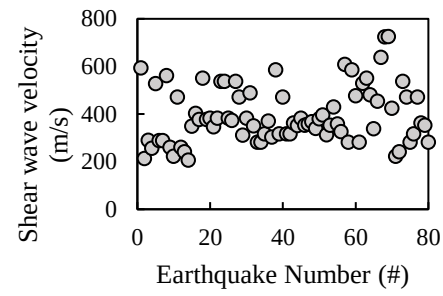
(a) For moment magnitude



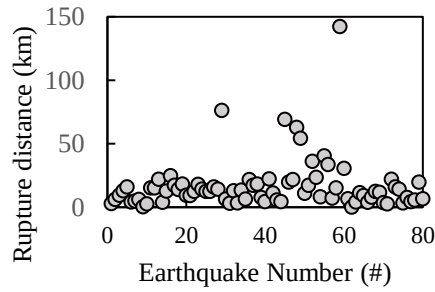
(b) For peak ground acceleration



(c) For Arias intensity



(d) For shear wave velocity



(e) For rupture distance

Figure 4-5: Scatter of the features of the ground motion records in the data set developed for Chapter 4.

4.3 Deterioration of Bridge Components

This subsection is written to evaluate the known aging mechanisms and present the deterioration factors that are considered in this study. Two main bridge components that are subject to deterioration are piers and bearings. A discussion is included in the end of this introduction about the severity of the individual and combined aging of bridge piers and bearings. Other than that, bridge deterioration might be applied to shear keys by reducing the material strength. However, preliminary observations by the author revealed that the reduction is negligible, thus it is not included in this chapter.

There are several aging mechanisms concerning the bridge piers, and the most prominent of them is corrosion in steel reinforcement. Naturally, reinforcements are protected from corrosion process by the so-called passivation layer that is formed by pore solution, which is highly alkaline. However, this alkalinity might be influenced by localized chloride ingress, which effectively neutralizes the passivation layer, causing a corrosion initiation that later on leads to the replacement of steel with rust products [85]. This phenomenon is widely observed on bridges that are located in marine areas where they are subjected to airborne chloride ions or direct salty water contact, and at cold winter regions where deicing salts are used to sustain the traffic. Highway bridges located in areas where a significant temperature difference is observed during the day are subjected to freeze and thaw cycles. Volumetric expansion of water molecules due to freezing causes the formation of microcracks inside the concrete pores. Consequently, those pores absorb more water, which leads to more microcracks in the next cycle. This causes significant deterioration in durability, especially for the bridges having contact to water and humidity [86]. Pouring of mass concrete in bridge pier construction might result in alkali-aggregate reaction (AAR). AAR is a chemical reaction between aggregates and cement, which creates an undesired volumetric expansion of concrete, immediately

leading to internal cracks. Such deformities might gradually increase as the bridge ages, eventually reaching the external surface. Sulfate attack is another mechanism that would internally or externally deteriorate reinforced concrete structures. Internal sulfate attack is quite similar to AAR, as it happens due to large concrete pours. Big temperature elevation leads to the delayed formation of ettringite. This delayed formation causes unexpected volumetric expansion inside the concrete, which leads to internal cracking. External sulfate attack is common for the bridges that are in contact with sulfate rich water, i.e. sea water, ground water, rivers etc. It causes the formation of gypsum and ettringite on a large scale around a RC element and initiates cracking [87].

All the aging mechanisms in concrete cause some form of cracking, while corrosion induces both microcracks and alterations in reinforcement material. The effects of cracking are observed on the mechanical properties of concrete, which might be implemented to FE models by small modifications on concrete material properties. One interesting point here is that it is shown that microcracks in concrete may exacerbate the chloride ingress, which would decrease the corrosion initiation time [39]. For this and similar reasons, combination of aging effects and its multiple adversities are an interesting field of study. Theoretically, a highway bridge that is located in Northeastern America would be in contact with chloride and sulphate rich water and subsequently would be subjected to chloride induced corrosion, sulphate attack, AAR and freeze-thaw mechanisms. Even though design codes are developed to ensure durability, the effects of such deterioration mechanisms on the structural performance are well pronounced in a hazard scenario.

Elastomeric bearings and their mechanical properties are highly sensitive to temperature. It most prominently induces thermal oxidation on the rubber material used in elastomeric pads. Thermal oxidation is a process where the oxygen molecules are

reacting with the rubber material, releasing carbon dioxide and water. Rubber material, as the reactant, is hardened as a result of this process, thus there is an increase in its horizontal stiffness. The seminal work conducted by Itoh and Gu [56] revealed that this hardening is most pronounced in the early ages of the bridge, in a period called “spontaneous reaction”. The horizontal stiffness of the rubber material quickly goes up to certain levels until an equilibrium point, where the diffusion of oxygen molecules into the material is practically halted due to the accumulation of oxidation products.

A preliminary study was conducted by Yılmaz and Aygün [58] to observe the individual and combined effect of thermal oxidation and chloride-induced corrosion on the seismic risk of highway bridges. Time-dependent fragility curves were generated at certain ages of the bridge, implementing the deterioration effects to the bridge piers and elastomeric bearings, then the risk curves were generated for different ages of the bridge. The subtle aim of this study was to determine whether the deterioration in bearings are influential on the overall seismic risk or not. The key findings of this study are noted below. Readers are referred to the original proceeding for further details.

- Chloride-induced corrosion in RC piers is the main contributing factor towards the seismic risk increase over the lifetime of the bridge, and its adverse effects are most pronounced in mid-to-later stages of the bridge service life.
- The effect of thermal oxidation in bearings to the lifetime risk of the bridge is negligible. However, due to increased horizontal stiffness, the performance of bearings increases slightly at lower damage levels.

Results of this study prompted the author to neglect the deterioration in bearings and continue solely with the effects of chloride-induced corrosion in bridge piers. Corrosion primarily results in a decrease in steel area of the reinforced concrete section. Its secondary effects include a strength loss in cover concrete due to the cracking induced

by chloride ingress, a change in yield strength due to the rust accumulation and a change in confined concrete strength due to a loss in transverse reinforcement area. These effects are detailed in following subsections.

4.3.1 Chloride-Induced Corrosion at Reinforced Concrete Piers

Piers are the pivotal bridge elements to resist seismic loading. For this reason, any alteration in its performance may impact the overall response against an earthquake event. Corrosion is one of the most influential deterioration mechanisms that might occur on an RC section, as its effects are well pronounced on both the reinforcement and the concrete properties.

Corrosion initiation time for a RC section is mainly governed by the thickness of the cover concrete, environmental conditions and the overall quality of the concrete product. If the concentration of chloride near the reinforcement is assumed constant over time, corrosion initiation time can be estimated with the following equation [40] [41]:

$$T_i = \frac{x^2}{4D_c} \left[\operatorname{erf}^{-1} \left(\frac{C_0 - C_{cr}}{C_0} \right) \right]^{-2} \quad (4-6)$$

where T_i is the corrosion initiation time, x is the clear cover of concrete (cm), D_c is the chloride diffusion coefficient (cm^2/year), C_0 is the equilibrium chloride concentration at concrete surface (% weight of concrete), C_{cr} is the critical chloride content at which corrosion begins (% weight of concrete) and erf is the Gaussian error function which is defined as:

$$\operatorname{erf}(z) = \frac{2}{\sqrt{\pi}} \int_0^z e^{-t^2} dt \quad (4-7)$$

For the case of marine exposure to chloride ions, minor improvements to include the quality of concrete and environmental conditions are introduced into Equation 4-8 by Duracrete [88]:

$$T_i = \left(\frac{x^2}{4k_e k_c D_c (t_0)^{n_{cl}}} \left[\operatorname{erf}^{-1} \left(\frac{C_0 - C_{cr}}{C_0} \right) \right]^{-2} \right)^{\frac{1}{(1-n_{cl})}} \quad (4-8)$$

where k_e is the environmental factor, k_c is the curing factor, t_0 is the age of the concrete at which the compliance test is performed and n_{cl} is the age exponent which takes the cement paste densification into account. The values for the aforementioned parameters are taken from Duracrete [88] and summarized below. Environmental factor k_e changes with the exposure condition and Duracrete [88] provided the values for these four cases: submerged, tidal, splash-zone and atmospheric exposure to salty water. The study bridge has contact with river water herein, which is not as chloride-laden as salty sea water. Hence for this study, such contact is assumed to be represented with the atmospheric exposure condition.

Table 4-2: Values of the parameters taken for corrosion initiation estimation.

Parameter	Unit	Value
x	(mm)	50
k_e	(N/A)	0.265
k_c	(N/A)	1.5
D_c	(mm ² /year)	473
t_0	(year)	0.077
n_{cl}	(N/A)	0.362
C_0	(by weight %)	3.879
C_{cr}	(by weight %)	0.5

After corrosion initiation, the steel is gradually replaced by rust products at a certain rate r_{corr} as the primary effect of corrosion. If a uniform corrosion is assumed, the diameter of the reinforcing bar at a given time t can be calculated as:

$$D(t) = D_i - r_{corr}(t - T_i) \quad (4-9)$$

where D_i is the initial diameter of the reinforcing bar and $D(t)$ represents the diameter at time t . The constant corrosion rate r_{corr} (in mm/year) can be calculated as [43]:

$$r_{corr} = 0.0116 \frac{37.8 - (1 - w/c)}{x} \quad (4-10)$$

where w/c represents the water to binder ratio in the concrete mixture, which can be estimated using the Bolomey's formula, which utilizes the compressive strength of the concrete [43]:

$$w/c = \frac{27}{f'_{cyl} + 13.5} \quad (4-11)$$

where f'_{cyl} represents the cylindrical strength of concrete in MPa. A noteworthy argument would be held against the coefficient “0.0116” in Equation 4-10. It is highly influential on corrosion rate, thus should be depicted carefully. The value 0.0116 is attributed to a typical environmental condition, for 75% humidity and average temperature of 20°C, which is applicable to many locations around the globe [43]. It may be replaced with a tailor-fit version for the location of interest for future studies. It should also be noted that uniform corrosion along the bar and a constant corrosion rate is assumed in this study. It is known that the chloride-induced corrosion can cause a more localized corrosion, i.e. corrosion products accumulate in certain clusters along the bar (referred to as pitting corrosion). Furthermore, the chloride diffusion slows down due to the gradual accumulation of rust, thus resulting in a time-dependent corrosion rate, which tends to

decrease as the bridge ages. Nevertheless, uniform and constant-rate corrosion is deemed as a valid assumption, as this assumption is widely observed in the literature.

4.3.2 Secondary Effects of Corrosion

Alongside the primary phenomenon of steel area reduction, chloride-induced corrosion collaterally produces several secondary effects.

Strength of the reinforcement decreases due to the replacement of steel materials with rust products. The yield and the ultimate strengths of the reinforcement can be determined from the following formula proposed by [48]:

$$f(t) = (1 - 0.005Q_{corr}(t))f_0 \quad (4-12)$$

where f_0 is the initial yield or ultimate strength of the rebar, $Q_{corr}(t)$ is the percentage (%) loss in rebar diameter at time t .

Reduction in confined concrete strength might be reflected into FE models by making a slight modification in the confined concrete compressive strength through Mander et al. [47]'s equation. The process is as follows:

1. Compute the remaining transverse reinforcement diameter at time t with Equation 4-9.
2. Compute the yield strength of the transverse reinforcement at time t with Equation 4-12.
3. Determine the effective lateral confining pressure at t $f_l(t)$ via Mander et al. [47].
4. Evaluate the confined concrete strength at time t $f_c(t)$ as in Equation 4-13:

$$f_c(t) = f'_{cyl} \left(2.254 \sqrt{1 + \frac{7.94f_l(t)}{f'_{cyl}}} - 2 \frac{f_l(t)}{f'_{cyl}} - 1.254 \right) \quad (4-13)$$

An important remark can be made about the cover concrete strength. It is known that the chloride-induced corrosion induces microcracks in the clear cover of the section, which collaterally causes a loss in compressive strength of the cover concrete. Preliminary calculations made by the author using the analytical model proposed in Rao et al. [46] revealed that the cover concrete strength might degrade from 32.50 MPa down to 2.01 MPa in 100 years of service life. However, implementing this phenomenon into the FE models caused massive convergence issues, thus excluded from the methodology herein. Readers are referred to Rao et al. [46] and Shekhar et al. [20] for further details.

4.3.3 Summary

Aging effects considered on a reinforced concrete section are schematically summarized below in Figure 4-6. The modeling parameter values for different ages of the bridge are summarized in Table 4-3 below.

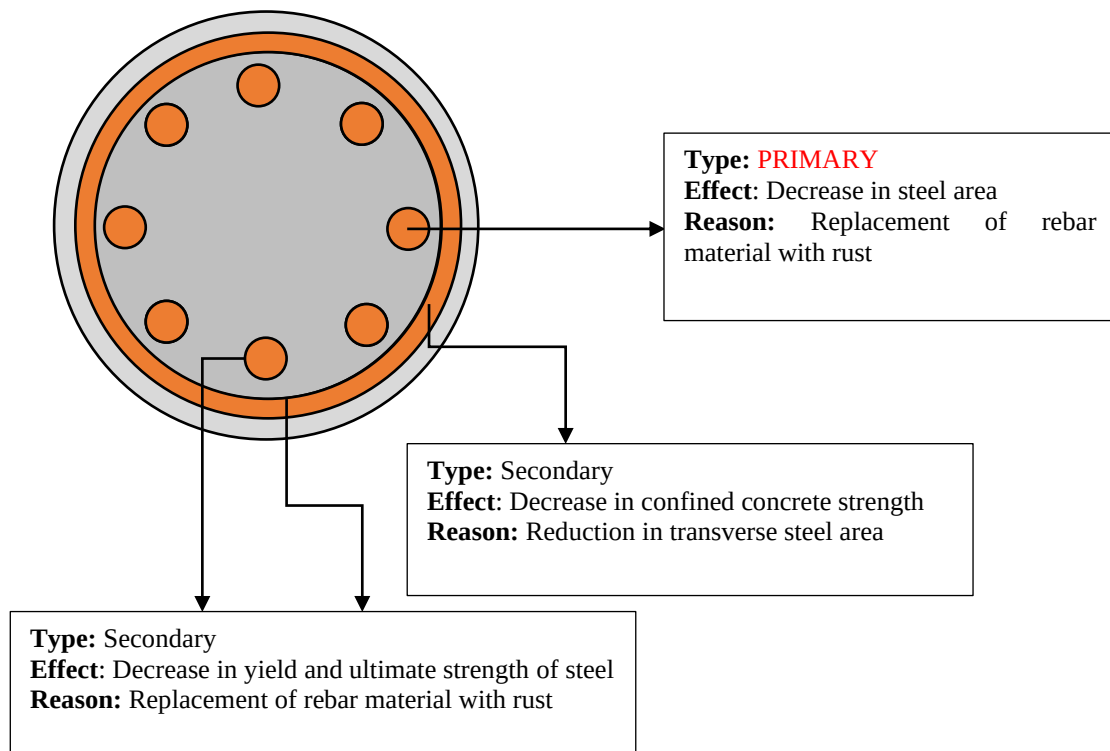


Figure 4-6: Schematical representation of deterioration on the reinforced concrete section.

Table 4-3: Modeling parameter values for different ages of the bridge.

Case	Modeling parameters				
	d_{long} (mm)	f_{y_long} (MPa)	f_{u_long} (MPa)	d_{trans} (mm)	f_{y_trans} (MPa)
0-yr	36.00	475.00	650.00	25.00	475.00
25-yr	35.91	474.39	649.16	24.91	474.12
50-yr	34.97	468.22	640.73	23.97	465.24
75-yr	34.04	462.06	632.29	23.04	456.36
100-yr	33.10	455.89	623.85	22.10	447.48

* d_{long} = Bar diameter of longitudinal steel, f_{y_long} = Yield strength of longitudinal steel, f_{u_long} = Ultimate strength of longitudinal steel, d_{trans} = Bar diameter of transverse steel, f_{y_trans} = Yield strength of transverse steel

4.4 Bridge Modeling

All study bridges considered herein are modeled in the *Open System for Earthquake Engineering Simulation* (OpenSEES) platform [68]. The reason for this selection is to benefit from the vast and modifiable library of materials and elements, and from its extensive tools for nonlinear structural analysis.

The standard seismic design philosophy for ordinary bridges is to allow damage in columns, which can be repaired without serious repercussions. Foundation systems and superstructure elements are required to be designed as capacity-protected elements as post-earthquake repair for these elements would be much more difficult than columns. This implies that the superstructure will essentially stay in its elastic range during an earthquake event. Shear keys are used to resist the lateral movement of the superstructure that might occur during an earthquake event, but in the meantime they are ideally required to act as fuse elements in order to reduce the transfer of inertial forces from the superstructure to substructure elements. However, they are generally designed as stiff and

strong elements which can hardly experience (the intended) damage in practice. Abutments are utilized to absorb the energy of longitudinal and transverse movements of the superstructure, receives damage by doing so. Bearings are used to prevent the joint movement of superstructure and substructure, and by doing so it limits the damage in latter. These bearing systems might undergo excessive deformations and damage until they slip out from their installments. Following these general rules, nonlinearity is implemented to FE models for abutments, bearings, shear keys, bridge columns and extended pile shafts. Foundation is assumed to have been extended to a firm rock, thus the full-fixity condition is applied to the bases of the piles. The superstructure (box-girder) is modeled with elastic elements. To account for the lateral and vertical skin resistance of the soil, p - y and t - z springs are implemented respectively in the FE models. Properties for the p - y and t - z springs are calculated with the methodologies presented in API [70] and Mosher [89] respectively.

Rigid elements are used in several places in the model. For example, at a bridge bent, they are placed between tips of the columns and the center of the bridge girder to implement the monolithicity and ensure the connectivity. Furthermore, it has been used to connect the nodes representing the abutment backwall and the backfill soil.

In the following subsections, the modeling approach for each of the bridge elements are detailed and discussed. The bridge modeling procedure employed in this thesis study follows the approach applied in Yilmaz et al. [13].

4.4.1 Modeling of the Superstructure

In ordinary highway bridges, superstructures are designed to stay in elastic range during any earthquake scenario. Because of that, *elasticBeamColumn* element within the library of OpenSEES [68] is used to represent the superstructure in general. In this element object, the program requires the user to define the area (A) of the section, elastic

modulus of concrete, shear modulus of concrete, the moment of inertia in the local y and z direction (I_y and I_z respectively), and the polar moment of inertia (J) of the section. In this study, the sectional parameters of a superstructure element are attained from the geometric properties of the box-girder, as demonstrated in Figure 4-7.

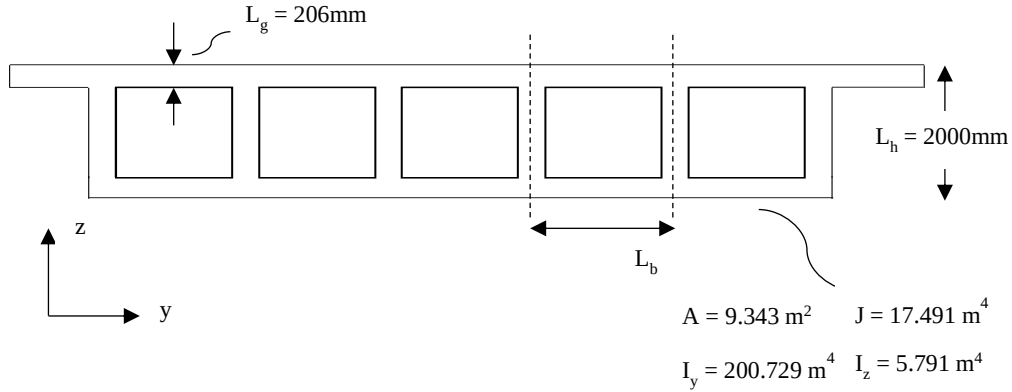


Figure 4-7: Geometric properties of the box-girder.

4.4.2 Modeling of the Columns and Extended Pile Shafts

Piers are the main lateral load resisting elements of bridges. They can undergo excessive deformations and should be designed to bear large earthquake demands to limit damage. To account for the inherent inelastic behavior, a fiber section is developed in OpenSEES [68] that can incorporate the nonlinear material definitions of concrete and reinforcing steel. *Concrete07* material model is employed for the concrete core and the cover; and the input parameters for these models are determined in accordance with Mander et al. [47]. *Steel02* material model is used to reflect the characteristics of the longitudinal reinforcements of the section. *dispBeamColumn* element object assigned with the aforementioned fiber section is implemented to represent the bridge columns and the extended pile shafts. Note that for the study bridge, the columns and extended pile shafts share the same sectional properties and reinforcement as can be seen in Figure 4-8.

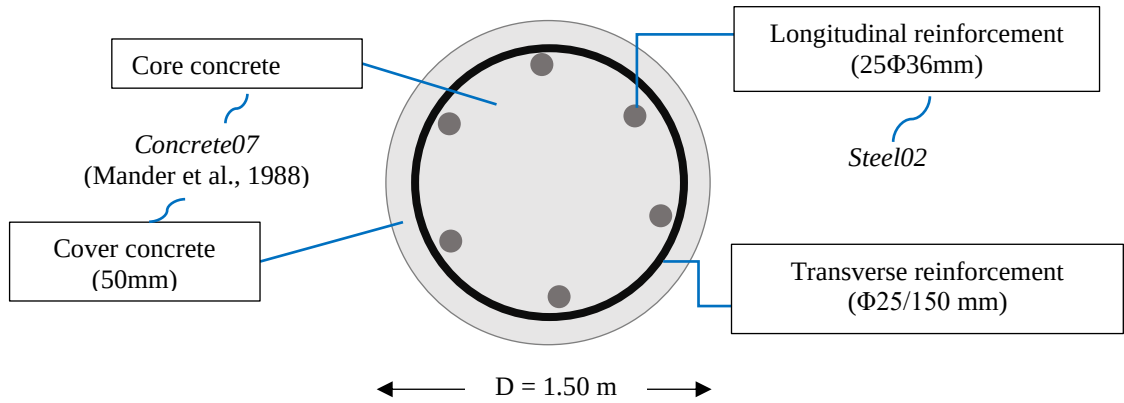


Figure 4-8: Section details of the columns and extended pile shafts.

Fiber section builder in OpenSEES [68] requires the user to divide the section into definite number of fibers. Amount of this subdivisions determines the accuracy of the result: sections with higher discretization yield more accurate results than its counterparts with less subdivisions. However, increasing the discretization in a fiber model leads to an increase in computational expenses, both in performance and in time. Thus, an optimized fiber section should be developed, which would solve the system accurately with a feasible amount of computational power.

For that reason, a validation study for a fiber section is conducted to find the optimal discretization in such fiber sections. A column experiment conducted by Moyer and Kowalsky [90] is utilized in that endeavor, the test cantilever column configuration is modeled in OpenSEES using several alternative fiber sections in *dispBeamColumn* elements, and the results are compared with the top displacement vs base shear force data obtained from the experiment. The resulting comparison between the response of the most optimal fiber section and the test data is presented in Figure 4-9. As can be deduced from this figure, the determined fiber configuration and the considered material models is sufficient for further application in the bridge models that will be employed in the nonlinear time-history analyses.

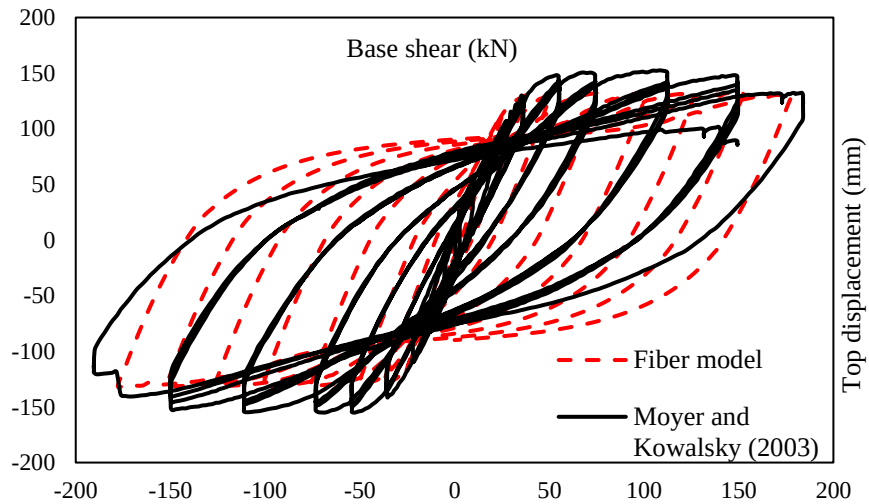


Figure 4-9: Validation of modeling fiber sections.

4.4.3 Modeling of Abutments, Bearings, Shear Keys

Abutments are one of the key elements in resisting the lateral shocks in bridge structures. In the study bridge, they are consisted of backwalls, wingwalls, bearings and shear keys. For all the elements listed here, *zeroLength* element objects are used by representing their behavior with the backbone curves defined in accordance with the observed element behaviors.

Abutment backwall is responsible from the interaction between the superstructure and the soil backfill. In seat-type abutments, superstructure is resting freely on the abutments until the seismic shocks make it to move towards the backwall in longitudinal direction and close the gap between them, or move against shear keys in transverse direction and disturb the bearings below the box-girder flanges. The gap between the superstructure and backwall at both ends of the bridge is assumed as $\Delta_{\text{gap}} = 50$ mm for the study bridge herein. *elasticPPGap* material object is used to define the backbone curve of the abutment longitudinal response, which is illustrated in Figure 4-10.

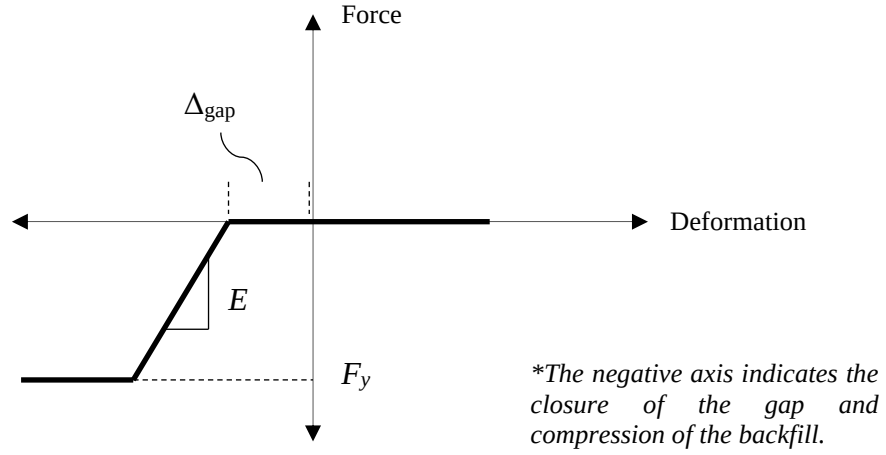


Figure 4-10: Backbone curve of the abutment in longitudinal direction for seat-type abutments (*elasticPPGap*).

Yield strength in the longitudinal direction (F_y) and the elastic stiffness (E) of the backwall element can be calculated from Equations 4-14 and 4-15 as recommended by Caltrans [91]:

$$F_y = (1.5) \times A_e \times 239kPa \times \frac{h_{bw}}{1.7} \quad (4-14)$$

$$E = K_i \times w \times \frac{h_{bw}}{1.7} \quad (4-15)$$

where, A_e is the effective abutment wall area (in m^2), w is the width (in m) and h_{bw} is the height of the abutment backwall (in m), respectively. K_i refers to the initial stiffness of the abutment, and it is recommended as 14.35 kN/mm/m for poor embankment fill, and 28.7 for a suitable fill. In this study, the average value of these two, which is 21.52 kN/mm/m is used for the initial stiffness value. Please note that Equation 4-14 is amplified with a factor of 1.5 (i.e. increased by 50%) to account for the dynamic loading conditions as applied in Yilmaz et al. [13]. Readers are referred to the original document Caltrans [91] for further details on abutment backwall response.

Aviram et al. [92]'s study is implemented to assess the backbone curves for the abutment response in transverse direction. In this methodology, the interaction between the wingwall and backfill in the transverse direction is represented via elastic-perfectly plastic backbones (as demonstrated in Figure 4-11), thus *elasticPP* (elastic perfectly plastic) material definition is assigned to the two zero-length elements at both sides of the abutment. Note that the actual compression-only response of the wingwalls are assumed to be shared equally by the considered zero-length elements with *elasticPP* material responding in both compressional and tensional directions, that's why the yielding force is divided into two on the assigned backbone curve shown in Figure 4-11. A rigid link is used to represent the whole abutment element connecting the wingwall elements at each end of the abutment.

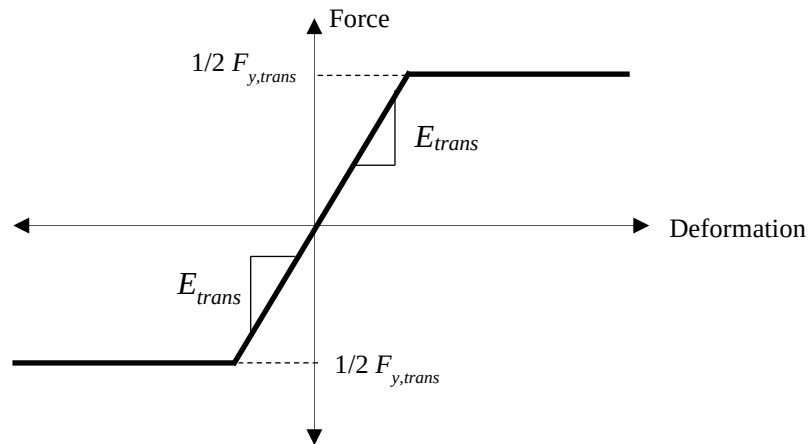


Figure 4-11: Backbone curve for the abutment response in transverse direction (*elasticPP*).

Abutment yield strength ($F_{y,trans}$) and stiffness (E_{trans}) in the transverse direction are determined by making slight modifications to the backwall longitudinal response described in Caltrans [91]:

$$F_{y,trans} = (1.5) \times C_w \times (h_{ww} \times w_{eff}) \times (239kPa) \times \frac{h_{ww}}{1.7} \quad (4-16)$$

$$E_{trans} = C_w \times K_i \times w_{eff} \times \frac{h_{ww}}{1.7} \quad (4-17)$$

$$w_{eff} = C_L \times w \quad (4-18)$$

where w is the wing wall length, h_{ww} represents the abutment wing wall height, w_{eff} is the effective wall length, C_L is the effectiveness factor and C_w is the participation coefficient. C_w and C_L is taken as 4/3 and 2/3 respectively, and w can be taken as 1/2 or 1/3 of the backwall length as recommended by Aviram et al. [92]. Please note that the force capacity is again increased by 50% to account for the dynamic loading conditions.

Bearing response in two horizontal directions are represented with the backbone curve shown in Figure 4-12. Again, *elasticPP* material is used. Yield strength ($F_{y,bearing}$) and the initial stiffness ($E_{bearing}$) of this backbone can be computed as:

$$F_{y,bearing} = \mu \times N_{bearing} \quad (4-19)$$

$$E_{bearing} = \frac{G \times A}{h_{rt}} \quad (4-20)$$

where μ is the coefficient of friction between the bearing and the concrete surface (taken as 0.4 in this study), $N_{bearing}$ is the axial load that is exerting on the bearing system, G is the shear modulus of the bearing pad and A is the cross-sectional area of the elastomer. $N_{bearing}$ can be obtained from the gravity analysis results.

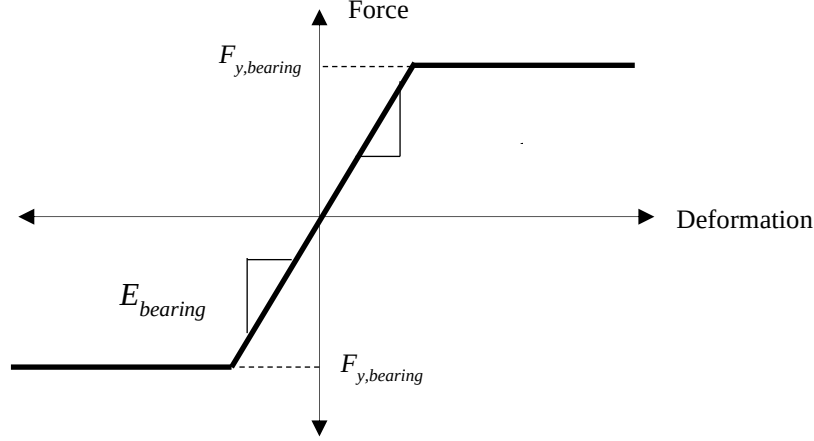


Figure 4-12: Backbone curve for the response of the elastomeric bearing systems (*elasticPP*).

The hysteretic backbone curve proposed by Megally et al. [93] is utilized for modelling the response of the shear keys, as shown in Figure 4-13. *Steel01* material definition in OpenSEES is employed to represent the behaviour as a bilinear backbone curve. Yield strength of the shear key, $F_{y,sk}$ can be calculated as [91]:

$$F_{y,sk} = \alpha \times P_{DL} \quad (4-21)$$

where α is a factor that specifies the range that $F_{y,sk}$ is allowed to vary based on the shear key design and P_{DL} is the total superstructure dead load reaction at the abutment (shear keys only exist at the abutments in the study bridge). α can be taken between 0.5-1.0 according to Caltrans [91], while for this study it is taken at its mean value of 0.75. The yield displacement value $\Delta_{y,sk}$ is assumed to be 15 mm and accordingly the initial stiffness of the shear key E_{sk} can be calculated from $F_{y,sk} / \Delta_{y,sk}$. The post-elastic slope of the bilinear curve can be taken as $b = 2.5\%$ of the initial slope (E_{sk}) per Aviram et al. [92].

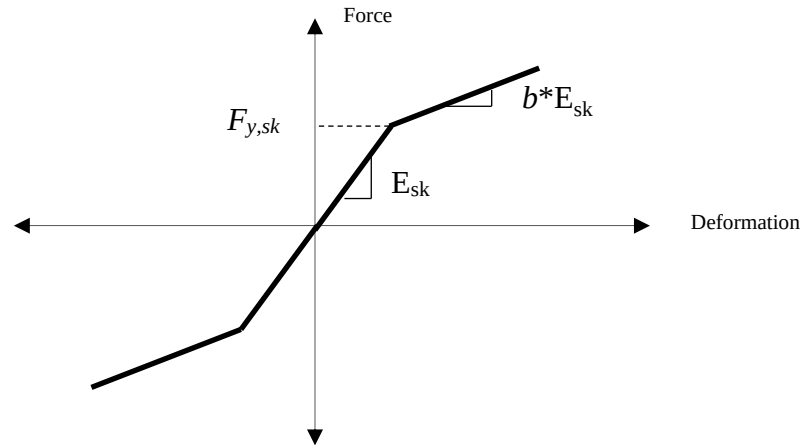


Figure 4-13: Backbone curve for the response in shear keys (*Steel01*).

4.4.4 Summary

Summary for the modeling details is presented schematically in Figure 4-14 below.

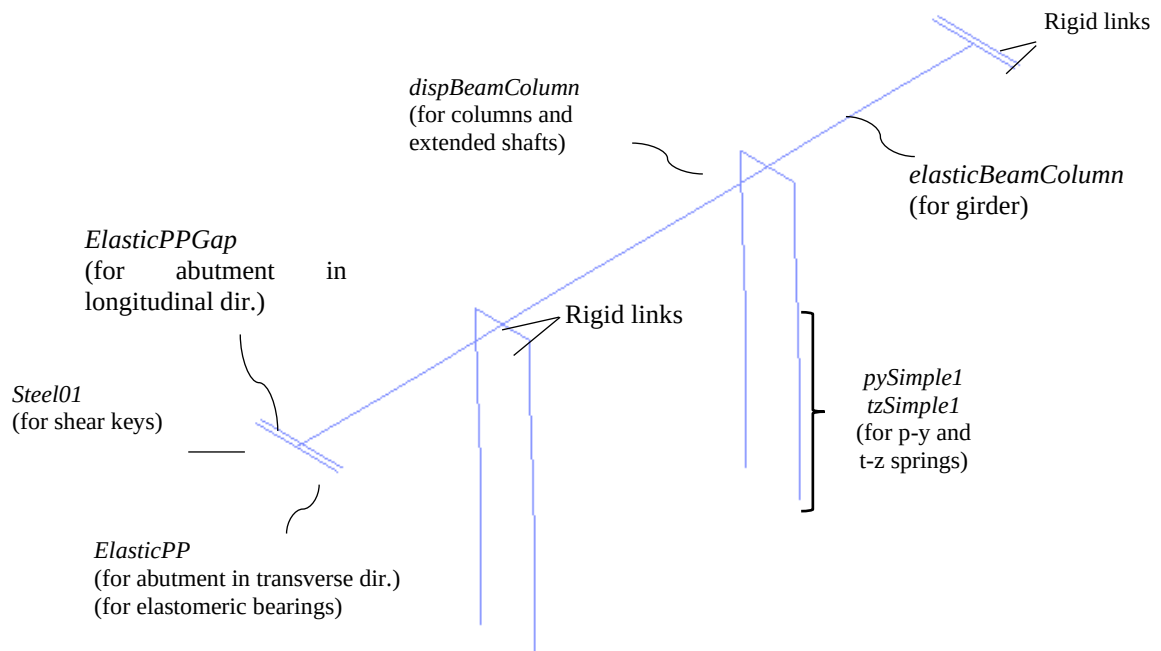


Figure 4-14: Modeling details of the study bridge.

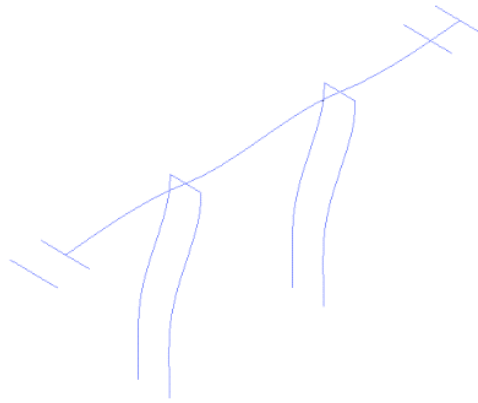
4.5 Seismic Analysis of the Bridges

The demands are obtained from the NLTH analyses of the FE models of the study bridge. In this sub-chapter, modal analysis results are presented in 4.5.1 for the first three modes of the bridge structure, and the concept of Rayleigh damping is introduced. In 4.5.2, the methodology for the NLTHA is discussed for the quantification of the earthquake direction uncertainty. 4.5.3 presents the example responses for the different levels of aging and scour in terms of longitudinal displacement recorded at the top of the column.

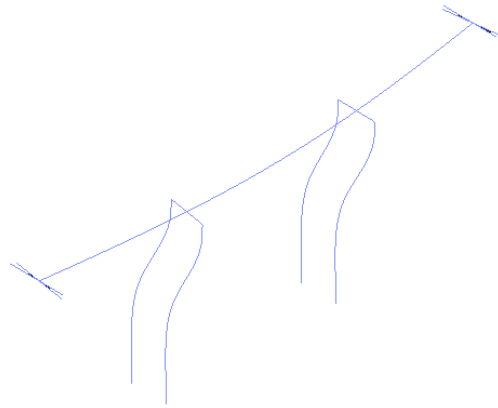
FE analyses are conducted in OpenSEES [68] software due to its diligence in implementing nonlinearity. Numerical outputs are further processed in MATLAB to obtain the maximum demands on elements and displacement histories on nodes of the FE model. For all considered cases of scour and aging, this procedure is repeated.

4.5.1 Modal Analysis

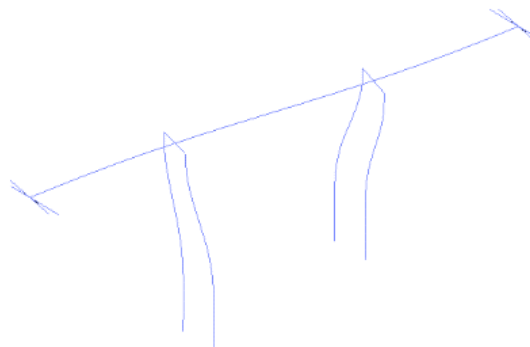
The mode shapes for the first three modes of the study bridge are presented in Figure 4-15. It is observed that the 1st mode of the study bridge is a longitudinal mode, 2nd mode is a transverse mode and the 3rd mode is a torsional mode. The periods of the first three modes for all cases of aging and scour are shown in Table 4-4. This tables shows that the increases in scour depth and bridge age results in an elongation of the modal periods. For scour case, this indicates to bridge getting more flexible due to an increase in unsupported length of the piers, which influences the lateral stiffness of the column. For aging, the increase in periods attributed to the mechanical and physical alterations in reinforcements.



(a)



(b)



(c)

Figure 4-15: Mode shapes for the (a) longitudinal, (b) transverse and (c) torsional modes of the study bridge.

Table 4-4: Modal periods for all considered cases.

Analysis No.	Scour Depth	Bridge Age	Periods (s)		
			Longitudinal	Transverse	Torsional
1	0 m	0-yr	0.820	0.724	0.373
2	0 m	25-yr	0.829	0.740	0.385
3	0 m	50-yr	0.853	0.748	0.386
4	0 m	75-yr	0.864	0.751	0.386
5	0 m	100-yr	0.869	0.753	0.386
6	1 m	0-yr	0.892	0.724	0.373
7	1 m	25-yr	0.902	0.740	0.385
8	1 m	50-yr	0.929	0.748	0.386
9	1 m	75-yr	0.940	0.751	0.386
10	1 m	100-yr	0.946	0.753	0.386
11	2 m	0-yr	0.964	0.724	0.374
12	2 m	25-yr	0.974	0.740	0.386
13	2 m	50-yr	1.003	0.748	0.386
14	2 m	75-yr	1.016	0.751	0.386
15	2 m	100-yr	1.022	0.753	0.387
16	3 m	0-yr	1.035	0.724	0.374
17	3 m	25-yr	1.045	0.740	0.386
18	3 m	50-yr	1.076	0.748	0.386
19	3 m	75-yr	1.090	0.751	0.387
20	3 m	100-yr	1.097	0.753	0.387
21	4 m	0-yr	1.105	0.724	0.374
22	4 m	25-yr	1.116	0.740	0.386
23	4 m	50-yr	1.148	0.748	0.386
24	4 m	75-yr	1.162	0.751	0.387
25	4 m	100-yr	1.169	0.753	0.387

Rayleigh damping matrix is constructed in OpenSEES [68] via introducing the mass and stiffness coefficients to the program. Rayleigh damping parameters A_0 and A_1 , referring to the mass and stiffness components, are calculated from the following equations by dictating the 1st and 3rd mode periods to have a damping ratio of 5%:

$$A_0 = \frac{\zeta_d - \frac{w_{n,j}}{w_{n,i}} * \zeta_d}{\left(\frac{0.5}{w_{n,j} - 0.5 * \left(\frac{w_{n,j}}{w_{n,i}^2} \right)} \right)} \quad (4-22)$$

$$A_1 = \zeta_d - \left(\frac{A_0}{2w_{n,i}} \right) * \left(\frac{2}{w_{n,i}} \right) \quad (4-23)$$

where ζ_d refers to the damping ratio, $w_{n,i}$ and $w_{n,j}$ refer to the circular frequency of the 1st and 3rd modes, respectively.

4.5.2 Nonlinear Time-History Analysis of the Bridge Models

NLTHA is a “step-by-step analysis” used for getting the response of a structure throughout an earthquake loading. It is considered as the most proper method for obtaining the structural demands under dynamic earthquake loading while considering the nonlinearity of the structural elements.

In this study, nonlinear time-history analyses are conducted by applying the horizontal components of the earthquake records to the three-dimensional bridge model. There is an ongoing debate about whether the inclusion of vertical ground motions in NLTHA is sensitive in terms of the seismic response or not. As a contribution to this debate, Avşar [78] conducted a sensitivity analysis on a three-dimensional bridge model by performing NLTHA with a set of 9 ground motions. NLTHA's were performed for two cases: First with the horizontal components of the ground motions (H+H), second with the horizontal components and the vertical component (H+H+V). The results obtained from this study revealed that the effect of vertical ground motion on the seismic response is limited. This study further pointed out to the increased computational demands arising from this inclusion. Thus, in this thesis, the vertical component of the ground motions is neglected in the seismic analyses.

In order to account for the uncertainty in the earthquake direction, two horizontal components, component-1 and component-2, are assigned to the longitudinal and transverse direction of the three-dimensional bridge model respectively for the first set of

analyses. For the second set, the direction assignment is reversed. In other words, component-1 is assigned to the transverse direction and the component-2 is assigned to the longitudinal direction of the bridge. As a result, two responses are obtained for the two possible combination of the ground motion components. In fragility analysis, they are considered as two separate analyses.

Flowchart for the NLTHA is presented in Figure 4-16.

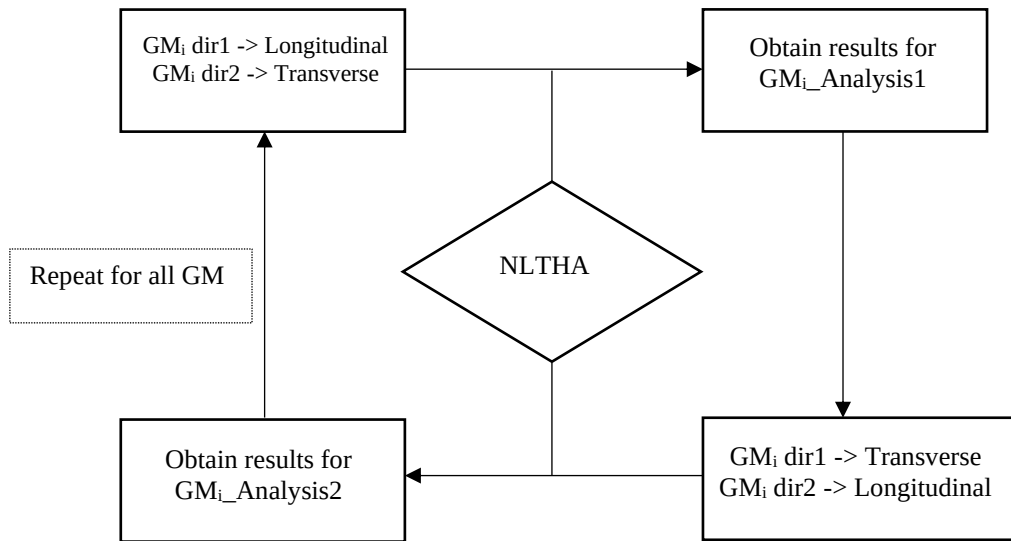
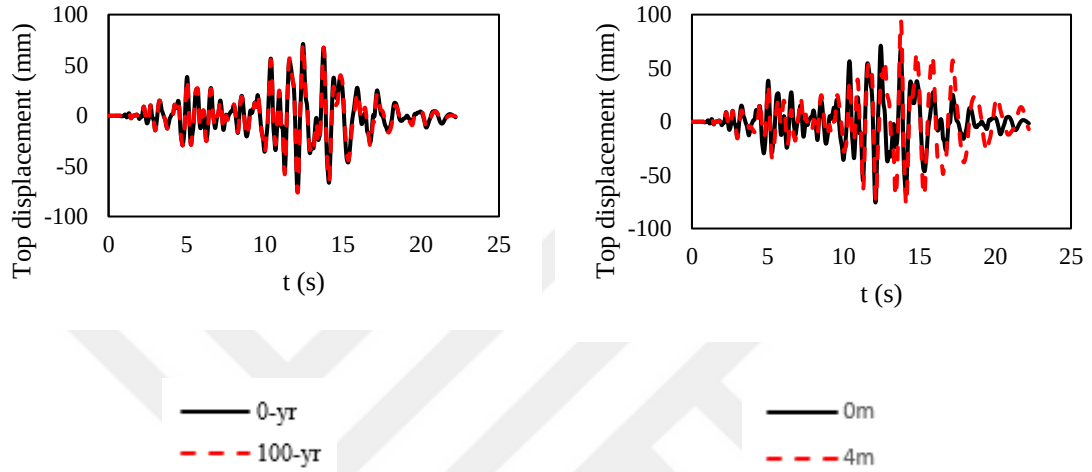


Figure 4-16: Flowchart for the NLTHA.

4.5.3 Seismic Response of the Study Bridge for Extreme Aging and Extreme Scour Case

In order to make a preliminary observation about the effects of aging and scour to the seismic response, a NLTHA is conducted for a ground motion record (RSN0727) obtained from NGA-West2 database [71]. This ground motion is scaled with a factor of two to amplify the demand levels and two horizontal components are simultaneously applied in the longitudinal and transverse direction of the bridge model. Longitudinal displacements at the topmost node of the left column of Bent-2 are recorded, and they are presented in Figure 4-17 for the extreme case of aging (100-yr) and scour (4m). It is

observed that the aging phenomenon causes a negligible increase in displacement demand. In contrast, scouring quite clearly increases the displacements in the top of the columns. Nevertheless, a better picture on the performance assessment of the bridge under different levels of scour and aging would be obtained at the end of the fragility assessment procedure.



(a) Comparison for extreme aging

(b) Comparison for extreme scour

Figure 4-17: Column top displacement response history for a ground motion record.

4.6 Probabilistic Framework for Generating Time-Dependent Fragility Curves of Bridges

There are various sources of uncertainties pertaining to the seismic vulnerability of bridges including the modeling uncertainties, the selection of the optimal intensity measure and limit state definitions. A discussion about the uncertainties were previously held in Section 3.4 and further detailed in Appendix B. Results arising from the sub-study in Chapter 3 prompted the author to resort to *Approach-1* for generating time-dependent fragility curves for ordinary highway bridges. In Section 4.6.1, *Approach-1* and the way it treats the uncertainties are revisited. Furthermore, some arguments are presented in

Section 4.6.2 with the aim of contributing to the on-going debate about the optimal intensity measure to be used in fragility curves. Lastly, the component limit states are defined and the procedure for obtaining the system fragility curves are presented in Section 4.6.3.

4.6.1 Consideration of Uncertainties

Fragility curves are generally defined with two-parameter lognormal distributions that represent the probability of exceeding a damage state under a certain intensity level [10]:

$$F(x_j, c_k, \zeta_k) = \Phi \left[\frac{\ln \left(\frac{x_j}{c_k} \right)}{\zeta_k} \right] \quad (4-24)$$

where the $F()$ denotes the probability of exceeding a damage state k of the bridge component under the ground motion j with the intensity measure x_j . c_k denotes the lognormal mean (i.e. median), and ζ_k represents the lognormal standard deviation (i.e. dispersion) of the fragility curve for the k^{th} damage state, respectively. These fragility parameters, i.e. median and dispersion, should be determined to construct a fragility curve.

For this thesis, the methodology entitled *Approach-1* in Section 3.5.1 is employed to estimate the fragility parameters. The summary of this methodology is as follows: Modeling parameters are taken at their expected (median) values to construct the bridge model. This model is further subjected to the entire ground motion set in NLTHA's. A binary damage assessment is conducted for all ground motions in the data set, using the maximum demands arising from each analyses. The results from this assessment are

further combined with the associated intensity measure of the ground motions to obtain the fragility parameters, using the Likelihood Function presented in Equation 4-25 [10]:

$$L = \prod_{j=1}^N [F(x_j; c_k; \zeta_k)]^{r_j} [1 - F(x_j; c_k; \zeta_k)]^{1-r_j} \quad (4-25)$$

in which r_j is the indicator of damage, and F is the fragility function in Equation 4-24. r_j takes the value of 1 or 0 depending on whether the damage state k is exceeded or not, respectively. Dispersion value ζ_k is assumed as 0.6 per HAZUS [69]. c_k is determined after an iterative process in which several median values are tried to reach the maximum value of L , which represents the highest probable observation. This method is widely employed in the literature for generating fragility curves for highway bridges [13] [30] [94].

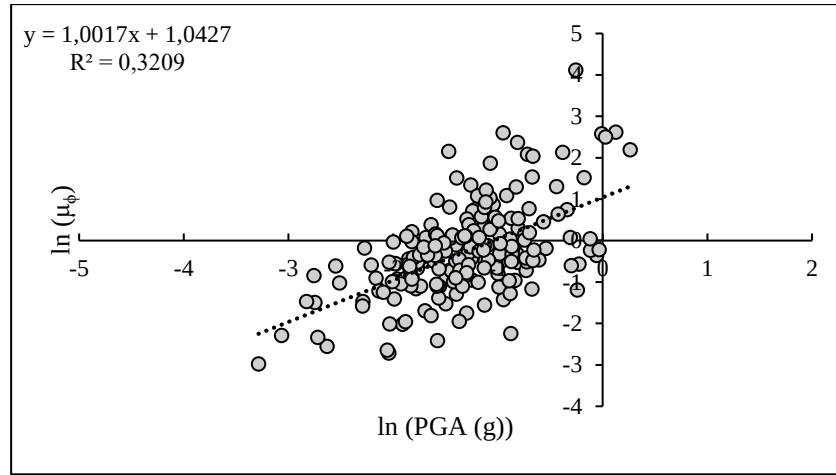
The most prominent argument against the current methodology would be about its treatment of uncertainties. In contrast, there are methods (as in the case of *Approach-2* mentioned in Chapter 3) that require the analyses to be conducted on generated “nominally identical but statistically significant” bridge samples in order to account for the probability distributions pertinent to the input parameters. The sub-study conducted in Chapter 3 revealed that the current methodology (*Approach-1*) steadily maintains 50% confidence level in fragility curves, while yielding a more conservative fragility curve compared to the one obtained from the use of PSDM (*Approach-2*), thus it is satisfactory. Another argument would be about the assumed dispersion value of $\zeta = 0.6$. It is possible to calculate the dispersion value alongside with median in MML. However, the assumption made here is observed to be satisfactory, as the dispersion value that is determined via PSDM yielded a similar value in Chapter 3. Thus, this assumption is

continued in this study and by doing so, the parameter estimation process is simplified solely into a calculation of the median value.

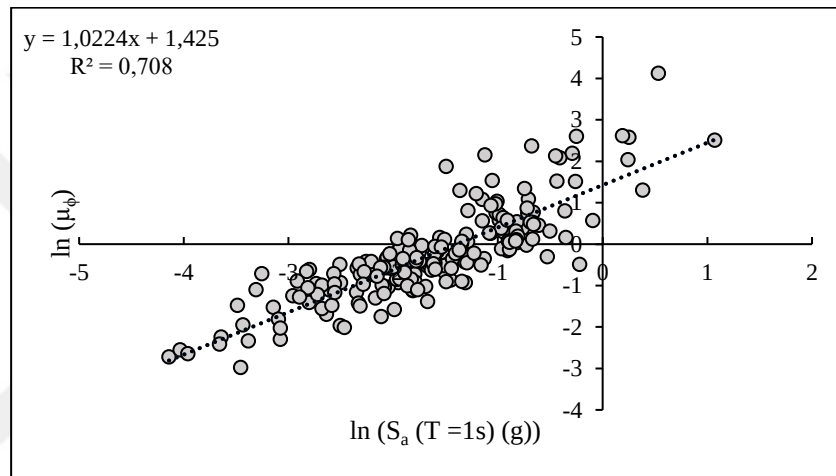
4.6.2 Seismic Intensity Measure

There is an ongoing debate about the optimal intensity measure (IM) in quantifying the seismic vulnerability of bridges. The term “optimality” has multiple avenues for discussion. As it is listed in Ramanathan [26], an optimal IM is *sufficient, efficient, hazard computable, practical* and *proficient*. “Sufficiency” is achieved by using an IM that is less dependent to the ground motion characteristics such as magnitude and distance. An “efficient” IM is the one that has a less variation in estimated demand for a given intensity level. Practicality is defined as the “dependence of the demand upon the IM”, thus it is achieved by selecting an IM which satisfies that the seismic demand is observed to be dependent of IM. Proficiency is a “measure of efficiency and practicality”. A lower dispersion value refers to a more proficient IM in the seismic fragility curves. Hazard computability refers to the IM’s representation in the hazard maps. Most hazard computable ones are presented in terms of PGA and S_a at specific periods [26].

PSDM can be utilized to quantify the optimality of an IM. Typically, lower β_D and ζ , and higher b and R^2 values indicate a more optimal IM in a PSDM [26]. In this study, PGA and S_a ($T=1s$) are compared for their sufficiency, efficiency, proficiency and practicality. PSDMs that were developed for Chapter 3 are used for this matter, and Figure 4-18 shows the two PSDMs produced for PGA and $S_a(T=1s)$. $S_a(T=1s)$ is chosen as the intensity measure of this study, due to it being more compliant to the abovementioned criteria.



(a)



(b)

Figure 4-18: PSDMs for (a) PGA (in g) and (b) $S_a(T=1s)$ (in g).

4.6.3 Component Limit States and Definition of System Fragility

Analytical fragility curves are developed with the simulated damage data. To obtain the damage levels, the demands arising from the analyses are compared with the pre-specified capacity limits. These capacity limit states are quantitatively defined by experts or as a result of experimental studies via relating them with the qualitative component damage definitions for the intermediate and ultimate damage states. These component damage states should be further managed in a way that a cohesive system damage definition that represents the global performance of the bridge is reached.

Damage can be observed in bridge columns due to excessive flexural and shear deformations. For the ordinary bridges considered herein, the columns are assumed to be in compliance with the current seismic design practices, thus the occurrence of shear failure mechanism prior to a flexural failure is prevented. Curvature ductility (μ_ϕ) is chosen as the EDP (engineering demand parameter) for the bridge columns and extended pile shafts to represent the flexural damage. It is defined as:

$$\mu_\phi = \frac{\phi_d}{\phi_y} \quad (4-26)$$

where ϕ_d stands for the curvature that is computed by using the rotation responses resulting from NLTHA and ϕ_y is the yield curvature of the RC section. ϕ_y might be calculated from built-in section analysis tools that are embedded in commercial structural analysis software, or from the analytical models that are constructed on experimental data [95]. Curvature ductility limits considered in this thesis are obtained from the values presented for the post-1990 bridges per Ramanathan [26].

The EDP to address the damage in abutments is the displacement at the superstructure level of the bridge. Longitudinal deformations in abutments might occur in active and passive directions. For the study bridge, the active deformation is ignored due to the accepted modeling approach. The threshold limits for the passive deformation $\Delta_{\text{long,passive}}$ are defined as follows: the minor and moderate damage states are specified with the yield and maximum displacement, respectively. The yield displacement is obtained from the backbone curve expressing the interaction between backwall and backfill, and the maximum displacement is assumed to be 5% of the backwall height per Shamsabadi et al. [96]. For the abutment damage the transverse direction (i.e. wingwall), the threshold limits are directly obtained from Ramanathan [26].

Seismic damage in bearings are associated with its horizontal deformations (Δ_b) in this study. For the longitudinal direction, minor damage is set to be observed after the bridge deck starts to slide. Bearing collapse is defined as the horizontal deformation where the bridge deck completely falls off from the seat. For the other damage states (i.e. moderate and extensive), the threshold limits are assumed to be evenly distributed between the two extreme limits (minor and collapse). For the transverse direction, only minor and moderate damage states are defined with the same limits as used for longitudinal direction.

Damage state for the bridge system is defined such that it is represented with the maximum damage level of all components. In other words, the system is assumed to exceed a certain damage threshold if any of the components are observed to sustain this damage level. For example, if piers are in moderate damage and the bearings are in minor damage, the system is accepted to sustain moderate damage.

The threshold limits being used herein are summarized in Table 4-5 below.

Table 4-5: Threshold limits for the bridge components.

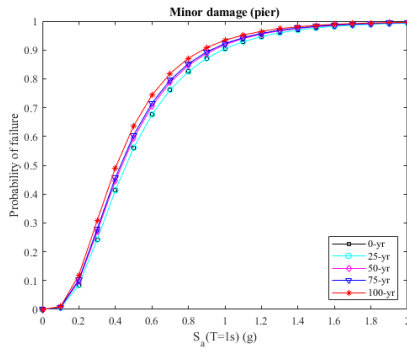
Component	EDP	Limits			
		Minor	Moderate	Extensive	Collapse
Bridge pier	Curvature ductility	1.0	4.0	8.0	12.0
Abutment	Passive def. (mm)	34.5	103.7	-	-
	Transverse def. (mm)	50.0	150.0	-	-
Elastomeric bearing	Longitudinal def. (mm)	40.0	76.0	304.5	533.0
	Transverse def. (mm)	40.0	76.0	-	-

4.7 Time-Dependent Seismic Fragility Curves of Bridges subject to Flood-Induced Scour

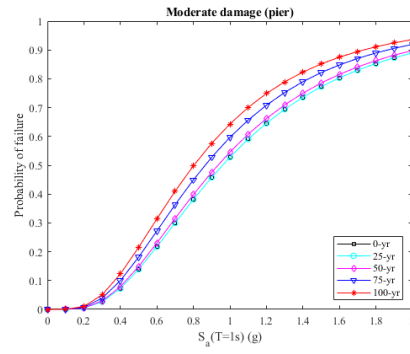
In this sub-chapter, the component and system fragility curves are presented for the study bridge. The effect of aging and scour on the bridge vulnerability is discussed by presenting the fragility curves for increasing levels of aging and scour. After that, time-dependent multi-hazard fragility surfaces are further generated for different ages of the bridge to demonstrate the combined effects of aging and scouring on bridge performance.

4.7.1 Fragility of Piers

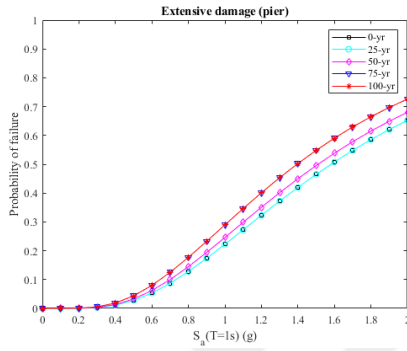
Piers are the prime target for the aging mechanisms considered herein. Figure 4-19 shows the fragility curves developed for piers (considering both the columns and extended pile shafts) at different ages of the bridge and when the scour depth is equal to 0 m (intact condition). As a general trend obtained from Figure 4-19, seismic fragility of piers increases as the bridge ages, since chloride-induced corrosion has an adverse effect on the performance of bridge piers. This effect is more pronounced at high damage states and especially after 75 years of service life. Furthermore, it is observed that the fragility curve for the pristine (0-yr) state and the one belonging to 25 years of age are overlapping. This is because the corrosion initiates in 22.7 years of the bridge service life and the rebars are still relatively untarnished at this instant of the bridge. It should be noted that similar conclusions about the effect of aging on seismic fragility of piers can be drawn for other scour depth cases. The whole spectrum of time-dependent fragility curves defined in term of the median values can be found in Appendix E.



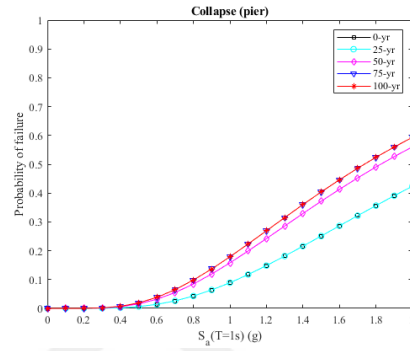
(a) Minor damage



(b) Moderate damage



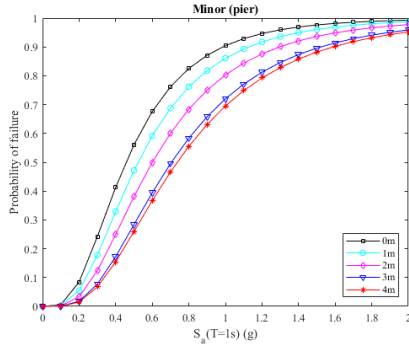
(c) Extensive damage



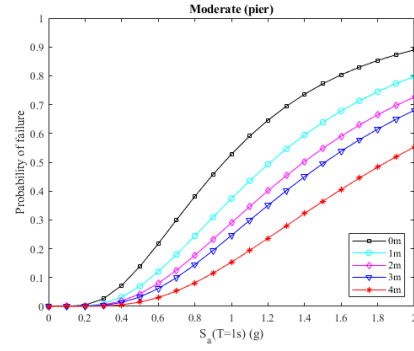
(d) Collapse state

Figure 4-19: Fragility curves of piers for different ages of the bridge (for scour depth = 0 m).

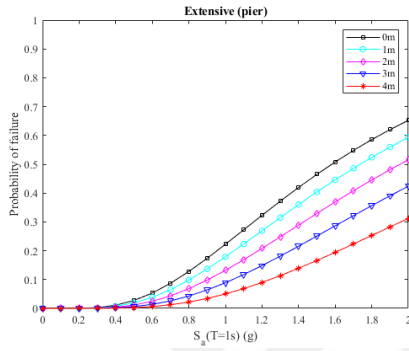
Figure 4-20 shows the fragility curves developed for piers under different scour depths and when the bridge age is equal to 0-yr (pristine condition). This figure demonstrates that scouring at pier foundations enhances the performance of the bridge piers as the scour depths are increased. This is particularly attributed to the foundation type used in the study bridge, where the columns become more flexible after an increase in the unsupported length due to scour and accordingly the reduction in curvature demands along this length.



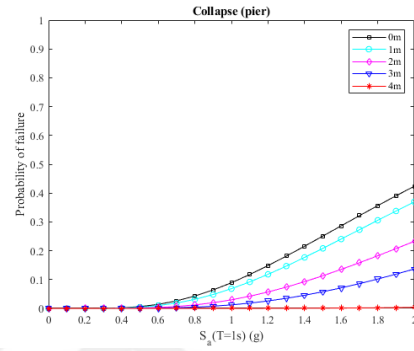
(a) Minor damage



(b) Moderate damage



(c) Extensive damage



(d) Collapse state

Figure 4-20: Fragility curves of piers for different scour depths (when bridge age = 0-yr)

4.7.2 Fragility of Bearings

Figure 4-21 shows the fragility curves of bearings at different ages of the bridge when the scour depth is equal to 0 m (intact condition). Elastomeric bearings of the study bridge are observed to stay in elastic range along the transverse direction under all ground motions. Hence, the fragility curves presented in Figure 4-21 are the ones produced from the longitudinal deformations that the elastomeric bearings undergo. Furthermore, no instance of collapse state has been encountered in the analyses. Thus, the fragility curve pertinent to collapse state does not exist on the figures below. Figure 4-21 shows that bearings are virtually unaffected by the chloride-induced corrosion in the bridge piers, especially at minor and moderate damage states. This can be attributed to the correlation

between the bearing damage and column top displacement. As previously discussed in Section 4.5.3, aging of the bridge does not cause a significant increase in top displacement. On the other hand, the aging effect creates some negative impact on bearing fragilities for extensive damage state after 75 years of age.

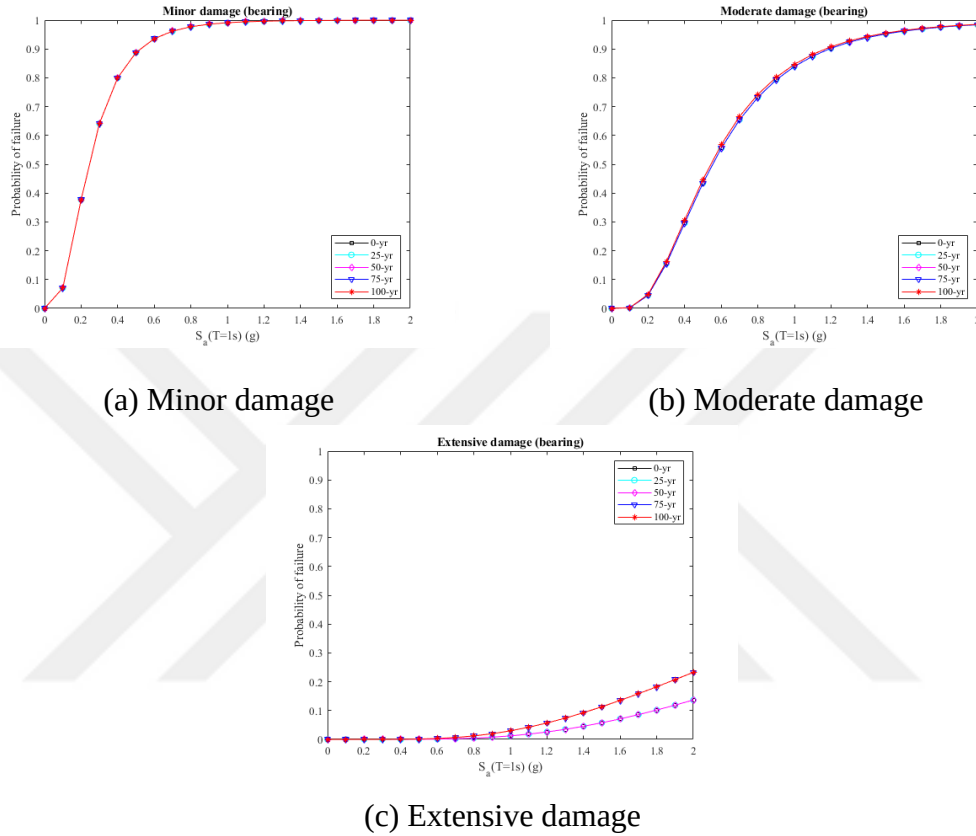


Figure 4-21: Fragility curves of bearings for different ages of the bridge (for scour depth = 0 m).

Figure 4-22 shows the fragility curves of bearings for different scour depths and when the bridge age is equal to 0-yr (pristine condition). It is clearly identified from this figure that bearing fragilities go up with increasing scour depths, as this is being more gradual at minor damage state. The reason for such outcome is that there is a significant increase in top displacement due to flood-induced scour. So, this inevitably increases the deformations in elastomeric bearings and causes damage.

Again, it is noted that in this section only the results for the scour depth being equal to 0 m and bridge age being equal to 0-yr, are demonstrated in Figures 4-21 and 4-22, respectively. But for the other cases similar outcomes are obtained as the ones shown herein.

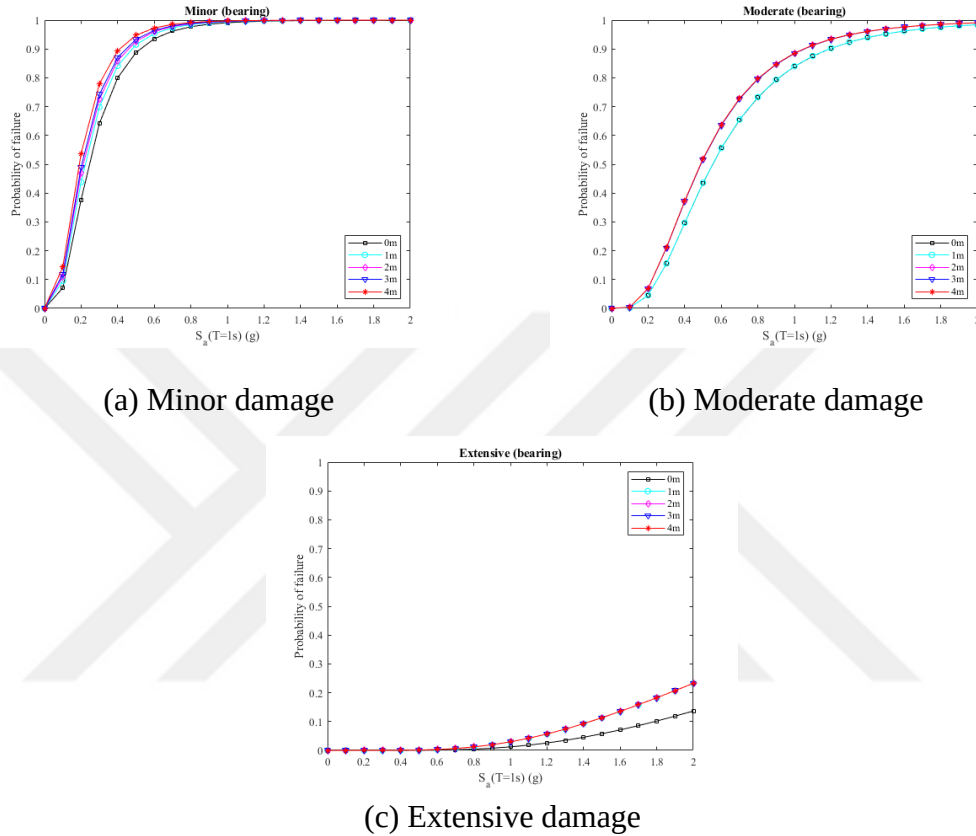


Figure 4-22: Fragility curves of bearings for different scour depths (when bridge age = 0-yr)

4.7.3 Fragility of Abutments

Damage in abutments is correlated with the superstructure level displacement as in the case of bridge bearings. Because of that, the results are similar for both the increasing aging and increasing scour cases. Fragility curves of the abutments according to the response in the longitudinal and transverse directions are presented in Figure 4-23 and Figure 4-24, respectively for different ages of the bridge and when the scour depth is

equal to 0 m. It can be said that bridge aging due to the deterioration in piers does not make a significant influence on the fragilities of abutments.

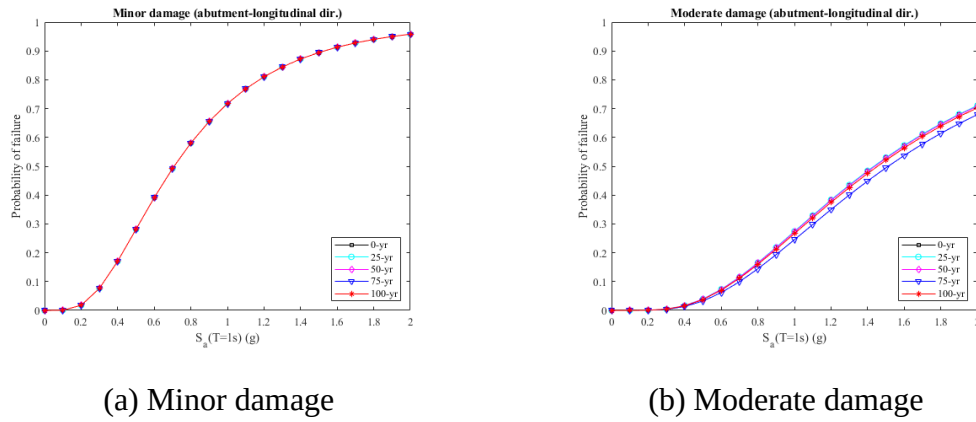


Figure 4-23: Fragility curves of the abutments in longitudinal direction for different ages of the bridge (for scour depth = 0 m).

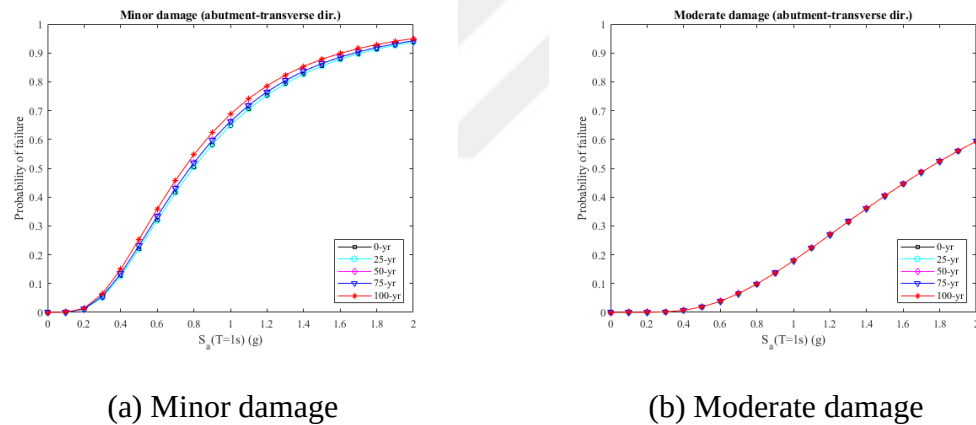


Figure 4-24: Fragility curves of the abutments in transverse direction for different ages of the bridge (for scour depth = 0 m).

Fragility curves of the abutments according to the response in the longitudinal and transverse directions are presented in Figure 4-25 and Figure 4-26, respectively for different scour depths and when the bridge age is equal to 0-yr. As can be observed from these figures, abutment fragilities are mostly getting increased with the increasing levels

of scour depths, which is associated with the greater top displacement demands as the bridge get more flexible.

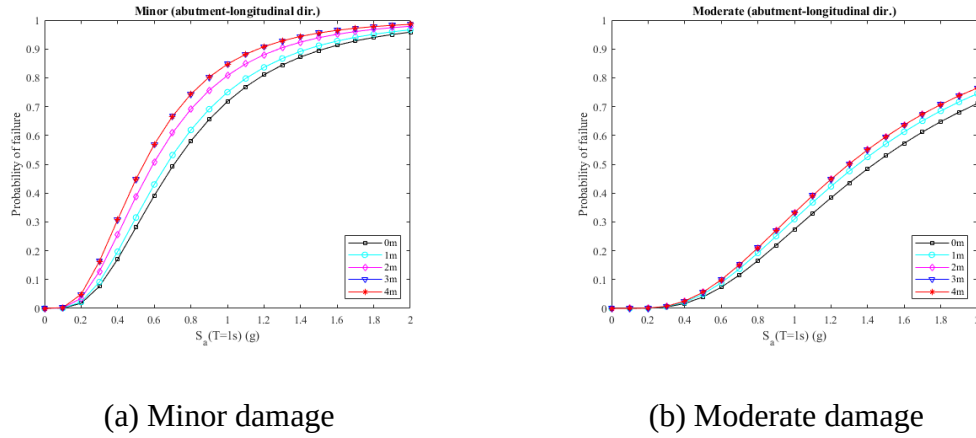


Figure 4-25: Fragility curves of the abutments in longitudinal direction for different scour depths (when bridge age = 0-yr).

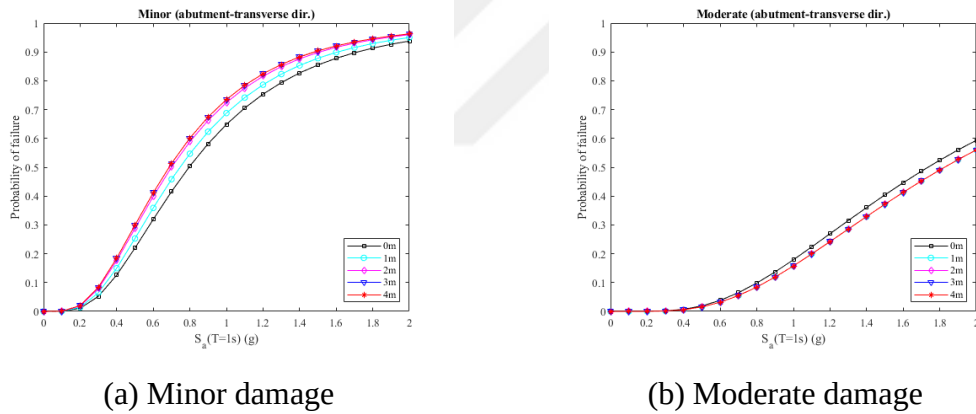


Figure 4-26: Fragility curves of the abutments in transverse direction for different scour depths (when bridge age = 0-yr).

4.7.4 System Fragility Curves for Different Aging and Scour Levels

System fragilities are generated by projecting the maximum component damage level to the system level. It is observed that the fragilities at lower damage states (minor and moderate) are controlled by bearing and abutment, whereas the higher damage states (extensive and collapse) are exclusively controlled by the damage in piers. This results in

an increase in system vulnerability for extensive damage and collapse states due to the aging in piers, especially after 75 years of service life, as shown in Figure 4-27, which presents the system fragility curves for different ages of the bridge and when the scour depth is equal to 0 m.

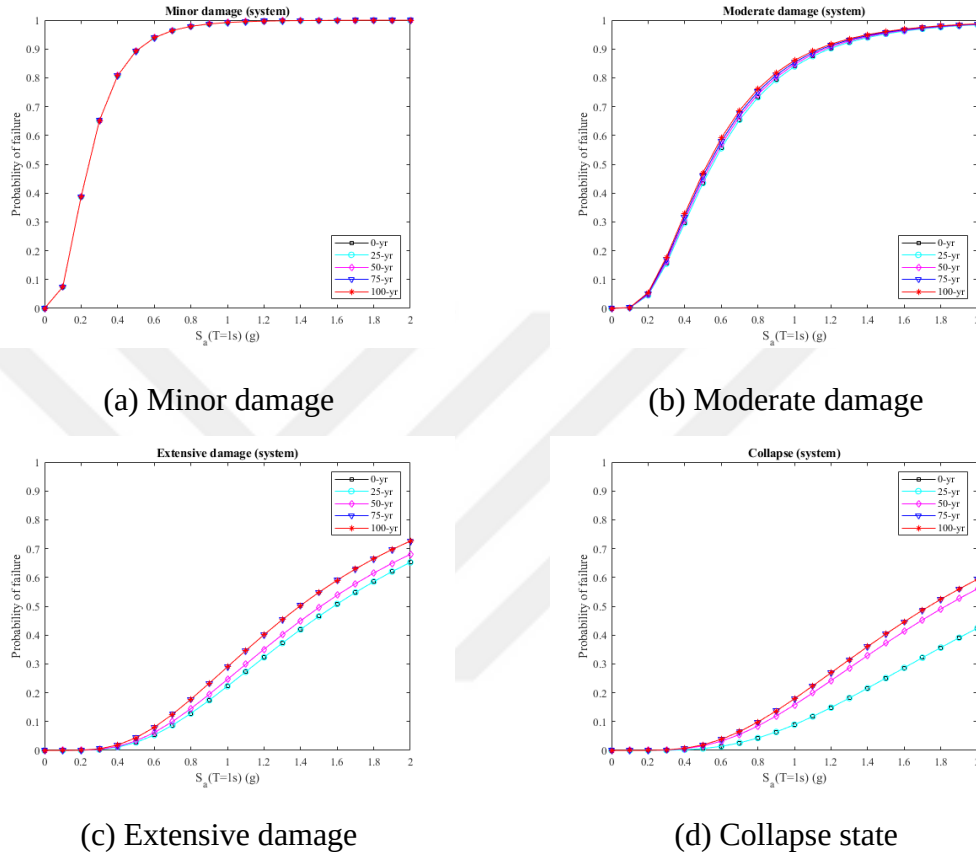


Figure 4-27: System fragility curves for different ages of the bridge (for scour depth = 0 m).

In contrast, as the scour increases, there is a subsequent increase in vulnerability at minor and moderate damage states due to a rise in demand levels in bearings and abutments, whereas in extensive and collapse damages there is an increase in performance due to decreased damage levels in piers, particularly due to the type of foundation used in the study bridge. This outcome can be observed from Figure 4-28, which shows the

system fragility curves for different scour depths and when the bridge age is equal to 0-yr.

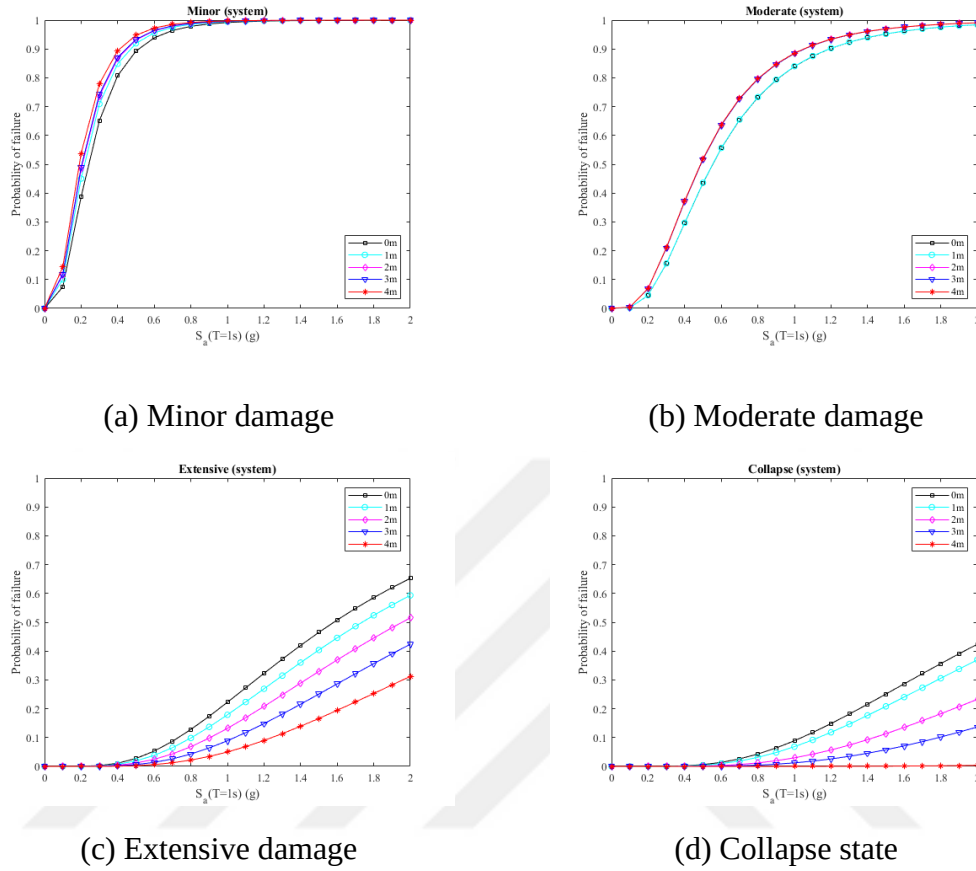


Figure 4-28: System fragility for different scour depths (when bridge age = 0-yr).

4.7.5 Fragility Surfaces

Surface plots representing the probability of exceedance under varying seismic intensity levels and different scour depths (i.e. 0, 1, 2, 3 and 4 m) are presented below in Figures 4-29 to 4-33 for several ages of the bridge (0, 25, 50, 75 and 100-yr). They are generated based on the fragility curves derived for the system level (presented in Section 4.7.4). These “fragility surfaces” are useful for decision makers in calculating the multi-hazard risk for a given service stage (the combination of bridge age and scour depth existing at the bridge site) of the ordinary highway bridges. The median values of the

ismic fragility curves for all combinations of aging and scour are presented in Appendix E.

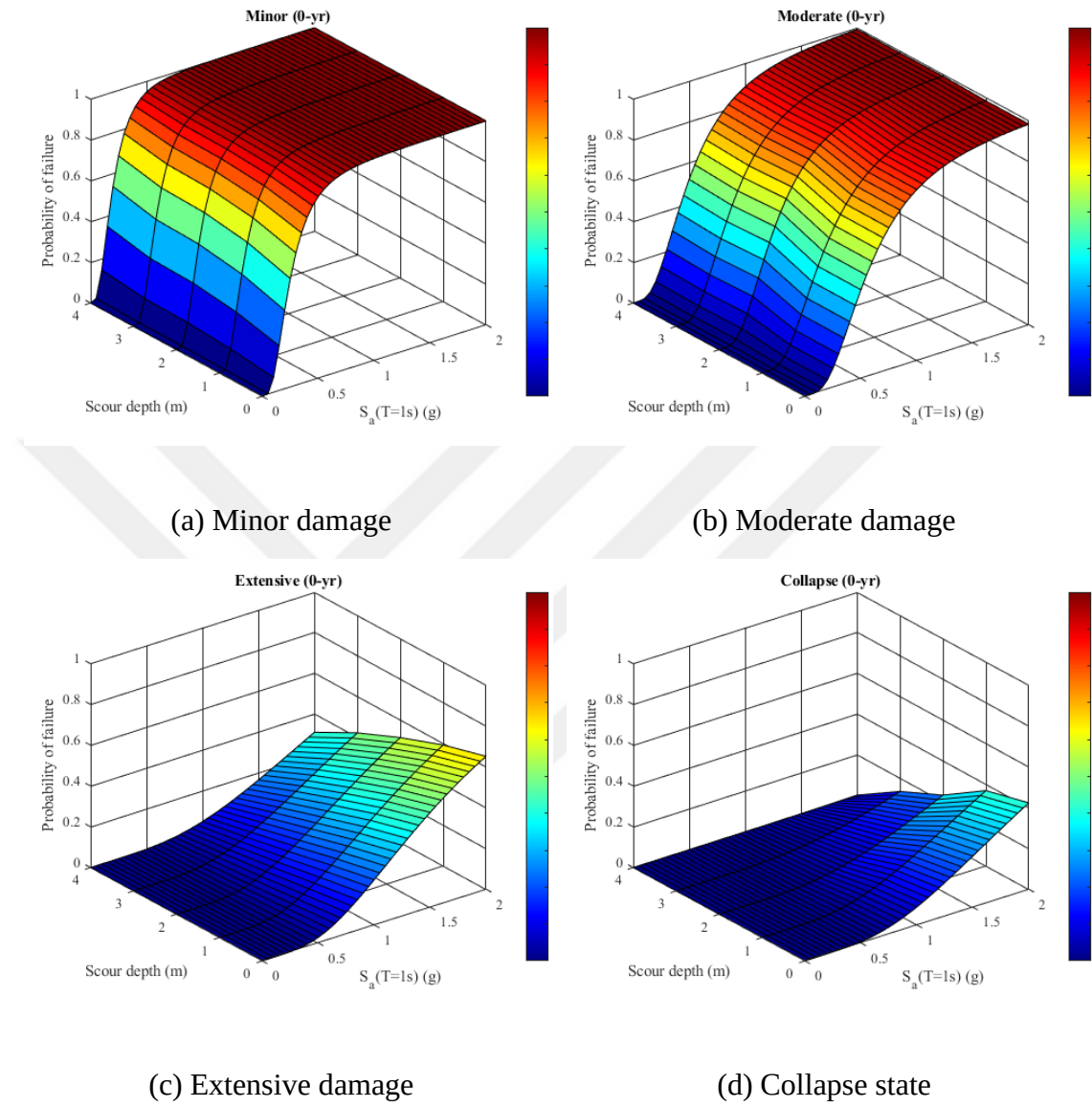
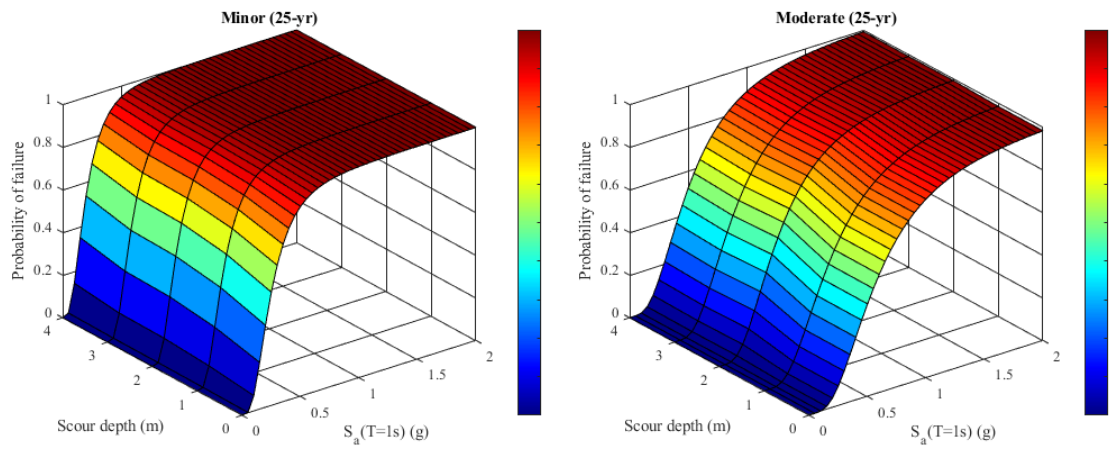
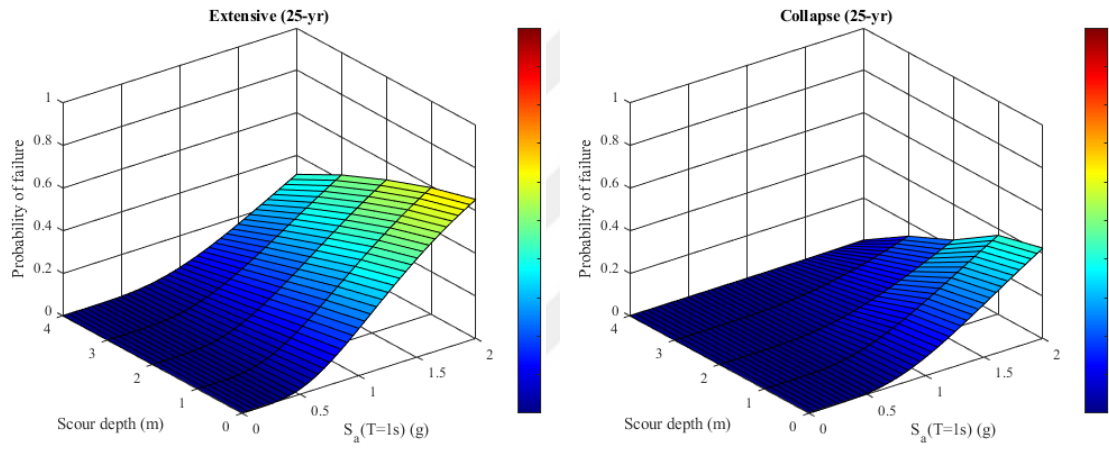


Figure 4-29: Multi-hazard fragility surfaces for the pristine (0-yr) state of the bridge.



(a) Minor damage

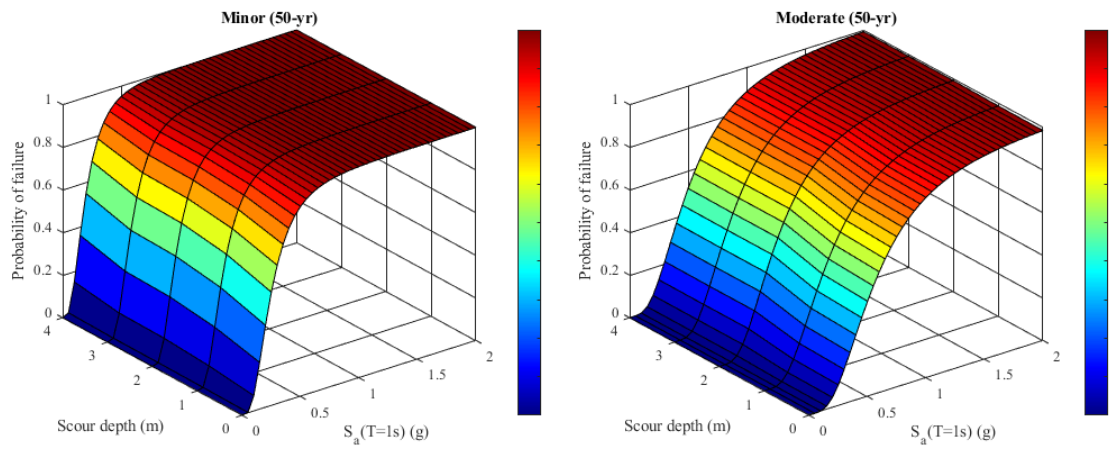
(b) Moderate damage



(c) Extensive damage

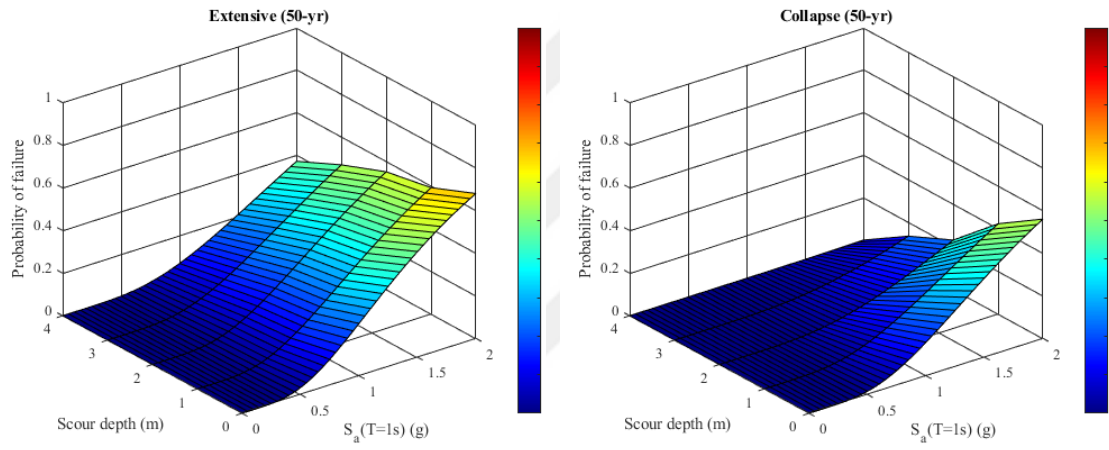
(d) Collapse state

Figure 4-30: Multi-hazard fragility surfaces for the 25-yr old bridge.



(a) Minor damage

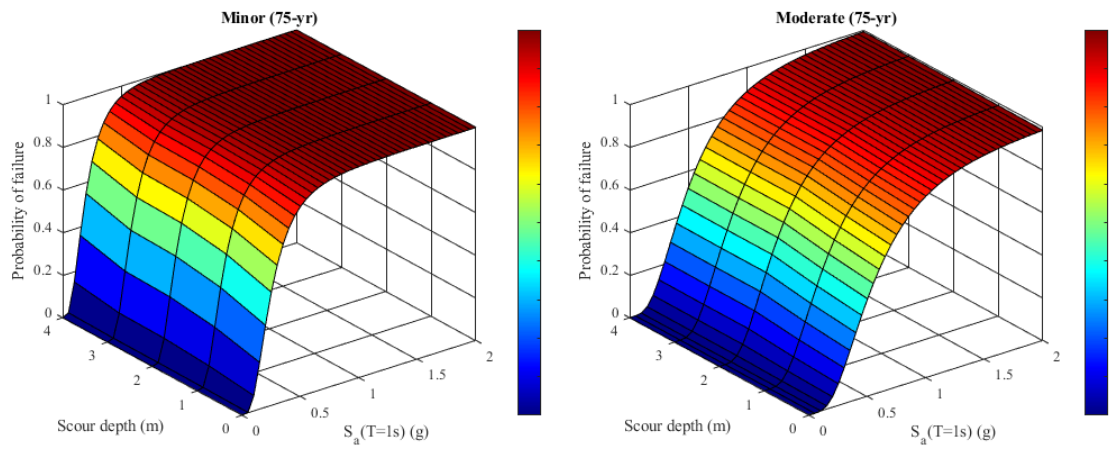
(b) Moderate damage



(c) Extensive damage

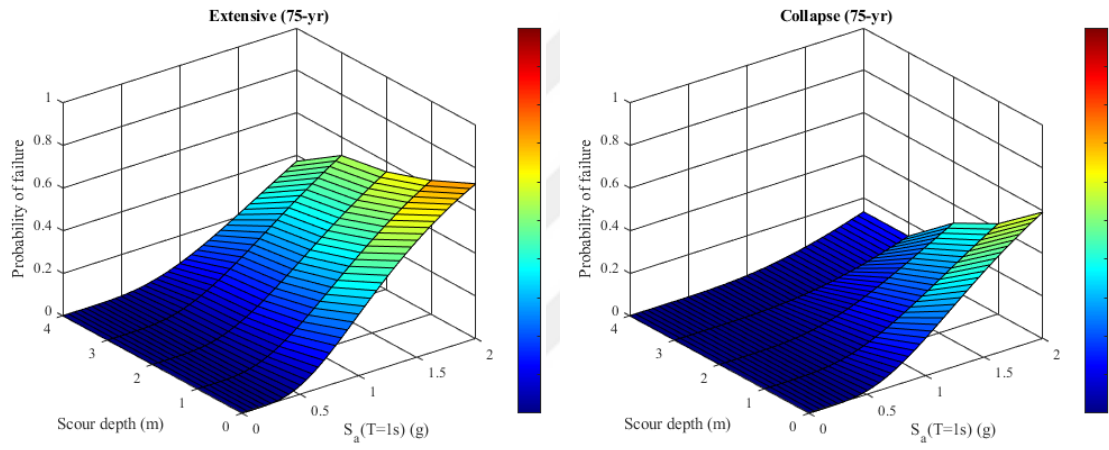
(d) Collapse state

Figure 4-31: Multi-hazard fragility surfaces for the 50-yr old bridge.



(a) Minor damage

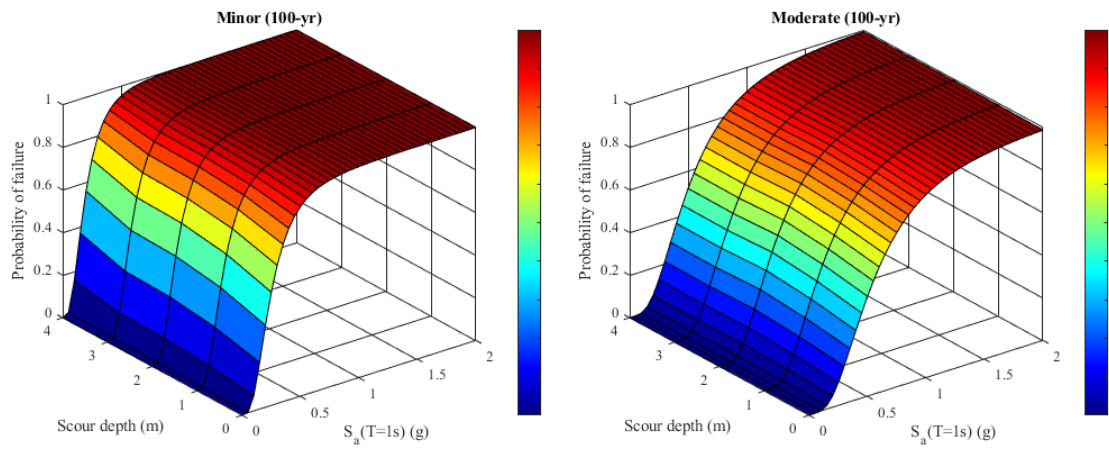
(b) Moderate damage



(c) Extensive damage

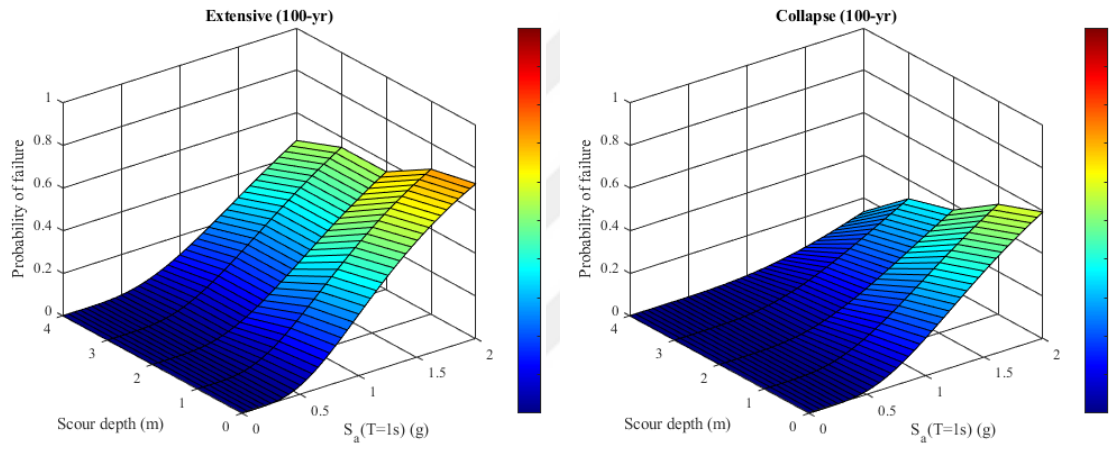
(d) Collapse state

Figure 4-32: Multi-hazard fragility surfaces for the 75-yr old bridge.



(a) Minor damage

(b) Moderate damage



(c) Extensive damage

(d) Collapse state

Figure 4-33: Multi-hazard fragility surfaces for the 100-yr old bridge.

CHAPTER V

CONCLUSIONS

5.1 *Summary of the Thesis*

Ordinary highway bridges, being the most vulnerable elements to earthquakes in a highway system, are subject to aging over time due to various environmental degradation mechanisms. Furthermore, river-crossing bridges are subject to flood-related problems, most notably against the scouring phenomenon. Bridges in Turkey, both being located in a seismically-active region and having been embraced by an immense web of rivers, may be of subject to both of earthquake and flood hazards. To investigate the combined effects of these and to fill the present research gap, this thesis is dedicated to generating time-dependent multi-hazard fragility curves for ordinary aging highway bridges that is subjected to flood-induced scour.

To achieve that, first a side study is conducted to evaluate different practices in developing seismic fragility curves and to choose the optimal methodology. In this study, fragility curves are generated for a generic bridge pier with two most used methods (MML and PSDM). Results have shown that these two methods are in close agreement at lower damage levels, whereas at higher damage levels they are observed to be quite deviated from each other. This is attributed to the ground motion set being limited. However, increasing the number of records in the ground motion set would increase the computational expenses exponentially. MML is observed to yield more conservative results than PSDM at higher damage levels, thus judged as more plausible. Then,

confidence intervals for the pier fragility curves are computed to compare the confidence levels with respect to the curves generated by these two methods. Results of this assessment revealed that the fragility curves generated with MML maintain a solid 50% confidence level for all damage states, whereas the curves generated via PSDM sways toward the lower confidence levels at higher damage states. Results obtained from this study prompted the author to use MML in time-dependent multi-hazard fragility analyses for this thesis. The details are presented in Chapter 3.

There are various aging mechanisms that may affect the ordinary highway bridges. In literature, analytical models for the chloride-induced corrosion deterioration in bridge piers and thermal oxidation phenomenon in elastomeric rubber bearings are presented. To see the individual and combined significance of these effects, a preliminary study by the author was conducted on a three-dimensional generic highway bridge model. Reduction in longitudinal and transverse reinforcement is projected into the FE models for chloride-induced corrosion in piers, along with the increase in shear stiffness of the elastomeric bearing pads due to thermal oxidation. Results of this separate study have shown that the deterioration in bearing could be negligible in general, only being slightly pronounced at lower damage levels for the early ages of the bridge. Furthermore, the effect of chloride-induced corrosion in piers is observed to be influential towards the end of the bridge service life, especially at higher damage levels. These results prompted the author to project the primary and secondary effects of chloride-induced corrosion to the pier fiber models in generating time-dependent seismic fragility curves for ordinary highway bridges within the scope of the current thesis.

For the main analyses, the ordinary highway bridge that represents the characteristics of highway bridges in Turkey is devised from the literature. Then, a ground motion set with a moderate-to-high seismicity is assembled. For 5 different scour levels

(0, 1, 2, 3 and 4 m), nonlinear time-history analysis analyses are conducted for the 5 distinct ages of the bridge (0, 25, 50, 75 and 100-yr). By using the damage data obtained from these analyses, seismic fragility curves at component and system levels are generated via MML for all combinations of scour depth and bridge age. Then, the system fragility curves are considered to produce the resultant fragility surfaces. The outcomes obtained in this thesis can be beneficial for decision makers and bridge designers for evaluating the vulnerability of aging ordinary highway bridges subjected to different levels of pier deterioration and scouring.

5.2 Major Outcomes

This thesis is clustered around three sub-studies: (i) Evaluation of fragility parameter estimation methods, (ii) Evaluation of aging mechanisms for ordinary bridges, (iii) Time-dependent fragility assessment of aging ordinary highway bridges subject to flood-induced scour. Key findings for these are listed below.

(i) In evaluation of the fragility parameter estimation methods:

- MML and PSDM are in close agreement at lower damage levels, whereas they present different results for higher damage states. MML is more conservative than PSDM for extensive and collapse damage states.
- Fragility curves generated via MML maintain 50% confidence level among all damage states, whereas the curves generated via PSDM sway towards the lower confidence levels at extensive and collapse damage states.
- The fragility dispersion value of 0.6 is a reasonable assumption, as it is in close agreement with the dispersion value yielded by PSDM method.

(ii) In evaluation of the different aging mechanisms for ordinary bridges:

- Chloride-induced corrosion in bridge piers causes significant alterations in the seismic performance of bridges, especially after 75 years of service life.
- Thermal oxidation in bearings have a negligible effect on the seismic vulnerability of bridges, and its effect is limited to the lower damage states in the early ages of the bridge.

(iii) In time-dependent seismic fragility assessment ordinary highway bridges subject to flood-induced scour:

- Aging effects are mostly apparent in the component fragility of the bridge piers, especially adversely affecting after 75 years of service at the higher damage levels. Performance in bearings and abutments are virtually unscathed by chloride-induced corrosion in piers.
- Effects of scour are well pronounced in all bridge components. Bearing and abutment fragility increase as the scour depth increases, especially at lower damage levels, due to a rise in displacement demand at the top of the bridge piers. Contrary to this, vulnerability of bridge piers decrease as the scour depth increases, particularly due to the foundation type of the study bridge used in the current study. This is attributed to the flexibility gained from the removal soils in the column base.
- In the system level, seismic fragilities at the lower damage states are dominated by the performance of bearings and abutments, whereas the higher damage states are controlled by the seismic vulnerability of bridge piers.

5.3 Recommendations for Future Studies

The procedures introduced in this thesis might be repeated for different types of superstructure and foundation types, such as continuous I-girder bridges and with pile group foundations. Furthermore, several more deterioration mechanisms might be implemented to the FE models such as sulphate attack or alkali-aggregate reaction, given that an analytical model is present for such deterioration processes. Parameter estimation methods might be evaluated on three-dimensional bridge models in the context of performance and seismic fragility assessment, with the inclusion of several other fragility methods. Lastly, this study might be extended to include more scour depths.

5.4 Limitations

The vulnerability assessment conducted here is limited to the context of fragility curves obtained from the simulations conducted upon a characteristic highway bridge. Furthermore, the ground motion set employed here is developed to represent the seismicity of Turkey region, even though it is broadband and thorough.

APPENDIX A

DETAILS OF THE GROUND MOTION DATA SET DEVELOPED FOR EVALUATION OF FRAGILITY METHODOLOGIES

Ground motion data set utilized in Chapter 3 are summarized in Table A.1 below, with its properties and the corresponding PGA and $S_a(T=1)$ values.

Table A-1: Ground motion properties for the set developed for Chapter 3.

GM_no	GM_name	GM_dir	Scale	PGA (g)	$S_a(T=1s)$ (g)
1	RSN0030	085	1	0.44	0.19
2	RSN0030	355	1	0.37	0.16
3	RSN0031	050	1	0.25	0.17
4	RSN0031	320	1	0.27	0.15
5	RSN0033	205	1	0.36	0.21
6	RSN0033	295	1	0.27	0.12
7	RSN0134	000	1	0.42	0.03
8	RSN0134	180	1	0.13	0.01
9	RSN0144	000	1	0.22	0.05
10	RSN0144	180	1	0.29	0.07
11	RSN0145	160	1	0.14	0.10
12	RSN0145	250	1	0.25	0.14

Table A-1: Ground motion properties for the set developed for Chapter 3 (Cont'd).

13	RSN0151	150	1	0.04	0.04
14	RSN0151	240	1	0.05	0.05
15	RSN0152	067	2	0.2	0.12
16	RSN0152	337	2	0.23	0.16
17	RSN0154	213	2	0.22	0.15
18	RSN0154	303	2	0.21	0.19
19	RSN0164	147	1	0.17	0.10
20	RSN0164	237	1	0.16	0.31
21	RSN0171	000	1	0.32	0.42
22	RSN0171	270	1	0.3	0.37
23	RSN0221	270	1	0.25	0.03
24	RSN0221	360	1	0.21	0.07
25	RSN0222	265	1	0.18	0.10
26	RSN0222	355	1	0.26	0.14
27	RSN0231	000	1	0.43	0.21
28	RSN0231	090	1	0.27	0.17
29	RSN0232	254	1	0.32	0.09
30	RSN0232	344	1	0.24	0.12
31	RSN0235	254	1	0.39	0.10
32	RSN0235	344	1	0.44	0.15
33	RSN0236	090	1	0.23	0.19
34	RSN0236	180	1	0.19	0.09
35	RSN0239	000	1	0.48	0.16

Table A-1: Ground motion properties for the set developed for Chapter 3 (Cont'd).

36	RSN0239	090	1	0.19	0.06
37	RSN0240	090	1	0.37	0.15
38	RSN0240	180	1	0.48	0.10
39	RSN0249	000	1	0.38	0.10
40	RSN0249	090	1	0.33	0.20
41	RSN0250	000	1	0.95	0.25
42	RSN0250	090	1	0.41	0.56
43	RSN0320	090	1	0.16	0.18
44	RSN0320	180	1	0.15	0.19
45	RSN0445	059	1	0.15	0.05
46	RSN0445	329	1	0.07	0.03
47	RSN0448	250	1	0.42	0.14
48	RSN0448	340	1	0.29	0.46
49	RSN0450	220	1	0.08	0.12
50	RSN0450	310	1	0.11	0.19
51	RSN0451	195	1	0.71	0.65
52	RSN0451	285	1	1.3	1.11
53	RSN0461	150	1	0.16	0.17
54	RSN0461	240	1	0.31	0.43
55	RSN0547	270	1	0.27	0.17
56	RSN0547	360	1	0.24	0.14
57	RSN0548	270	1	0.21	0.17
58	RSN0548	360	1	0.18	0.32

Table A-1: Ground motion properties for the set developed for Chapter 3 (Cont'd).

59	RSN0549	180	1	0.25	0.25
60	RSN0549	270	1	0.18	0.28
61	RSN0558	270	1	0.45	0.41
62	RSN0558	360	1	0.4	0.61
63	RSN0598	262	1	0.13	0.02
64	RSN0598	352	1	0.21	0.03
65	RSN0619	060	1	0.37	0.16
66	RSN0619	330	1	0.48	0.33
67	RSN0724	45	1	0.14	0.14
68	RSN0724	135	1	0.2	0.18
69	RSN0725	270	1	0.47	0.35
70	RSN0725	360	1	0.29	0.37
71	RSN0727	045	1	0.58	0.19
72	RSN0727	135	1	0.84	0.69
73	RSN0733	000	1	0.17	0.27
74	RSN0733	090	1	0.14	0.19
75	RSN0739	250	1	0.25	0.22
76	RSN0739	340	1	0.24	0.17
77	RSN0740	250	1	0.06	0.13
78	RSN0740	340	1	0.08	0.11
79	RSN0752	000	1	0.51	0.50
80	RSN0752	090	1	0.44	0.30
81	RSN0754	195	1	0.16	0.16

Table A-1: Ground motion properties for the set developed for Chapter 3 (Cont'd).

82	RSN0754	285	1	0.18	0.17
83	RSN0785	000	1	0.16	0.40
84	RSN0785	090	1	0.1	0.30
85	RSN0787	270	1	0.19	0.58
86	RSN0787	360	1	0.28	0.53
87	RSN0811	000	1	0.37	0.55
88	RSN0811	090	1	0.65	0.60
89	RSN0821	000	1	0.5	0.66
90	RSN0821	180	1	0.39	0.89
91	RSN0832	000	1	0.11	0.19
92	RSN0832	090	1	0.15	0.23
93	RSN0838	000	1	0.13	0.23
94	RSN0838	090	1	0.14	0.20
95	RSN0848	000	1	0.28	0.21
96	RSN0848	180	1	0.42	0.38
97	RSN0850	000	1	0.17	0.25
98	RSN0850	090	1	0.15	0.36
99	RSN0864	000	1	0.27	0.41
100	RSN0864	090	1	0.28	0.54
101	RSN0881	045	1	0.22	0.27
102	RSN0881	135	1	0.16	0.18
103	RSN0901	270	1	0.48	0.12
104	RSN0901	360	1	0.54	0.20

Table A-1: Ground motion properties for the set developed for Chapter 3 (Cont'd).

105	RSN0942	090	2	0.2	0.27
106	RSN0942	360	2	0.16	0.15
107	RSN0949	090	1	0.35	0.54
108	RSN0949	360	1	0.31	0.30
109	RSN0951	220	1	0.1	0.11
110	RSN0951	310	1	0.06	0.17
111	RSN0953	009	1	0.44	1.07
112	RSN0953	279	1	0.49	0.95
113	RSN0963	090	1	0.57	0.54
114	RSN0963	360	1	0.51	1.00
115	RSN0974	177	1	0.37	0.06
116	RSN0974	267	1	0.21	0.06
117	RSN0986	195	1	0.19	0.21
118	RSN0986	285	1	0.16	0.28
119	RSN1139	076	1	0.07	0.03
120	RSN1139	346	1	0.06	0.04
121	RSN1141	090	1	0.33	0.64
122	RSN1141	180	1	0.28	0.36
123	RSN1148	000	1	0.21	0.12
124	RSN1148	090	1	0.13	0.13
125	RSN1159	090	2	0.2	0.44
126	RSN1159	180	2	0.17	0.52
127	RSN1160	000	1	0.19	0.13

Table A-1: Ground motion properties for the set developed for Chapter 3 (Cont'd).

128	RSN1160	090	1	0.16	0.06
129	RSN1166	090	2	0.25	0.58
130	RSN1166	180	2	0.18	0.37
131	RSN1612	090	1	0.14	0.08
132	RSN1612	180	1	0.15	0.09
133	RSN1614	090	1	0.13	0.08
134	RSN1614	180	1	0.1	0.15
135	RSN1617	090	1	0.51	0.13
136	RSN1617	180	1	0.89	0.14
137	RSN1619	000	2	0.24	0.12
138	RSN1619	090	2	0.12	0.07
139	RSN2703	090	1	0.21	0.09
140	RSN2703	180	1	0.12	0.09
141	RSN2708	180	2	0.16	0.41
142	RSN2708	270	2	0.21	0.38
143	RSN2734	090	1	0.32	0.63
144	RSN2734	180	1	0.34	0.33
145	RSN4101	090	1	0.52	0.18
146	RSN4101	360	1	0.8	0.13
147	RSN4111	090	1	0.23	0.31
148	RSN4111	360	1	0.25	0.25
149	RSN4122	090	1	0.79	0.13
150	RSN4122	360	1	0.43	0.12

Table A-1: Ground motion properties for the set developed for Chapter 3 (Cont'd).

151	RSN0030	085	2	0.89	0.37
152	RSN0030	355	2	0.74	0.31
153	RSN0164	147	2	0.34	0.20
154	RSN0164	237	2	0.31	0.61
155	RSN0221	270	2	0.51	0.06
156	RSN0221	360	2	0.42	0.15
157	RSN0240	090	2	0.74	0.30
158	RSN0240	180	2	0.97	0.21
159	RSN0320	090	2	0.32	0.36
160	RSN0320	180	2	0.3	0.39
161	RSN0450	220	2	0.16	0.24
162	RSN0450	310	2	0.22	0.39
163	RSN0548	270	2	0.42	0.33
164	RSN0548	360	2	0.36	0.64
165	RSN0549	180	2	0.5	0.50
166	RSN0549	270	2	0.35	0.56
167	RSN0598	262	2	0.26	0.04
168	RSN0598	352	2	0.41	0.05
169	RSN0733	000	2	0.34	0.54
170	RSN0733	090	2	0.28	0.37
171	RSN0754	195	2	0.32	0.32
172	RSN0754	285	2	0.36	0.35
173	RSN0785	000	2	0.32	0.79

Table A-1: Ground motion properties for the set developed for Chapter 3 (Cont'd).

174	RSN0785	090	2	0.21	0.59
175	RSN0821	000	2	0.99	1.31
176	RSN0821	180	2	0.77	1.78
177	RSN0832	000	2	0.23	0.39
178	RSN0832	090	2	0.29	0.46
179	RSN0838	000	2	0.26	0.47
180	RSN0838	90	2	0.27	0.39
181	RSN0850	0	2	0.34	0.49
182	RSN0850	90	2	0.31	0.72
183	RSN0881	45	2	0.45	0.54
184	RSN0881	135	2	0.33	0.37
185	RSN0963	90	2	1.14	1.09
186	RSN0963	360	2	1.03	1.99
187	RSN0986	195	2	0.37	0.43
188	RSN0986	285	2	0.33	0.55
189	RSN1148	0	2	0.42	0.24
190	RSN1148	90	2	0.27	0.25
191	RSN1160	0	2	0.38	0.25
192	RSN1160	90	2	0.32	0.12
193	RSN1612	90	2	0.27	0.16
194	RSN1612	180	2	0.3	0.19
195	RSN1614	90	2	0.26	0.17
196	RSN1614	180	2	0.2	0.31

Table A-1: Ground motion properties for the set developed for Chapter 3 (Cont'd).

197	RSN2703	90	2	0.41	0.17
198	RSN2703	180	2	0.25	0.19
199	RSN2734	90	2	0.64	1.27
200	RSN2734	180	2	0.68	0.66



APPENDIX B

SENSITIVITY STUDY FOR THE MODELING PARAMETERS PERTINENT TO ORDINARY HIGHWAY BRIDGES

Uncertain parameters pertaining to a highway bridge are presented in Table B-1 below. These include modeling parameters, scour parameters and corrosion parameters. These are compiled after a literature survey conducted by our research group. Assumptions are made for some of the parameters of which the pertaining distribution parameters are not found in the literature survey.

Table B-1: Uncertain parameters compiled from literature.

Uncertain parameters		Unit	Distrib ution	Distribution parameters		Reference
<i>Modeling parameters</i>						
Concrete unit weight	γ_{conc}	kg/m ³	normal	μ : 2400	δ : 0.05	Lee et al. [72]
Wearing surface unit weight	γ_{ws}	kg/m ³	normal	μ : 2200	δ : 0.25	Nowak et al. [73]
Elastic modulus of reinforcement	E _s	Gpa	normal	μ : 190	δ : 0.01	Lee et al. [72]
Expected compressive strength of concrete (substructure)	f _{ce, sub}	Mpa	normal	μ : 32.5	δ :0.1 25	Choi [74]
Expected compressive strength of concrete (I-girder)	f _{ce, I-girder}	Mpa	normal	μ : 52	δ : 0.125	Choi [74]
Expected compressive strength of concrete (box girder)	f _{ce, box-girder}	Mpa	normal	μ : 40.3	δ : 0.125	Choi [74]
Yield strength of reinforcement	f _{ye}	Mpa	lognorm al	λ : 6.16	ζ : 0.08	Elingwood and Hwang [75]

Table B-1: Uncertain parameters compiled from literature. (Cont'd)

Damping ratio	ζ	-	normal	μ : 0.045	δ : 0.278	Bavirisetty et al. [77]
Abutment stiffness	K_{abut}	kN/m m/m	uniform	lower : 14.35	upper : 28.7	Caltrans [91]
Longitudinal steel ratio	ρ_{long}	%	uniform	lower : 1.0	upper : 4.0	KGM [76]
Abutment - superstructure gap (I-girder)	$\delta_{g, I-girder}$	mm	uniform	lower : 25	upper : 50	Assumption
Abutment - superstructure gap (box girder)	$\delta_{g, box-girder}$	mm	uniform	lower : 50	upper : 75	Assumption
Shear modulus of elastomeric bearing	G_{el}	ksi	uniform	lower : 0.080	upper : 0.175	Caltrans [97]
Coefficient of friction between elastomeric bearing and concrete	μ_{el}	-	lognormal	μ : 0.4	ζ : 0.10	Mander et al. [55]
Soil parameters						
Friction angle	$\phi_{p, soil}$	degrees	normal	μ : 35	δ : 0.12	Zhang [98]
Unit weight of soil	γ_{soil}	kN/m ³	normal	μ : 18.64	δ : 0.1	Zhang [98]
Scour parameters						
K2 (correction factor for flood attack)	K_2	-	uniform	lower : 1.0	upper : 1.5	Arneson et al. [5]
K3 (correction factor for riverbed condition)	K_3	-	normal	μ : 1.1	δ : 0.05	Johnson [99]
Slope of the river	S	-	lognormal	μ : 0.002	δ : 0.25	Assumption
Manning's coefficient	n	-	lognormal	μ : 0.025	δ : 0.275	Hydraulic Engineering Center [100]
Local scour modeling factor	λ_{sc}	-	normal	μ : 0.55	δ : 0.52	Johnson [99]
Aging parameters						
Diffusion coefficient (28 day)	$D_{cl, 0}$	mm ² /yr	normal	μ : 473	σ : 43.2	Duracrete [88]
Environmental correction factor	k_e	-	Gamma	μ : 0.265	σ : 0.045	Duracrete [88]
Curing time correction factor	k_c	-	Beta	μ : 1.5	σ : 0.3	Duracrete [88]
Aging factor	n_{cl}		Beta	μ : 0.362	σ : 0.245	Duracrete [88]

Table B-1: Uncertain parameters compiled from literature. (Cont'd)

Critical chloride content (fully-saturated)	C_{cr}	% mass	normal	μ : 0.50	σ : 0.15	Duracrete [88]
C_s parameter	A_{cs}		normal	μ : 7.758	σ : 1.360	Duracrete [88]
	ε_{cs}		normal	μ : 0	σ : 1.105	
Test correction factor	k_t		normal	μ : 0.832	σ : 0.024	Duracrete [88]
Modelling uncertainty	x_j		lognormal	μ : 1	δ : 0.05	Duracrete [88]
Corrosion rate	r_{corr}	mm/yr	lognormal	lower : 0.218	upper : 0.328	Choe et al. [38]

These parameters are further utilized in the sensitivity study to observe the most influential parameters in seismic response of bridges. Four cases are investigated: 1) No Aging and No Scour, 2) Aging and No Scour, 3) No Aging and Scour, 4) Aging and Scour. For the cases with aging, corrosion initiation time and corrosion rate are depicted as the aging inputs, and for cases with scour, scour depth is chosen as the scour input for sensitivity analyses. The probability distributions for corrosion initiation time and scour depth are determined via Monte Carlo Simulation by combining the input parameter distributions listed in Table B-1 and their formulas. The process is as follows: 1,000,000 samples are randomly taken from the input parameters via Monte Carlo Simulation procedure. These random sample combinations are used in formulas to attain the histograms. Then, the distributions are fitted with MATLAB.

Corrosion initiation time is estimated from the following formula in Equation B-1 [88]:

$$T_i = \left(\frac{x^2}{4k_e k_c D_c (t_0)^{n_{cl}}} \left[\text{erf}^{-1} \left(\frac{C_0 - C_{cr}}{C_0} \right) \right]^{-2} \right)^{\frac{1}{(1-n_{cl})}} \quad (\text{B-1})$$

where x is the concrete cover depth (mm). For this study, x is taken as 50 mm. Other input parameters are explained in Table B-1.

Scour depth is determined from the following formula in Equation B-2 [5]

$$y_s = 2.0\lambda_{sc}K_1K_2K_3y_1^{0.35}a^{1.35}Fr_1^{0.43} \quad (B-2)$$

where y_1 is the flow depth directly upstream of the pier, a is the pier width (m) and Fr_1 is the Froude number directly upstream of the pier. Froude number is a function of the mean velocity and the flow depth. Other parameters are explained in Table B-1. Flow depth and the mean velocity of the flow are determined with Manning's open channel equation. Readers are referred to Arneson et al. [5] for further details. Please note that a discharge value of 3000 m³/s is assumed to be used in Manning's equation to obtain a case where the local scouring is rather large.

Resulting probability distributions are presented in Figure B-1. For corrosion initiation time (years), a lognormal distribution is attained with median $\lambda = 3.24$ and lognormal standard deviation $\zeta = 0.80$. For scour depth (m), a normal distribution is attained with mean $\mu = 3.15$ and standard deviation $\sigma = 1.55$.

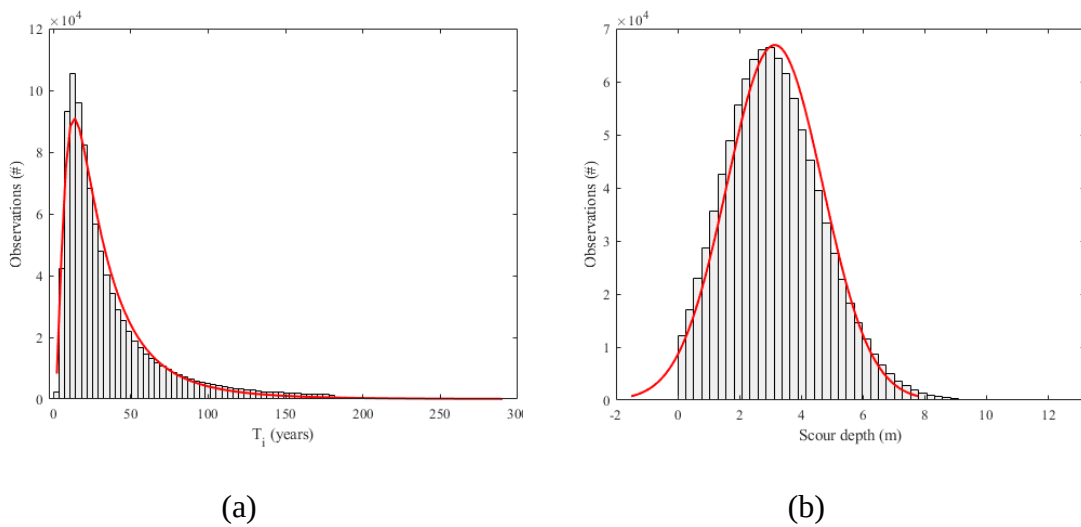


Figure B-1: Probability distributions for (a) corrosion initiation time (lognormal) and (b) scour depth (normal).

Curvature ductility demand of the piers are selected as the response criteria, and the “swings” are observed for all cases. Swings are obtained from observing the response difference from two instances of the parameter: One response is taken when the parameter is taken in their 2nd and the other in 98th percentile, and the swings are calculated accordingly to the “median” case, where the parameter is taken in the 50th percentile. Input values that are used in the sensitivity analysis are presented in Table B-2.

Table B-2: Input values for the sensitivity study.

Parameters	2 nd p.	98 th p.	Median
Unit mass of concrete (kg/m ³)	2160	2640	2400
Shear modulus of elastomer (Mpa)	0.55	1.2	0.875
Coefficient of friction of elastomer	0.2	0.6	0.4
Gap (mm)	50	75	62.5
Compressive strength of substructure (Mpa)	24.38	40.63	32.5
Compressive strength of superstructure (Mpa)	30.23	50.38	40.3
Yield strength of longitudinal reinforcement (Mpa)	405	545	475
Yield strength of transverse reinforcement (Mpa)	405	545	475
Damping ratio	0.02	0.07	0.045
Longitudinal reinforcement ratio	0.01	0.04	0.025
Steel elastic modulus (Gpa)	186.2	193.8	190
Friction angle (degrees)	33.6	36.4	35
Unit weight of soil (kN/m ³)	18.27	19.01	18.64
Unit mass of wearing surface (kg/m ³)	1100	3300	2200
Corrosion initiation time (years)	5.63	100	21.83
Corrosion rate (mm/year)	0.0218	0.0328	0.0273
Scour depth (m)	0	6.25	3.15

A full-fledged three-dimensional FE model of a generic bridge is developed in OpenSEES [68]. This bridge was initially developed as the study bridge of Yilmaz and Banerjee [67], which represents the characteristic bridges in California region, and readers are referred to that study for further details. This bridge is subjected to a strong

ground motion selected from NGA-West2 database [71]. The details of the ground motion are presented in Table B-3.

Table B-3: Details of the ground motion that is used for sensitivity analysis. Obtained from NGA-West2 database.

EQ name	Northridge-01 1994 (RSN963)
Station name	Castaic - Old Ridge Route
M_w	6.69
Scale	1
V_s (m/s)	450
PGA (g)	0.98

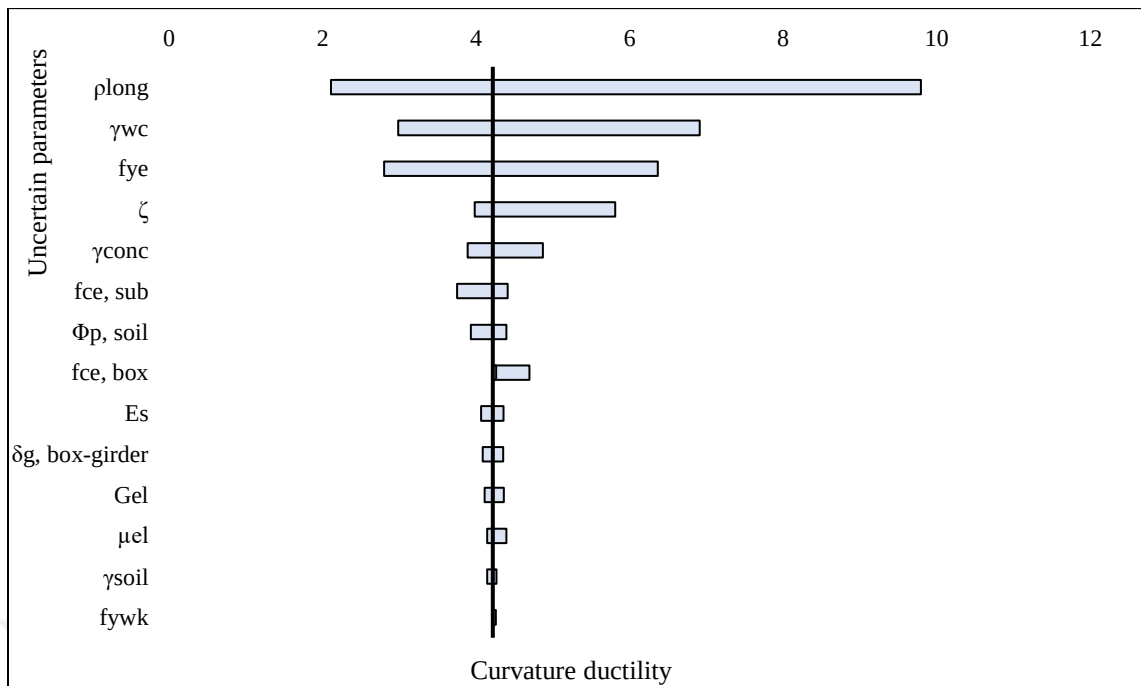
Much like the thesis methodology, ground motion is projected to the models in two horizontal directions, which is called “direction-1” in this document. Then, the ground motion components are swapped, in other words, the component in longitudinal direction is applied to transverse direction, and vice versa, which is called “direction-2”. Here, only the results for direction-1 is presented. Further presentation of the results for direction-2 is deemed as a waste of space due to the similarity of its results with direction-1. The individual significance of parameters is presented in a Tornado Diagram format, as shown in Figure B-2. Please note that only Figure B-2 (a) which refers to the “No Aging and No Scour” case is utilized for the study conducted in Chapter 3. Tornado Diagrams for other cases were constructed on the premise of using them in Chapter 4, however they didn’t find any use after the MML method was chosen instead of PSDM, which uses expected values as the inputs rather than taking samples from probability distributions. Nevertheless, results are presented with the aim of giving some idea about influential parameters on the seismic response of bridges that are located in the environments where the aging and scouring occurs.

Following conclusions might be drawn from Figure B-2:

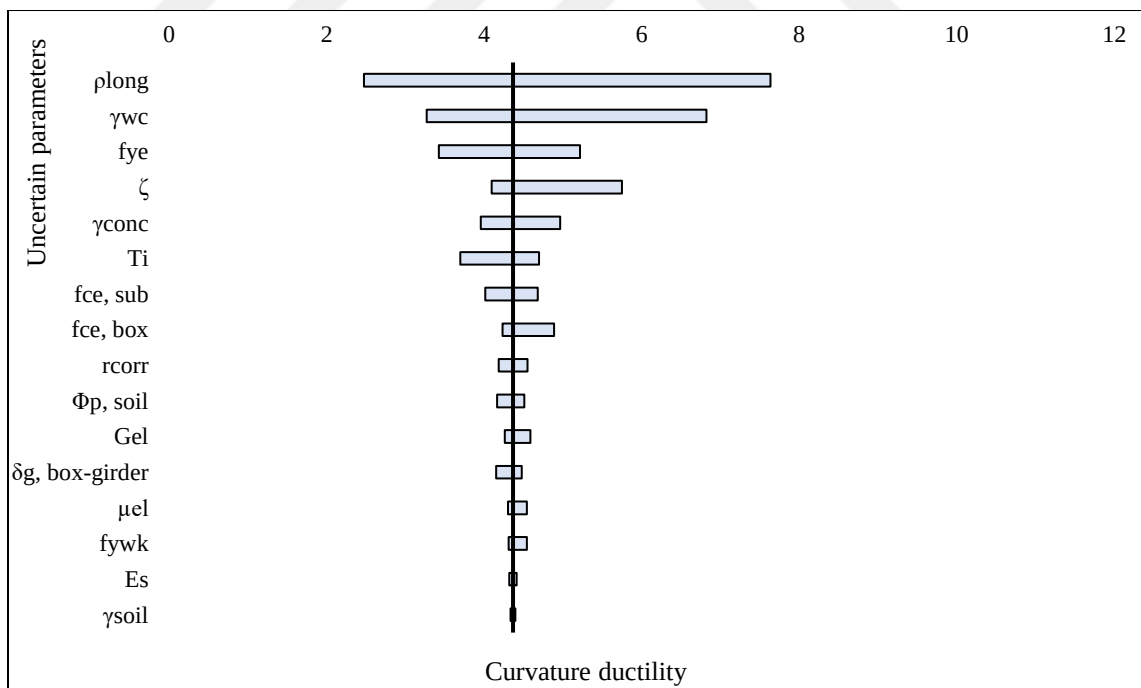
- Longitudinal reinforcement ratio (ρ_{long}) is observed to be the most influential and sensitive parameter for the seismic response of the highway bridges, for all four cases.
- Both the scour depth (y_s) and the corrosion initiation time (T_i) are observed to be sensitive, whereas corrosion rate (r_{corr}) is not. Scour depth is a more sensitive parameter than the corrosion initiation time, see Figure B-2 (d).

Top 6 most influential parameters of the Figure B-1 (a) are considered as uncertainty sources in the sampling process in Chapter 3, namely longitudinal reinforcement ratio (ρ_{long}), unit mass of wearing surface (γ_{wc}), yield strength of the longitudinal reinforcement (f_{ye}), damping ratio (ζ), unit mass of concrete (γ_{conc}) and the concrete compressive strength of the piers ($f_{\text{ce,sub}}$).

This study might be improved and its content can be expanded by being repeated for several strong ground motions and with more sophisticated sensitivity methods.

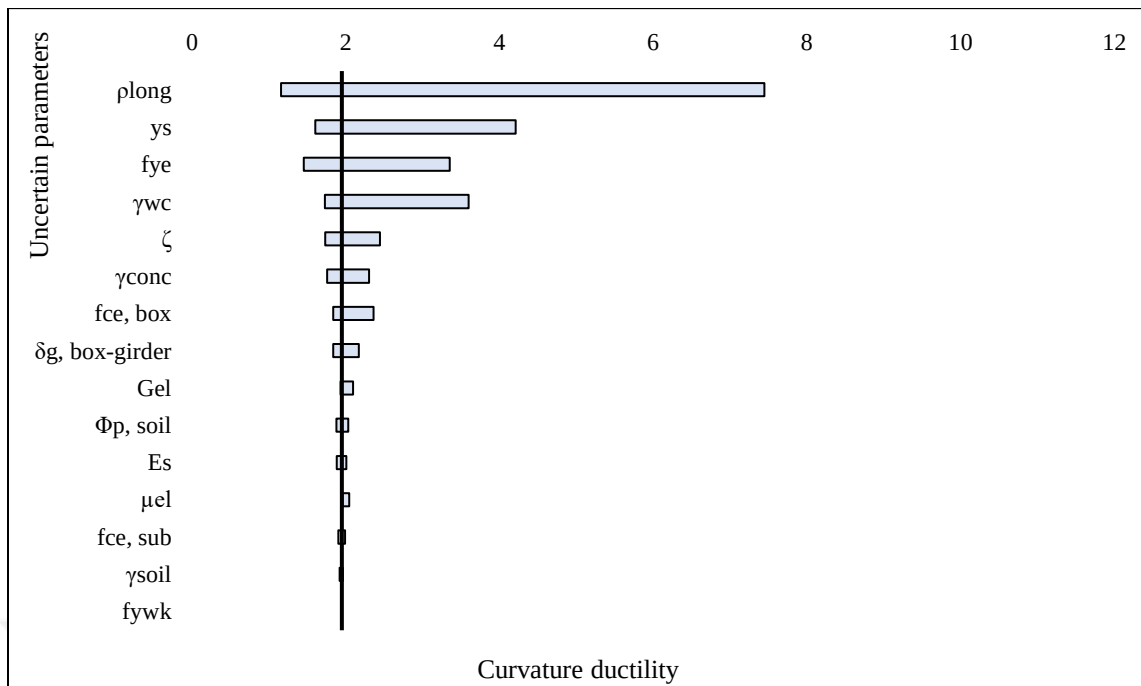


(a) No Aging / No Scour

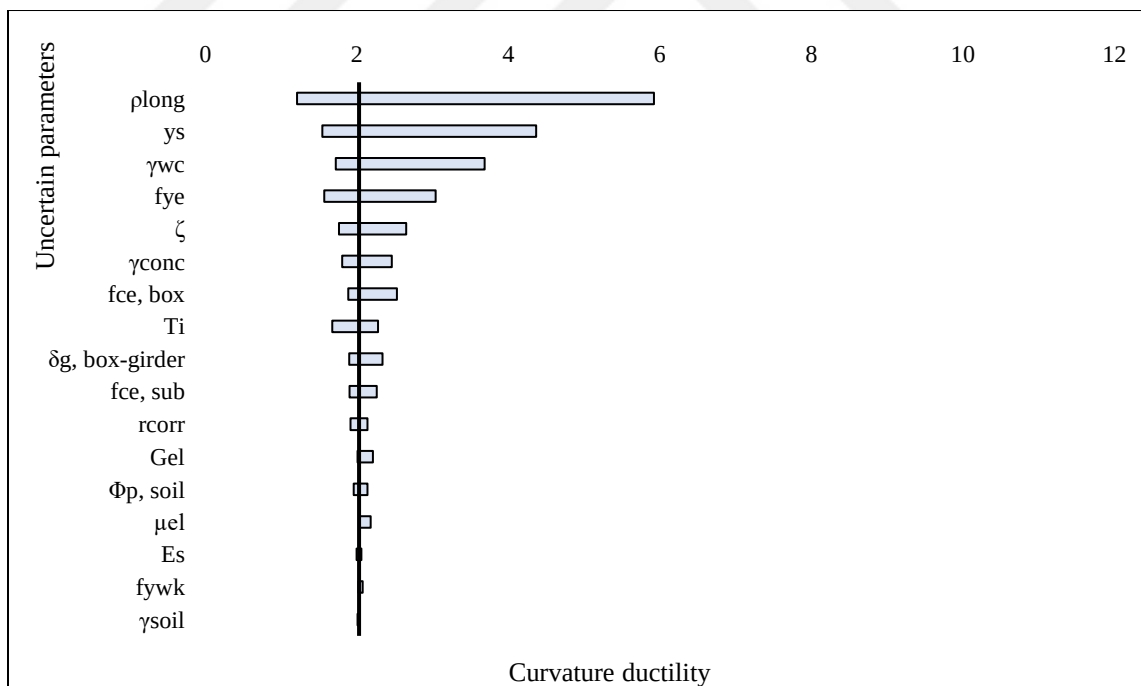


(b) Aging / No Scour

Figure B-2: Tornado diagrams for (a) No Aging / No Scour, (b) Aging / No Scour, (c) No Aging / Scour and (d) Aging / Scour cases.



(c) No Aging / Scour



(d) Aging / Scour

Figure B-2: Tornado diagrams for (a) No Aging / No Scour, (b) Aging / No Scour, (c) No Aging / Scour and (d) Aging / Scour cases. (Cont'd)

APPENDIX C

BRIDGE SAMPLES AND CONFIDENCE INTERVALS

The samples generated for the confidence interval study in Chapter 3 are shown in Table C-1. The sampled parameters are yield strength of the steel reinforcement (f_{ye}), concrete compressive strength of the piers ($f_{ce,sub}$), unit mass of concrete (γ_{conc}), unit mass of wearing surface (γ_{wc}), longitudinal reinforcement ratio (ρ_{long}) and damping ratio (ζ). Component fragility curve median values for the bridge piers are presented in Table C-2. These are obtained via MML (see 3.5.1). Lognormal standard deviation (i.e. dispersion) of the fragility is assumed as 0.6 per HAZUS [69].

Table C-1: Samples generated for the confidence interval study in Chapter 3.

#	f_{ye} (Mpa)	$f_{ce,sub}$ (Mpa)	γ_{conc} (t/m ³)	γ_{wc} (t/m ³)	ρ_{long}	ζ
1	474.58	34.72	2.07	2.97	0.03646	0.32113
2	482.81	30.24	3.04	0.75	0.01570	0.05279
3	462.76	32.24	2.76	2.40	0.03353	0.05056
4	478.15	33.23	2.97	1.46	0.02941	0.09970
5	473.33	34.21	2.17	1.56	0.02388	0.15028
6	465.96	35.46	2.32	2.65	0.02693	0.19994
7	469.22	32.40	2.48	3.27	0.01358	0.02857
8	473.74	28.26	2.84	1.00	0.02244	0.10417
9	486.40	31.24	2.44	3.47	0.03832	0.05510
10	484.15	30.46	2.72	2.75	0.02626	0.18723
11	464.92	37.64	1.72	1.98	0.02460	0.11767
12	485.34	31.29	2.36	1.87	0.01670	0.23789
13	476.63	33.69	2.28	1.67	0.02280	0.04105
14	471.83	31.90	2.01	1.58	0.02786	0.03049
15	475.88	34.16	2.65	2.05	0.03137	0.25307
16	470.79	30.90	2.58	2.13	0.01897	0.10771
17	471.44	29.81	2.56	2.89	0.02108	0.02448
18	480.40	29.08	2.06	3.84	0.04257	0.08460
19	478.97	33.06	2.90	2.48	0.00546	0.00013
20	479.60	32.03	2.64	1.79	0.04387	0.13054
21	475.32	31.73	2.38	2.69	0.02928	0.16030

Table C-2: Samples generated for the confidence interval study in Chapter 3. (Cont'd)

22	467.40	35.05	2.55	2.22	0.03538	0.07414
23	476.16	32.97	2.52	1.22	0.01186	0.01011
24	481.64	32.51	1.94	3.13	0.01774	0.07162
25	474.37	36.07	2.41	2.37	0.02003	0.01543
26	478.48	30.60	2.23	2.18	0.03073	0.07335
27	477.07	33.45	2.20	2.56	0.00864	0.09143
28	470.22	33.76	2.13	2.29	0.03335	0.03681
29	472.52	31.62	2.30	1.37	0.01437	0.02088
30	468.79	32.76	1.85	1.93	0.02580	0.15446

Table C-3: Fragility curve median values for the confidence interval study.

Analysis	Minor	Moderate	Major	Collapse
1	0.96	3.21	77.28	77.28
2	0.18	0.62	0.87	1.08
3	0.33	0.92	1.22	1.64
4	0.39	1.45	1.77	2.50
5	0.42	1.13	1.43	1.43
6	0.54	1.81	2.50	3.21
7	0.13	0.42	0.79	0.91
8	0.33	1.02	1.60	2.50
9	0.41	1.27	2.07	2.50
10	0.48	1.60	2.50	3.21
11	0.44	1.36	1.81	2.50
12	0.43	1.46	2.06	3.21
13	0.26	0.97	1.24	1.77
14	0.31	1.07	1.27	2.07
15	0.75	2.50	3.21	3.21
16	0.27	0.96	1.45	1.60
17	0.21	0.70	0.92	1.19
18	0.51	1.81	2.50	2.50
19	0.02	0.09	0.22	0.33
20	0.64	2.50	2.50	3.21
21	0.51	1.81	2.50	3.21
22	0.42	1.27	2.50	2.50
23	0.10	0.34	0.58	0.84
24	0.26	0.93	1.18	1.60
25	0.20	0.64	0.92	1.18
26	0.39	1.18	1.81	2.50
27	0.11	0.33	0.57	0.91
28	0.33	1.07	1.81	2.50
29	0.15	0.44	0.82	0.96
30	0.49	1.81	2.50	2.50

APPENDIX D

DETAILS OF THE GROUND MOTION DATA SET DEVELOPED TO OBTAIN THE TIME-DEPENDENT FRAGILITY CURVES FOR ORDINARY HIGHWAY BRIDGES

Details for the ground motion data set that is developed with CMS (Conditional Mean Spectrum) for Chapter 4 is presented in Table D-1. GM names are presented as it is referred in NGA-West2 database [71].

Table D-1: Ground motion properties developed for generating the time-dependent multi-hazard fragility curves.

GM No	GM Name	Scale	PGA (g)	Sa (T=1s) (g)
1	RSN0001	1	0.17	0.14
2	RSN0006	1	0.29	0.55
3	RSN0030	1	0.44	0.24
4	RSN0031	1	0.31	0.21
5	RSN0033	1	0.37	0.24
6	RSN0095	1	0.43	0.44
7	RSN0096	1	0.27	0.29
8	RSN0145	1	0.25	0.17
9	RSN0158	1	0.41	0.47
10	RSN0160	1	0.80	0.62
11	RSN0164	1	0.19	0.31
12	RSN0167	1	0.21	0.11

Table D-1: Ground motion properties developed for generating the time-dependent multi-hazard fragility curves. (Cont'd)

13	RSN0169	1	0.35	0.55
14	RSN0183	1	0.64	0.51
15	RSN0187	1	0.22	0.18
16	RSN0212	1	0.27	0.19
17	RSN0214	1	0.16	0.28
18	RSN0222	1	0.28	0.16
19	RSN0223	1	0.28	0.26
20	RSN0233	1	0.18	0.21
21	RSN0235	1	0.46	0.18
22	RSN0236	1	0.26	0.21
23	RSN0239	1	0.48	0.16
24	RSN0243	1	0.32	0.17
25	RSN0248	1	0.32	0.21
26	RSN0249	1	0.45	0.21
27	RSN0250	1	1.02	0.60
28	RSN0265	1	0.67	0.65
29	RSN0281	1	0.17	0.20
30	RSN0320	1	0.19	0.26
31	RSN0448	1	0.46	0.47
32	RSN0457	1	0.26	0.24
33	RSN0461	1	0.32	0.45
34	RSN0502	1	0.16	0.28
35	RSN0547	1	0.29	0.21

Table D-1: Ground motion properties developed for generating the time-dependent multi-hazard fragility curves. (Cont'd)

36	RSN0548	1	0.21	0.35
37	RSN0549	1	0.27	0.36
38	RSN0550	1	0.17	0.07
39	RSN0558	1	0.45	0.72
40	RSN0585	1	1.54	0.73
41	RSN0724	1	0.22	0.22
42	RSN0725	1	0.47	0.48
43	RSN0727	1	0.85	0.69
44	RSN0821	1	0.50	1.09
45	RSN0832	1	0.15	0.29
46	RSN0848	1	0.42	0.42
47	RSN0850	1	0.19	0.41
48	RSN0855	1	0.16	0.15
49	RSN0862	1	0.13	0.21
50	RSN0864	1	0.30	0.67
51	RSN0881	1	0.23	0.32
52	RSN0884	1	0.11	0.31
53	RSN0900	1	0.25	0.60
54	RSN0901	1	0.65	0.23
55	RSN0902	1	0.23	0.50
56	RSN0930	1	0.12	0.15
57	RSN1111	1	0.52	0.41
58	RSN1158	1	0.40	0.74

Table D-1: Ground motion properties developed for generating the time-dependent multi-hazard fragility curves. (Cont'd)

59	RSN1159	1	0.11	0.32
60	RSN1166	1	0.12	0.34
61	RSN1605	1	0.53	0.92
62	RSN1611	1	0.11	0.16
63	RSN1612	1	0.16	0.12
64	RSN1614	1	0.15	0.17
65	RSN1615	1	0.27	0.21
66	RSN1617	1	0.90	0.19
67	RSN1618	1	0.16	0.18
68	RSN1633	1	0.55	0.65
69	RSN1787	1	0.34	0.51
70	RSN8166	1	0.40	0.27
71	RSN0160	2	1.60	1.24
72	RSN0169	2	0.70	1.10
73	RSN0250	2	2.03	1.21
74	RSN0265	2	1.33	1.31
75	RSN0461	2	0.64	0.91
76	RSN0558	2	0.90	1.45
77	RSN0585	2	3.09	1.45
78	RSN0727	2	1.69	1.37
79	RSN0848	2	0.85	0.83
80	RSN1605	2	1.06	1.85

APPENDIX E

FRAGILITY ANALYSIS RESULTS FOR THE ORDINARY AGING HIGHWAY BRIDGES SUBJECT TO FLOOD- INDUCED SCOUR

Table E-1: Fragility median values for piers.

Analysis No	PIER			
	Minor	Moderate	Extensive	Collapse
1	0.4562	0.9586	1.5813	2.2445
2	0.4562	0.9586	1.5813	2.2445
3	0.4336	0.9336	1.5102	1.8258
4	0.4266	0.8633	1.3945	1.7359
5	0.4055	0.8023	1.3945	1.7359
6	0.5219	1.2125	1.7359	2.4422
7	0.5219	1.2125	1.7359	2.4422
8	0.4953	1.0438	1.7359	2.2445
9	0.4555	0.9797	1.6461	2.2445
10	0.4336	0.9797	1.5102	1.9547
11	0.6	1.3945	1.9547	3.0992
12	0.5789	1.3945	1.9547	3.0992
13	0.5688	1.3945	1.8461	3.0992
14	0.5594	1.2547	1.8461	2.6828
15	0.5133	1.1281	1.8461	2.4273
16	0.7047	1.5102	2.2445	3.8594
17	0.6898	1.5102	2.2445	3.8594
18	0.6497	1.4516	2.0883	3.8594
19	0.6008	1.4516	1.9547	3.8594
20	0.5898	1.3451	1.8461	2.6828
21	0.7359	1.8461	2.6828	-
22	0.7359	1.8461	2.6828	-
23	0.7209	1.7484	2.4273	-
24	0.7209	1.5162	2.4273	3.8594
25	0.6766	1.4563	2.0883	3.8594

Table E-2: Fragility median values for bearings (in longitudinal dir.).

Analysis No	BEARING (LONG.)			
	Minor	Moderate	Extensive	Collapse
1	0.2414	0.5516	3.8594	-
2	0.2414	0.5516	3.8594	-
3	0.2414	0.5516	3.8594	-
4	0.2414	0.5516	3.0992	-
5	0.2414	0.5414	3.0992	-
6	0.2195	0.5508	3.0992	-
7	0.2195	0.5508	3.0992	-
8	0.2195	0.5313	3.0992	-
9	0.2195	0.5219	3.0992	-
10	0.2195	0.5039	3.0992	-
11	0.2094	0.4875	3.0992	-
12	0.2094	0.4875	3.0992	-
13	0.2094	0.4875	3.0992	-
14	0.2094	0.4875	3.0992	-
15	0.2094	0.4875	3.0992	-
16	0.2031	0.4961	3.0992	-
17	0.2031	0.4961	3.0992	-
18	0.2031	0.4875	3.0992	-
19	0.2031	0.4875	3.0992	-
20	0.2031	0.4875	3.0992	-
21	0.1891	0.4859	3.0992	-
22	0.1891	0.4859	3.0992	-
23	0.1859	0.4859	3.0992	-
24	0.1859	0.4859	3.0992	-
25	0.1867	0.4859	3.0992	-

Table E-3: Fragility median values for abutments, for both longitudinal and transverse directions.

Analysis No	ABUTMENT (LONG.)				ABUTMENT (TRANSVERSE)			
	Minor	Moderate	Extensive	Coll apse	Minor	Moderate	Extensive	Coll apse
1	0.708	1.4334	-	-	0.7953	1.7352	-	-
2	0.708	1.4334	-	-	0.7953	1.7352	-	-
3	0.708	1.4391	-	-	0.7773	1.7352	-	-
4	0.708	1.5125	-	-	0.7773	1.7352	-	-
5	0.708	1.4539	-	-	0.7453	1.7352	-	-
6	0.6672	1.3477	-	-	0.7453	1.8258	-	-
7	0.6539	1.3477	-	-	0.7453	1.8258	-	-
8	0.6539	1.3477	-	-	0.7453	1.8258	-	-
9	0.6669	1.3477	-	-	0.7453	1.8258	-	-
10	0.6669	1.3477	-	-	0.7453	1.8258	-	-
11	0.593	1.2977	-	-	0.6995	1.8258	-	-
12	0.593	1.2977	-	-	0.6995	1.8258	-	-
13	0.5922	1.2977	-	-	0.6995	1.8258	-	-
14	0.5953	1.2977	-	-	0.6995	1.8258	-	-
15	0.5633	1.2977	-	-	0.6995	1.8258	-	-
16	0.5406	1.2977	-	-	0.6859	1.8258	-	-
17	0.5406	1.2977	-	-	0.6859	1.8258	-	-
18	0.5406	1.2977	-	-	0.6859	1.8258	-	-
19	0.5406	1.2977	-	-	0.6859	1.8258	-	-
20	0.5406	1.2977	-	-	0.6859	1.8258	-	-
21	0.5406	1.2977	-	-	0.6859	1.8258	-	-
22	0.5406	1.2547	-	-	0.6859	1.8258	-	-
23	0.5313	1.2547	-	-	0.6859	1.8258	-	-
24	0.5313	1.2547	-	-	0.6859	1.8258	-	-
25	0.5414	1.2547	-	-	0.6859	1.8258	-	-

Table E-4: Fragility median values for bridge system level.

Analysis No	SYSTEM			
	Minor	Moderate	Extensive	Collapse
1	0.2375	0.5516	1.5813	2.2445
2	0.2375	0.5516	1.5813	2.2445
3	0.2375	0.5414	1.5102	1.8258
4	0.2375	0.532	1.3945	1.7359
5	0.2375	0.5227	1.3945	1.7359
6	0.2156	0.5508	1.7359	2.4422
7	0.2156	0.5508	1.7359	2.4422
8	0.2156	0.5313	1.7359	2.2445
9	0.2156	0.5125	1.6461	2.2445
10	0.2156	0.4953	1.5102	1.9547
11	0.2053	0.4875	1.9547	3.0992
12	0.2053	0.4875	1.9547	3.0992
13	0.2053	0.4875	1.8461	3.0992
14	0.2053	0.4875	1.8461	2.6828
15	0.2053	0.4797	1.8461	2.4273
16	0.2031	0.4961	2.2445	3.8594
17	0.2031	0.4961	2.2445	3.8594
18	0.2031	0.4875	2.0883	3.8594
19	0.2031	0.4875	1.9547	3.8594
20	0.2031	0.4875	1.8461	2.6828
21	0.1891	0.4859	2.6828	-
22	0.1891	0.4859	2.6828	-
23	0.1859	0.4859	2.4273	-
24	0.1859	0.4859	2.4273	3.8594
25	0.1828	0.4859	2.0883	3.8594

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