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SEISMIC RELIABILITY ASSESSMENT OF AGEING INTEGRAL BRIDGES

By

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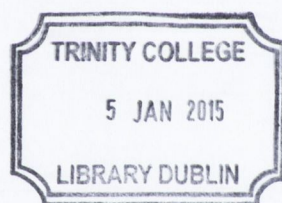
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May 2014

Department of Civil, Structural and Environmental Engineering

University of Dublin

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Thesis 10791

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SUMMARY

This thesis carries out a time dependent seismic risk assessment of ageing integral bridges as well as providing a comparison between the seismic performance of integral and jointed bridges.

Over the past two decades, the probabilistic risk assessment of highway bridges has developed rapidly in order to prioritise bridges for retrofit and rehabilitation based on their seismic risk. However, most of the current seismic risk assessment packages assume bridges remain in their as-built condition. For the risk assessment of RC bridges that lie in corrosive regions, deterioration may need to be considered.

The first part of this thesis proposes an approach for time-dependent bridge seismic fragility analysis that incorporates the influence of ageing on both the bridge capacity and the seismic demands, which are traditionally either neglected or emphasised singularly in seismic vulnerability modelling. This joint consideration is important since deterioration mechanisms such as corrosion of reinforcing steel can not only affect the resistance of key bridge components, but also the overall dynamic behaviour and demands on the structure. The fragility method is applied for the study of a group of multi-span integral concrete bridges, given the integral form's growing popularity, which can be attributed to the fact that they eliminate the need for maintenance associated with joints and bearings. This thesis firstly evaluates the impact of chloride induced corrosion of the columns and the deck on the bridge fragility, followed by foundation settlement of an intermediate pier. A probabilistic fragility analysis is carried out for the bridge at different points in time along the service life accounting for variation in bridge modeling parameters, corrosion parameters and ground motion. In addition to the columns and the abutments, the vulnerability of the deck is considered since its seismic performance when corroded or subjected to additional stresses from settlement has yet to be explored.

The results show that both the columns and the deck experience an increase in fragility with age, with the abutments experiencing a slight decrease in fragility with age. The columns are found to dominate the system fragility for this bridge type. Additionally,

settlements on the order of the discrete levels adopted for this thesis increased the system fragility at the slight, moderate and extensive damage states but their impact at the complete damage state is negligible.

The second part of this thesis compares the seismic performance of integral and jointed bridges, since the integral form is prevalent in bridge inventories worldwide, yet only a limited amount of studies have been carried out on its seismic performance. The results of this comparison show that although the integral bridge columns are more vulnerable to damage from seismic loads than the jointed bridge columns, the overall jointed bridge system is more vulnerable than the integral bridge system. The columns of the integral bridge are more vulnerable due to the fact that the superstructure is monolithic with the substructure and therefore more demand is transferred to the columns from the deck when the bridge is seismically loaded. Despite this, the presence of vulnerable components such as elastomeric bearings and shear keys, as well as the possibility of deck unseating, results in the integral bridge system being more resistant to seismic damage than the jointed bridge system.

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CHAPTER 1 - INTRODUCTION

1.1 Problem Description and Motivation

Throughout the world, many densely populated urban areas are located in earthquake prone regions and therefore are vulnerable to high consequence seismic events. In many cases, it takes years for communities to recover from the economic and social consequences of an earthquake. In recent years, numerous high magnitude earthquakes have occurred worldwide causing devastation in their wake in terms of human casualties and physical damage to buildings and infrastructure. For this reason, earthquake mitigation in the form of retrofit programmes and improved building codes is imperative to limit the loss of life and structural damage of an earthquake.

In order to aid the recovery period of an earthquake, it is of vital importance to attempt to predict losses and damage caused by future earthquakes and their probabilities of occurrence. This helps to direct resources where they are needed most, in order to mitigate future earthquake damage. During the recovery period of an earthquake, the uninterrupted operation of local infrastructure and more specifically, bridges is vital for rescue operations as well as the rehabilitation of an area. For this reason, over the past two decades, the probabilistic risk assessment of highway bridges has developed rapidly in order to prioritise bridges for retrofit and rehabilitation based on their seismic risk.

One approach to facilitate decision making for seismic bridge retrofit and rehabilitation is the use of seismic fragility curves. Seismic fragility curves are conditional probability statements that give the probability of a bridge exceeding defined damage states for a given ground motion intensity level. In recent years, many studies have developed fragility curves for highway bridges (Yu et al., 1991, Hwang et al., 2000, Mackie and Stojadinovic, 2001, 2005a, Kim and Shinozuka, 2004, Nielson, 2005, Padgett, 2007,

Ramanathan et al., 2010, Ramanathan, 2012) and some of these have been adopted in seismic risk assessment packages such as HAZUS (FEMA, 2005), REDARS (Werner et al., 2006) and MAEVIZ (MAE and NCSA, 2006). However, the current seismic risk assessment packages assume bridges remain in their as built or pristine condition. This approach may be acceptable for relatively new bridges; however, it is acknowledged that many bridges that lie in earthquake prone regions have reached or passed the end of their design service life. Statistics in the United States (US) show that over half of the country's bridges have reached the end of their design service life (ASCE, 2013). These bridges have therefore been subjected to various forms of deterioration for many years, depending on their environment. Since deterioration affects the main force resisting components of a bridge, it is reasonable to assume it increases the vulnerability of bridges subject to seismic hazard. Therefore, the use of fragility functions based on bridges in their pristine condition may lead to unreliable loss estimates following the current risk assessment methods. This provides the main aim and motivation of the thesis.

Although very recently a limited amount of work has been carried out attempting to quantify the effect of deterioration on the seismic performance of bridges, a number of gaps still remain in the literature. The first gap addressed in this thesis is to provide new insight into the ageing fragility of the integral bridge type, which has not been studied in depth in the literature but is prevalent in bridge inventories worldwide. Integral bridges have become popular in recent years because they eliminate the need for maintenance associated with joints and bearings. Despite the growing popularity of integral bridges and the fact that they have been constructed for many decades, the design and analysis of these structures relies on a limited amount of technical references and design guidelines exist in few countries (Denton and Tsionis, 2012). Furthermore, there has been very little research on the seismic performance of integral bridges. To the author's knowledge, there have been a few studies carried out comparing the seismic fragility of bridges with integral and seat type abutments (Ramanathan, 2012, Zakeri et al.), however, these studies only examined bridges that were integral over the column bents. This provides the motivation for the second aim of this thesis, which is to directly compare the seismic fragility of integral and jointed

bridges, in order to draw conclusions on the change in seismic performance when integral construction is used.

Most of the work carried out to date on the ageing fragility of bridges focuses only on corrosion of the reinforced concrete (RC) columns. This thesis examines the effect of corrosion of the concrete columns and deck slab on the bridge fragility. Until now, most seismic fragility analyses have assumed the deck remains elastic under seismic loading following recommendations in the literature (Aviram et al., 2008). However, these assumptions were made based on bridges in their as-built condition and therefore, it needs to be investigated whether the inclusion of deck corrosion results in demands exceeding reduced limit state capacities. Furthermore, most of the work to date has focussed on the effect of deterioration on either the seismic demand or the seismic capacity, whereas the fragility formulation herein is posed to consider the influence of ageing on both the component capacities and demands. This is important since conditions such as corrosion of reinforcing steel have been shown to not only affect the resistance of key components, but also the overall dynamic behaviour and demands on the structure.

Finally, until now most of the research that attempts to quantify the effect of ageing on the seismic fragility of bridges has focused on corrosion of the reinforcing steel. However, it should be acknowledged that other components of a bridge at the end of its design life may be altered from their as-built condition. One such example is settlement of the pier foundations. Depending on local soil and environmental conditions, differential settlement may occur between piers inducing forces in the superstructure (INDOT, 2010). This thesis investigates the effect of settlement on the time dependent seismic fragility of integral bridges, and thus explores the relative need to account for such additional forms of ageing when evaluating bridge vulnerability to seismic loading.

1.2 Research Objectives

The previous section discussed the gaps in the literature addressed in this thesis. These include the historic lack of consideration of the joint effect of seismic and ageing

threats as well as the lack of research on the seismic performance of integral bridges. The main objective of this thesis is to examine the effect of ageing on the seismic performance of integral bridges. To this end, a group of three span integral bridges were chosen due to the popularity of this arrangement for highway overpasses. A second aim of this thesis is to directly compare the seismic performance of integral and jointed bridges. More specifically, these aims are achieved through the following steps:

- Generation of a group of stochastic three dimensional non-linear finite element models for a group of integral bridges with varying bridge length and column height. This involves the identification and probabilistic modelling of potentially uncertain modelling parameters.
- Development of a corrosion model for the RC columns and deck. This corrosion model provides the loss of steel cross section and strength as a function of time. Realistic levels of foundation settlement must also be chosen. The group of stochastic finite element models are then modified to account for the effects of ageing at different points in time along the service life of the structure.
- Development of time dependent seismic demand models by carrying out a Probabilistic Seismic Demand Analysis (PSDA) using the stochastic finite element models at various points in time along the service life of the bridge.
- Development of time dependent seismic capacity models for the critical bridge components by incorporating the corrosion model with models adopted from the literature and codes for estimating component capacities.
- Generation of time dependent seismic fragility curves for the individual bridge components as well as the overall bridge system. This enables conclusions to be drawn on the effect of ageing on the seismic performance of integral bridges.
- The final objective is to carry out a seismic fragility analysis for a group of three span jointed bridges with similar geometric properties to the integral bridges

under consideration. This will facilitate a direct comparison between the seismic performance of integral and jointed bridges.

1.3 Organisation of Thesis

This thesis is made up of eight chapters, outlined as follows:

Chapter 1 provides an outline of the research problem and the motivation for carrying out a time dependent seismic fragility analysis of the integral bridge type, accounting for the effects of ageing. The objectives of the work and an outline of the thesis are also presented.

Chapter 2 presents a review of the relevant literature. This includes a review of the use and performance of integral bridges; a description of the main deterioration mechanisms affecting concrete highway bridges; a state of the art summary of seismic risk assessment methods; a description of the evolution of fragility curve development and a review of the research carried out to date on the ageing fragility of bridges.

Chapter 3 provides extensive details about the modelling strategies for the bridge components including the superstructure; columns; foundations and abutments. Deterministic responses are presented to provide insight into the seismic response of the bridge components.

Chapter 4 provides details of the corrosion models for the RC columns and deck slab, where corrosion is modelled as a function of time. Details are also presented on the discrete levels of foundation settlement considered for this thesis. Modified deterministic responses are presented to provide insight into the effect of ageing on the deterministic response of the bridge components.

Chapter 5 outlines the framework that will be adopted in the development of time dependent analytical fragility curves for the integral bridges considered. Details are provided regarding the different aspects of the multi-phase framework including the selection of a suitable ground motion suite; development of a group of stochastic finite element models at various points along the service life of the bridges and formulation

of the time dependent seismic demand models and the time dependent capacity models.

Chapter 6 presents time dependent fragility curves considering the effects of ageing including corrosion and foundation settlement. Fragility curves are also presented considering the effect of column height and bridge length variation.

Chapter 7 develops seismic fragility curves for a group of jointed bridges with similar geometric properties to the integral bridges considered. A direct comparison is then presented between the seismic performance of the integral and jointed bridges.

Chapter 8 presents the conclusions from the present research as well as providing recommendations for future work.

CHAPTER 2 - LITERATURE REVIEW

In recent years, numerous high magnitude earthquakes have occurred worldwide causing devastation in their wake. In many cases, it takes years for communities to recover from the economic and social consequences of an earthquake. In the past decade, some of the most notable earthquakes in terms of damage caused have occurred in Japan, Haiti, and China. The 2011 Tōhoku earthquake, with a moment magnitude of 9.0, occurred 70km off the coast of Japan and triggered a subsequent tsunami that caused severe structural damage including several nuclear accidents and left 20,352 people dead. The 2010 earthquake that occurred in Haiti had a magnitude of 7.0 and caused 316,000 deaths. The 2008 Sichuan earthquake, with a magnitude of 7.9 resulted in 87,587 fatalities in China. From these few examples it can be seen that earthquake mitigation in the form of retrofit programmes and improved building codes is imperative to limit the loss of life and structural damage of an earthquake.

During the recovery period of an earthquake it is of vital importance that local infrastructure remains functional for emergency services to aid the recovery and rehabilitation of an area. For this reason, a large amount of research has been carried out on the seismic vulnerability of bridges. The integral bridge form has attracted widespread attention in recent years due to its improved durability and is the main bridge type under consideration in this thesis. The first section of this chapter will provide information on integral bridge use and performance based on findings in the literature.

In earthquake zones, it is also of vital importance to attempt to predict losses and damage caused by future earthquakes and their probabilities of occurrence. This helps to direct resources where they are most needed in order to mitigate earthquake damage. Over the past two decades the probabilistic risk assessment of highway bridges has developed rapidly. However, little attention has been devoted to the

seismic reliability of deteriorated structures. For the risk assessment of RC bridges in corrosive environments and earthquake prone regions, the effect of corrosion on the seismic performance needs to be taken into consideration. This provides the main aim and motivation for the thesis.

This thesis focuses on the ageing fragility of the integral bridge form and therefore, the first section of this chapter provides a review of the use and performance of integral bridges. The second section of this literature review includes a discussion on the deterioration of vulnerable bridge components, as well as a description of the corrosion process. A state of the art summary of existing seismic risk assessment methods is then provided, which includes a description of the main risk assessment software packages currently in use. A discussion on the evolution of fragility curve development is then presented, including the main fragility methods that are used for seismic risk assessment. The chapter is concluded with a review of recent work carried out on the ageing fragility of highway bridges.

2.1 Review of the Use and Performance of Integral Bridges

The traditional method of bridge construction involves the separation of superstructure and substructure through a system of joints and bearings. This type of construction dominated in the 1960's and 1970's and as these bridges aged over time, they required extensive maintenance due to the deterioration of joints. The major costs involved with servicing and rehabilitating these bridge types created a need for an alternative design with improved durability. As a result, integral construction has received a lot of attention over the past few decades. Integral or jointless bridges are those where the superstructure and substructure act as a monolithic unit with no joints or bearings connecting the two. The use of integral construction has increased significantly over the past few decades. In the United Kingdom (UK) for example, designers are now required to consider the integral form for bridges up to 60 m (The Highways Agency, 2001). This trend is also reflected in the United States (US) where at least 40 states are now building some form of jointless bridges and although superstructures with deck joints still predominate, the trend appears to be moving towards integral construction (Mistry, 2005). Despite the growing popularity of

integral bridges and the fact that they have been constructed for many decades, the design and analysis of these structures relies on a limited amount of technical references and design guidelines exist in few countries (Denton and Tsionis, 2012). This situation highlights the need for more research in the area to streamline the design process for better performance.

2.1.1 Integral Construction

As mentioned previously, integral bridges are designed so that the superstructure and substructure are continuous with each other. There are many different types of integral bridges, with different levels of continuity between the superstructure and substructure. Figure 2.1 (a) shows a bridge whose deck is made up of separate precast beams in each span. This design, which is continuous over the central support, provides a more durable option to the traditional approach which may have had a joint over the central support. The bridge in Figure 2.1 (b) takes the integral design a step further and is continuous over the internal support and integral with the abutments at both ends. Finally, Figure 2.1 (c) shows a fully integral bridge where the deck is integral with both abutments as well as the intermediate pier. The integral bridge type considered in this thesis is designed following the form of bridge (c), according to design drawings provided by Roughan & O'Donovan Consulting Engineers.

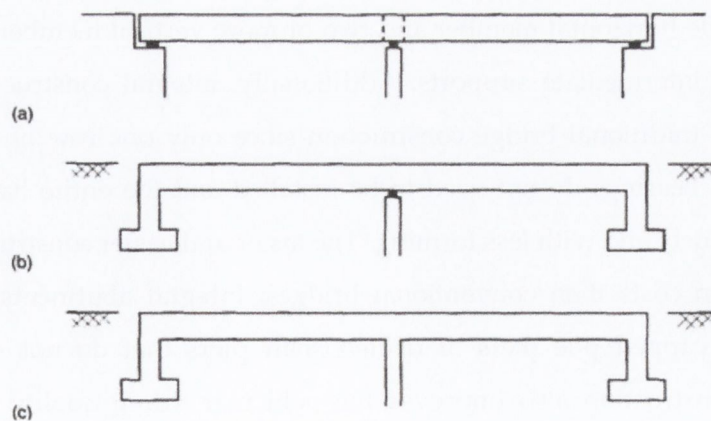


Figure 2.1 Integral bridges: (a) precast beams made integral over the interior support; (b) deck continuous over interior support and integral with abutments; (c) deck integral with abutments and pier (O'Brien and Keogh, 1999)

2.1.2 Advantages and Limitations of Integral Bridge Design

The main advantage associated with the integral form is the elimination of expansion joints and bearings, which results in major cost savings and an increase in service life (Mistry, 2005). The most frequently encountered corrosion problem for jointed bridges involves leaking expansion joints and seals which allow chloride-laden water from the application of de-icing salts, seep from the deck and attack girder ends, bearings and the reinforced concrete substructure. This situation means that bridges designed with such joints need to be monitored and repaired more regularly than integral bridges. Furthermore, the bearings themselves are often problematic with steel bearings seizing up over time due to corrosion and elastomeric bearings splitting or rupturing due to unanticipated movements (Paraschos and Amde, 2011). Integral construction therefore provides a more durable and cost effective solution to bridge design for specific span ranges. It must be acknowledged that although it is recognised that integral bridges are more durable than jointed bridges, they are also subject to chloride induced corrosion from airborne chlorides and de-icing salts. This thesis explores the effect of this type of corrosion on the seismic performance of the integral bridge type.

In addition to improved durability, Mistry (2005) outlines further advantages for the use of integral bridges. One such advantage is that the simple design of a fully integral bridge such as the one shown in Figure 2.1 (c), can be considered as a continuous frame with a single horizontal member and two or more vertical members depending on the number of intermediate supports. Additionally, integral construction is simpler and faster than traditional bridge construction since only one row of piles is used at the abutments, bearings do not need to be installed and the entire back wall can be cast simultaneously and with less forming. The faster and easier construction leads to lower construction costs than conventional bridges. Integral abutments are also generally built with capped pile piers or drilled shaft piers that do not require cofferdams. Integral construction also improves the vehicular riding quality due to the lack of expansion joints in the deck. It is also noted that integral abutments are more resistant to uplift, thus smaller end span to interior span ratios can be used without providing expensive hold-downs to expansion bearings. Finally, integral design provides a

greater live load capacity by distributing loads along the continuous and full depth diaphragm at bridge ends.

Paraschos and Amde (2011) carried out a survey of US Departments of Transportation (DOT) on the problems associated with integral abutment bridges. One of the problems noted by DOTs is the need for an approach slab and a joint at the end of the approach slab. Many states found problems with settlement of the approach slab due to longitudinal movements. This requires continuous maintenance and costly repairs. Another state mentioned that the integral form may not be suitable when temperature ranges are extreme. Integral abutment bridges accommodate thermal expansion and contraction through the rotation of foundation piles which is prevented when the ground is frozen during winter months. Finally, the imposed limits on the length and skew of integral bridges is seen as a limitation of the design and therefore, the majority of bridges greater than 60 m in length are not integral.

2.1.3 Seismic Performance of Integral Bridges

There is general agreement throughout the literature that integral bridges perform better than conventional bridges during seismic events (Mistry, 2005, Hassiotis et al., 2006, Paraschos and Amde, 2011). A primary benefit of integral bridges is the elimination of the unseating potential which is a common problem for traditional bridge design. Deck unseating of the simply supported spans of the Showa bridge during the 1964 Niigata earthquake in Japan can be seen in Figure 2.2. Another advantage is that the moment resisting connection between the column and pier cap provides increased redundancy in the structure and enhances the energy dissipation potential by increasing the number of plastic hinges that are needed to form a collapse mechanism. Mistry (2005) claims that integral abutments have consistently performed well in actual seismic events and significantly reduced or avoided problems such as back wall and bearing damage, associated with seat type abutments.

There have been a limited number of studies on the seismic performance of integral bridges which conclude that the integral bridges under consideration perform satisfactorily under seismic loads (Sritharan et al., 2005, Frosch et al., 2009). Additionally, Mitoulis and Tegos (2011), studied the response of two new integral

abutment configurations which use state of the art techniques to decouple the inservice response of the bridge from the backfill soil and utilise the backfill's resistance during an earthquake, aiming at reducing the seismic demand on bridges. However, to the author's knowledge, there have only been a few studies carried out comparing the seismic fragility of bridges with integral and seat type abutments (Ramanathan, 2012, Zakeri et al.). As mentioned previously, these studies only examined bridges that were integral over the column bents. This thesis will directly compare the seismic fragility of integral and jointed bridges, which is important in order to attempt to quantify the improvement in seismic performance when integral construction is used. It has also been suggested that without the system of joints and bearings which allows for superstructure movement in jointed bridges, greater demands may be placed on the abutments and substructure (Frosch et al., 2009). This claim will also be investigated in a later section of this thesis.



Figure 2.2 Deck unseating of the simply supported spans of the Showa bridge during the 1964 Niigata earthquake (Steinbrugge, 2011)

2.2 Deterioration of Reinforced Concrete Bridges

Many of the bridges across Europe and the United States are approaching the end of their design service life. According to a survey carried out by Transportation for America (2013), the average bridge age in the US is 43 years. Since the average design lifespan in the US is 50 years, it is noted that in less than 10 years the average bridge

age may exceed the average design service life (Transportation For America, 2013). In many cases, seismic zones throughout the US and Europe are also highly corrosive environments for structures. This is mainly due to the application of de-icing salts or the presence of airborne chlorides in coastal regions. As a result, many bridges located in seismic zones have suffered extensive levels of deterioration. In 2013 for example, one in nine, or just under 11% of bridges in the US were classified as being structurally deficient (ASCE, 2013). A bridge is deemed to be structurally deficient if it requires significant maintenance, rehabilitation or replacement. Structurally deficient bridges must be inspected every year since critical load-carrying elements were found to be in poor condition due to deterioration or damage (ASCE, 2013). A further 13.9% were classified as being functionally obsolete resulting in a situation where just less than one in four bridges in the US were labelled as deficient or obsolete. Functionally obsolete bridges no longer meet the current standards that are used today, for example, narrow lanes or low load-carrying capacity (ASCE, 2013). The Federal Highway Administration (FHWA) has estimated that the cost of repairing the deficient bridges in the US would be \$76 billion (Transportation For America, 2013). This figure encompasses the cost of repairing bridges that are deficient due to deterioration damage as well as bridges that were not designed to meet today's codes of practice. It has also been suggested that this figure is going to increase over the next few years as the average bridge age increases even further (Transportation For America, 2013).

The main bridge components that suffer the effects of ageing are the deck slab, deck girders, RC columns and bearings. Figure 2.3 (a) & (b) show two extreme cases of RC column corrosion. From the figures it can be seen that the concrete has spalled extensively and the reinforcement is severely corroded. Figure 2.4 (a) shows corrosion damage to the underside of a deck girder due to concentrated de-icing solution at a drain site. Figure 2.4 (b) shows the underside of a deck slab which has been seriously corroded displaying concrete spalling and steel corrosion. Figure 2.5 (a) & (b) show severely corroded steel bearings which have been mentioned previously as being particularly susceptible to deterioration and damage. Integral construction avoids these vulnerable bearings and therefore offers a major advantage over traditional jointed construction. Joints have also been mentioned as being problematic due to the fact that de-icing salts are allowed to seep through and attack girder ends, bearings and

the RC substructure. Figure 2.6 (a) shows extensive corrosion of a RC bent cap. This may be due to the presence of a joint allowing de-icing solution to seep down and cause corrosion. Figure 2.6 (b) shows corrosion induced girder damage caused by a failed joint and poor concrete cover over the girder ends. It must be noted that integral construction eliminates problematic joints and therefore prevents such damage to the concrete beams and substructure. From the figures it can be seen that the predominant deterioration mechanisms which affect highway bridges are degradation of concrete and corrosion of steel reinforcement. Another form of deterioration which is not shown in the figures but is investigated in this thesis is foundation settlement. The following sections discuss these principal forms of deterioration.



Figure 2.3 Corroding RC columns of highway bridges (Vector Corrosion Technologies, 2013, Aboutaha, 2012)



Figure 2.4 Corrosion of the underside of RC deck (Concrete Construction, 2011)



Figure 2.5 Corrosion of steel bearings (Freyssinet, 2013, Howard Holland, 2012)



Figure 2.6 Corrosion of a bent cap and beam end due to the presence of joints (MTO, 2013, Howard Holland, 2012)

2.2.1 Degradation of Concrete Bridges

According to Bertolini (2004), the degradation process of concrete can be classified as physical, mechanical, chemical or structural. Physical degradation is caused by thermal variations such as freeze thaw cycles. Mechanical degradation is caused by abrasion or erosion caused by external sources such as waves or vehicle impact. Chemical degradation is caused by an attack of acids, sulfates, ammonium and magnesium ions or alkali aggregate reactions. Structural degradation can be caused by overloading or repeated cyclic loading which can lead to fatigue damage. These degradation processes

can lead to concrete cracking and spalling which results in a loss of stiffness and strength of the affected structural members.

2.2.2 Corrosion of steel reinforcement

Corrosion of reinforcing steel has been noted as today's leading cause of deterioration of reinforced concrete structures in the US (ASCE, 2013). The two main causes of steel corrosion are carbonation and chloride ingress. These mechanisms are unusual in that they do not attack the integrity of the concrete, instead they pass through the concrete and attack the steel (Broomfield, 1996). Corrosion attack on the steel results in a loss of steel cross section, ductility and strength. It can also lead to cracking and delamination of the cover concrete due to the expansive action of the corrosion products. Finally, it can lead to a loss of bond strength between the rebar and the concrete. The mechanism itself is generally accepted as being electrochemical in nature (Liu and Weyers, 1998b, Bazant, 1979). The steel reinforcing bars embedded in concrete are protected by a thin protective iron oxide film (passive film) which is formed during the hydration of cement due to the highly alkaline environment. This passive film protects the steel from corrosion but can be destroyed by carbonation of concrete or by the presence of chloride ions and once this occurs, corrosion is initiated.

The corrosion process is generally considered to be characterised by two distinct phases according to the model developed by Tutti (1979), which has been reproduced in Figure 2.7. The first phase is the initiation of corrosion. During this phase the aggressive substances (CO_2 , chlorides) penetrate the concrete cover from the surface until they reach and destroy the passive film (Bertolini et al., 2004). The second phase is known as the propagation phase and begins when the protective passive film is destroyed. If oxygen and moisture are present at the surface of the reinforcement, corrosion will occur. From the figure it can be seen that this phase finishes when a limit state is reached beyond which consequences of corrosion cannot be tolerated (Bazant, 1979, Tutti, 1979). The following two sections discuss the corrosion process due to carbonation and chlorides.

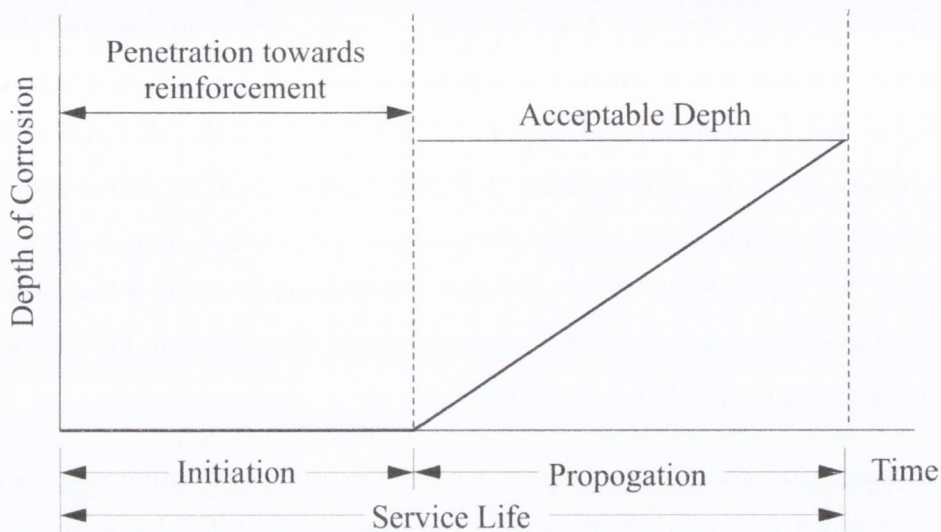


Figure 2.7 Two stage chloride induced corrosion model

2.2.2.1 Carbonation-Induced Corrosion

Carbonation occurs in moist environments when carbon dioxide present in the air dissolves in water to form a carbonic acid that can neutralize the normally alkaline environment at the level of the steel rebar, which protects the steel from corrosion. Once the passive film is destroyed, corrosion of the steel will occur once water and oxygen are present. A more aggressive form of corrosion occurs if chlorides are released into the pore solution. This may occur if a small amount of chlorides are present in the concrete due to either the use of raw materials containing these ions or the penetration of chlorides from the external environment (e.g. seawater, de-icing salts) (Bertolini et al., 2004).

The rate of carbonation depends on environmental factors such as humidity, temperature and carbon dioxide concentration, as well as the concrete characteristics such as its alkalinity and permeability (Bertolini et al., 2004). It is generally agreed by researchers that a maximum carbonation rate is achieved at a relative humidity of somewhere between 60% and 70% (Sergi, 1986). This is because the carbonation reaction only occurs in the presence of water so it becomes negligible in dry concrete. On the other hand, the diffusion of CO_2 is negligible if the concrete pores are completely saturated with water. It has been noted by Bertolini et al. (2004) that the

carbonation rate may vary from one part of a structure to another and that if one area of the structure is permanently sheltered, the rate of carbonation will be considerably higher than parts exposed to rainfall. In the case of a bridge, this may apply to the RC columns. The rate of carbonation will also increase with an increase in atmospheric CO_2 and temperature. In terms of the concrete composition, there is agreement that the lower the permeability of the concrete, which is achieved by a low water to cement (w/c) ratio, the lower the carbonation rate. More discussion on the rate of carbonation can be found in Bertolini et al. (2004).

Although carbonation may be prevalent in inland regions with no exposure to marine environments or chloride attack from the application of de-icing salts, there is general agreement (Saassouh and Lounis, 2012, Saraswathy and Song, 2007, Angst et al., 2009) that chloride induced corrosion is the most widespread and serious cause of deterioration of RC bridges worldwide. For this reason, chloride induced corrosion is chosen as the main deterioration mechanism for this thesis. That being said, recent work (Yoon et al., 2007, Stewart et al., 2011, Stewart et al., 2012, Wang et al., 2012) has suggested that increasing levels of atmospheric CO_2 and increasing temperatures worldwide due to climate change, may increase carbonation rates of RC structures. This may lead to carbonation becoming an issue in urban areas with high CO_2 levels and may need to be considered in future work.

2.2.2.2 Chloride-Induced Corrosion

Chloride contamination of concrete is caused by either contaminated raw materials used in the manufacture of concrete or from the environment. Environmental causes of chloride contamination include marine exposure and the application of de-icing salts. Similar to carbonation, chlorides in the concrete lead to the breakdown of the protective passive layer surrounding the steel. However, unlike carbonation which leads to a homogenous reduction in steel area along the length of the rebar in the region exposed, chloride-induced corrosion is generally characterised by pitting corrosion (Bertolini et al., 2004). Pitting corrosion occurs when chlorides lead to a local breakdown of the passive film. Areas that are no longer protected by the film act as anodes with respect to the still passive surrounding areas where the cathodic reaction of oxygen reduction takes place (Bertolini et al., 2004). Chloride initiation occurs when

the chloride content at the level of the reinforcing steel reaches a threshold or critical chloride content, C_{cr} . The initiation of corrosion depends on the rate of penetration of chloride ions through the concrete cover. In order to predict the initiation of corrosion, information on the chloride threshold and the diffusion of chlorides through the concrete is needed. Once corrosion has initiated, the loss of steel cross section will depend on the rate of corrosion, i_{corr} . Although it is noted here that chloride induced corrosion is characterised by pitting corrosion, uniform corrosion is assumed as a simplification following previous studies on the ageing fragility of bridges (Choe et al., 2008, Choe et al., 2009, Simon et al., 2010, Ghosh and Padgett, 2010, Alipour et al., 2011). An opportunity for future research would be to consider the effects of pitting corrosion in the model. Further discussion on the probabilistic modelling of chloride induced corrosion can be found in Chapter 4.

2.2.3 Foundation Settlement

Depending on the local soil and environmental conditions, differential settlement may occur between piers inducing forces in the superstructure when it is integral with the substructure (INDOT, 2010). According to a survey carried out on 314 bridges across the US and Canada (FHWA, 1985), 22% of piers surveyed experienced vertical movement. The majority (65%) of these piers experienced less than 4.9 cm of vertical movement; however, some bridges experienced movements of up to 20 cm. Although these findings suggest that the problem of settlement is much less widespread than the problem of reinforcement corrosion, its effect on the seismic response of a bridge remains a worthwhile study. To the author's knowledge, the effect of foundation settlement on the seismic performance of highway bridges remains unaddressed in the literature. Further discussion on the approach used in this thesis to model settlement effects on the seismic response of integral bridges can be found in Chapter 4.

2.3 Seismic Risk Assessment Techniques for Bridges

In seismic regions, it is of great importance to attempt to predict the potential damages and losses due to future earthquakes and their probabilities of occurrence. This is particularly vital so that the relevant authorities can plan for the allocation of resources

in the aftermath of an earthquake, making the recovery as fast and efficient as possible. Since bridges are key components of the infrastructure system, their full service in the aftermath of an earthquake may be essential for the use of emergency vehicles. This is one of the reasons why a significant amount of research has been carried out on the seismic risk assessment of bridges.

In order to carry out a Probabilistic Seismic Risk Assessment (PSRA), several steps must be followed, as outlined in Figure 2.8. This figure shows the PSRA framework presented by the Pacific Earthquake Engineering Research Centre (Baker and Cornell, 2006). From the figure it can be seen that seismic risk assessment is a multi-disciplinary problem that requires input from several areas of expertise. It requires an input from seismology and geotechnical engineering to quantify the ground shaking the structure may experience at its base; structural engineering to quantify the response of the structure and the resulting damage; and finance, public policy and construction cost estimating to help determine the social and economic consequences of the damage (Baker and Cornell, 2006). The strategy employed in this PSRA framework is to deconvolve the uncertainty in different parts of the seismic risk assessment problem such as the seismic hazard, structural performance (response and damage) and consequences (financial loss, interruption time) using the theorem of total probability, in an effort to achieve a consistent reliability-based approach for decision making (Cornell and Krawinkler, 2000; Mackie and Stojadinovic, 2005). It is noted that the process is modular, with each of the assessment modules being independent and linked together by pinch point variables (Kaplan and Garrick, 1981). In the framework outlined in Figure 2.8, the pinch point variables are the Intensity Measures (IM), the Engineering Demand Parameters (EDP) and the Damage Measures (DM). The final consequence is known as the decision variable and could also be termed a pinch point variable (Baker and Cornell, 2006). An important assumption of this methodology is that each step of the analysis depends only on the values of the previous pinch point variable and not on the scenario by which it was reached. For example, the response of the structure depends only on the earthquake intensity measure and not on variables such as the magnitude or distance of the earthquake (Baker and Cornell, 2006). Furthermore, the process is Markovian, for example, given knowledge of the EDP, the damage to building elements is independent of the IM (Baker and Cornell, 2006).

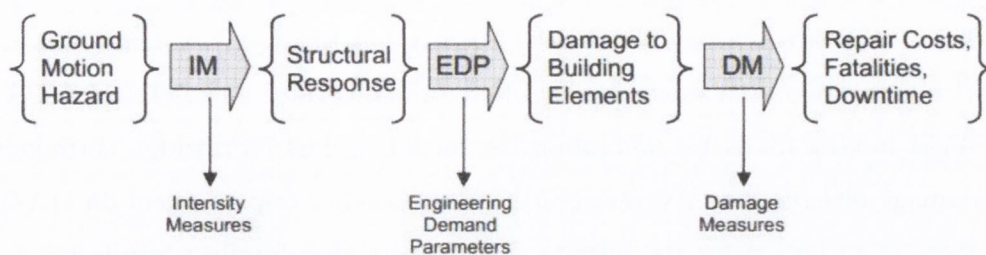


Figure 2.8 Schematic representation of seismic risk assessment method (Baker and Cornell, 2006)

One of the earliest attempts to formalize the seismic risk assessment process was carried out by Whitman et al. (1975). This work introduced Damage Probability Matrices (DPM) based on the 1971 San Fernando (and other) data (Lomnitz and Rosenblueth, 1976). It provided an organised framework for considering the ground motion hazard, building damage, direct and indirect costs. Since then, building on this framework, several groups have attempted to quantify the seismic risk of infrastructure systems. The Applied Technology Council (ATC) went on to create a basic package for the seismic risk assessment of infrastructure in California (ATC, 1985). This research used damage probability matrices and restoration functions which were based on expert opinion due to the lack of available earthquake data. In 1991, the ATC improved on this report by changing the damage matrices to continuous damage functions known as fragility curves (ATC, 1991).

A further step forward was made by the Federal Emergency Management Association (FEMA) in 1997, which put together a Geographical Information System (GIS) based risk assessment software package. The software was known as "HAZards US" or "HAZUS". This package was based heavily on the methodology and data that was presented in the ATC-13 report, which was limited in the sense that the damage probability data and restoration data was dependent on expert opinion. Several updates and improvements were made to the subsequent versions of HAZUS. In 2006, the Multidisciplinary Centre for Earthquake Engineering Research (MCEER), under the sponsorship of the FHWA, developed a new methodology for seismic risk analysis of

highway systems. Under this project, a public domain software package named REDARS 2 (Risks from Earthquake Damage to Roadway Systems) was developed. This software used a similar approach to estimate bridge damage to HAZUS but made slight modifications by calibrating the model against Northridge earthquake bridge damage observations (Werner et al., 2006). Another improvement on HAZUS is that there is an option for the user to submit their own fragility results for a particular bridge. This means that if the bridge under consideration is unusual and not adequately represented by the default models, reliable results can still be obtained from the program.

According to the risk assessment methodology presented in Figure 2.8, one of the key links in the methodology is to estimate the damage to structural components. This is carried out by estimating the damage to a structure as a function of ground motion intensity level using tools such as damage probability matrices or fragility curves. The following sections describe the current methods used to develop seismic fragility curves, since this thesis uses these tools to assess the seismic vulnerability of ageing bridges.

2.4 Evolution of Fragility Curve Development

Over the years fragility curve development has evolved with the availability of earthquake damage data and the improvement in computing power. The first ATC-13 report (ATC, 1985) relied on "expert opinion" to develop the damage probability functions due to the lack of available data from previous earthquakes. The ATC put together a panel of 42 experts, of which four were chosen to provide information on highway bridges. The experts were surveyed and asked to allocate a given structure to one of seven damage levels for a given ground motion intensity level based on the Modified-Mercalli Intensity scale. The DPMs were developed based on the results of the survey. These DPMs were subsequently used to develop fragility curves for the ATC-25 report (ATC, 1991). This methodology was obviously flawed due to its subjectivity and lack of correlation to actual earthquake damage reports. The reliability of the DPMs for highway bridges was also dependent on the expertise and knowledge of the four experts.

The 1989 Loma Prieta, 1994 Northridge and 1995 Kobe earthquakes provided a more complete data for the development of empirical fragility curves. Several researchers (Elnashai et al., 2004, Shinozuka et al., 2003, 2000b, Der Kiureghian, 2002, Yamazaki et al., 1999, Basoz and Kiremidjian, 1997) used this data to develop empirical fragility curves. Although differences exist between different researchers in the approach used, the methods are conceptually the same. The basic method involves assigning all bridges of a particular class to one of five damage states based on an inspection of post earthquake damage. The data is then organised based on the ground motion intensity level experienced by each bridge which is estimated from a shake map. The resulting DPMs provide an empirical basis for estimating the probability of a given damage state occurring, for a given bridge class and earthquake intensity level. Basoz and Kiremidjian performed a logistic regression analysis on the DPMs to develop fragility curves while Shinozuka et al. (2003) used the Maximum Likelihood Method to estimate the parameters of a lognormal probability distribution describing the fragility curves. Der Kiureghian (2002) used a Bayesian approach to develop fragility curves. This method had a number of shortcomings including a lack of adequate damage data, subjectivity of the damage state assessment and variations in the reported ground motion intensity levels at a given site depending on the source of the shake maps.

The third method for developing fragility curves uses analytical and simulation based techniques. While computing power a few decades ago may not have been sufficient to analyse complicated models, in recent years, advances in modelling capabilities have made it possible to develop fragility curves for bridges in areas without sufficient earthquake data. Many researchers have developed analytical fragility curves for highway bridges using a wide variety of different methodologies. The following sections describe the various analytical methods used to develop fragility curves. These methods include linear dynamic analysis, nonlinear static analysis and nonlinear dynamic analysis.

2.4.1 Linear Dynamic Analysis

One of the main forms of linear dynamic analysis, which has been used for the development of fragility curves, is Response Spectrum Analysis (RSA). RSA is one of the most simple and time efficient methods for estimating the maximum seismic

response of a structure for a given ground motion. The analysis requires either a ground motion response spectrum based on a single ground motion record or a design response spectrum which is a smooth representation of many response spectra. A response spectrum gives the maximum response of a single degree of freedom system (SDOF) (expressed as spectral displacement, velocity or acceleration) for a given natural period and level of damping. Figure 2.9 shows acceleration response spectra for the El-Centro earthquake with 4 levels of damping (Irfanoglu, 2012). Once the maximum response of the SDOF is obtained from the response spectrum, the maximum response for the multi degree of freedom system (MDOF) is found through superposition of the maximum modal responses, which is valid as long as the inelastic deformations are small. This is carried out using a modal combination rule such as the absolute sum (ABS) (Chopra, 2007), square-root-of-sum-of-squares (SRSS) (Rosenblueth, 1951) or complete quadratic combination (CQC) (Der Kiureghian, 1981). The method assumes linear behaviour of the structure and is therefore only applicable to bridges that are expected to remain within the elastic range based on cracked section properties. One of the limitations of this method is that when nonlinear behaviour occurs, the approach under predicts the displacement demand and over predicts the force demand.

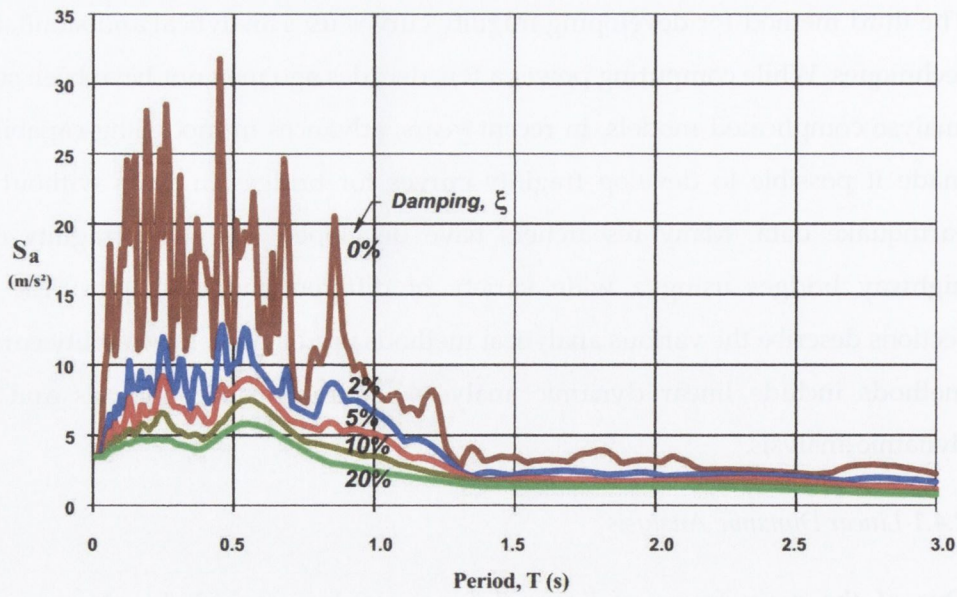


Figure 2.9 Acceleration response spectra of El-Centro 1940 earthquake with 2%, 5% and 10% damping (Irfanoglu, 2012)

RSA has been used for the development of fragility curves by Yu et al. (1991) and Hwang et al. (2000). Yu et al. (1991) used the approach to assess the seismic vulnerability of highway bridges in Kentucky. Each of the bridge piers was modelled as a SDOF system and their response was estimated using an elastic response spectrum. This approach was further developed by Hwang et al. (2000), which evaluated capacity/demand ratios for vulnerable bridge components and correlated these ratios to a particular damage state. The capacities were estimated following FHWA (1995) and the seismic demand was determined from an elastic spectral analysis according to the approach specified by AASHTO (1996). This was done for varying levels of PGA and the results were put into a damage frequency matrix and the resulting fragility curves were developed.

2.4.2 Non-Linear Static Analysis

Where linear dynamic analysis is inappropriate, for example, when it is expected that a bridge may experience inelastic behaviour under earthquake loads, nonlinear static analysis considers the non-linear response of a bridge but is less time consuming than a full non-linear dynamic analysis. The Capacity Spectrum Method (CSM) is a type of nonlinear static analysis, originally proposed by Freeman et al. (1975) and is the main method used in ATC-40 (1996). The CSM procedure compares the capacity of a structure (in the form of a pushover curve) to the demands on the structure (in the form of a response spectrum) (Freeman, 2004). The non-linear static pushover curve in the form force versus displacement is converted to a capacity spectrum in the form of spectral acceleration versus spectral displacement. The resulting spectrum is shown in Figure 2.10 (Dutta, 1999).

The seismic demand is then represented by a response spectrum plotted in the acceleration-displacement response spectrum format (ADRS). This is an alternative to the traditional spectral acceleration (S_a) versus period (T) format and is used for simple interpretation of the capacity spectrum and the superimposed response spectrum. An example of an ADRS response spectrum can be seen in Figure 2.11. From the figure it can be seen that the period (T) is represented by radial lines from the origin. A method for carrying out this conversion is described in Fajfar (1999). The elastic response

spectrum is then adjusted to account for non-linear behaviour by including high levels of damping up to 30% (Freeman, 2004).

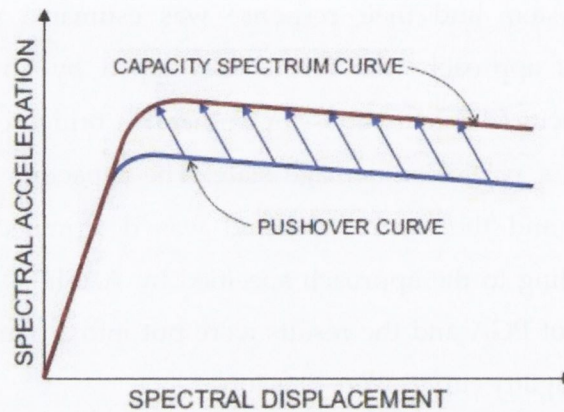


Figure 2.10 Conversion of the pushover curve to the capacity spectrum (Dutta, 1999)

Once the response spectrum is superimposed onto the capacity spectrum, the maximum or performance response for a deterministic analysis would be the point of intersection of the two curves. Of course, for a probabilistic analysis, there is uncertainty about the median demand and capacity as shown in Figure 2.12. Seismic fragility curves can then be generated from these spectra by comparing predefined capacity distributions at various damage states with demand distributions. This is carried out at various levels of a chosen intensity measure.

Due to its time efficiency, CSM has been used by many researchers (Moschonas et al., 2009, Banerjee and Shinozuka, 2007, Shinozuka et al., 2000b, Mander and Basoz, 1999, Dutta, 1999) to generate fragility curves for US highway bridges. The current fragility curves employed in HAZUS-MH are those proposed by Mander and Basoz (1999). Despite this, it must be noted that the guidelines found in ATC-40 on the use of CSM are specific to concrete buildings and very little guidelines exist on its application to bridges (Ramanathan, 2012). Another controversial element of CSM is the use of highly damped elastic spectra to estimate the maximum response of the bridge. Fajfar (1999) proposed a modification to CSM in the form of the N2 method which uses inelastic

demand spectra in ADRS format instead of elastic demand spectra with equivalent damping. In recent years, the N2 method has been implemented in European regulations (Eurocode 8, 2004). Gardoni et al. (2003) and Zhong et al. (2008) proposed a modification to the N2 method to aid in the development of probabilistic seismic demand models (PSDMs). Another variation on CSM is the Coefficient Method which is used in FEMA-273 (1997).

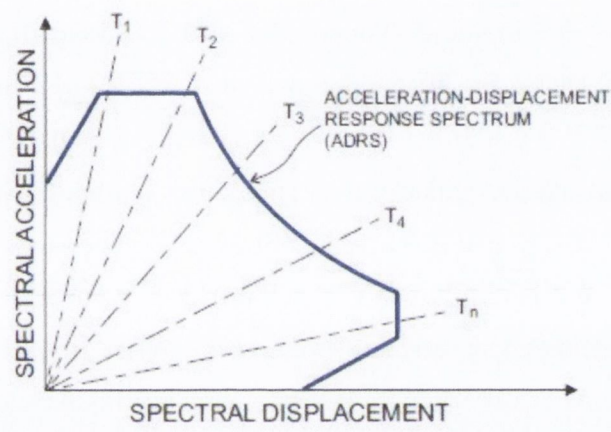


Figure 2.11 Acceleration Displacement Response Spectrum (Nielson, 2005)

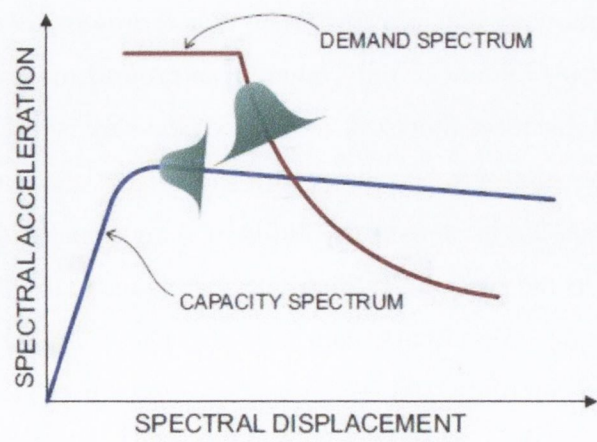


Figure 2.12 Probabilistic representation of demand and capacity spectra (Mander and Basoz, 1999)

2.4.3 Non-linear Dynamic Analysis

Nonlinear dynamic analysis or nonlinear time history analysis (NLTHA) is the most computationally expensive method for developing seismic fragility curves, however, it is also one of the most reliable methodologies available (Shinozuka et al., 2000b). This is due to the fact that it accounts for the nonlinear behaviour of the bridge neglected in linear methods and captures the dynamic demands on the structure more accurately than static methods. Due to its reliability, it has been used by many researchers for fragility curve development (Ramanathan, 2012, Ramanathan et al., 2010, Padgett, 2007, Nielson, 2005, Kim and Shinozuka, 2004, Mackie and Stojadinovic, 2005a, 2001). NLTHA offers the flexibility to consider analytical models with linear or nonlinear cyclic material characteristics and geometric nonlinearities such as P- Δ effects (Ramanathan, 2012). Using NLTHA, PSDMs are generated which are mathematical relations between the earthquake intensity and the structural response. These models provide best estimates of seismic demand for a given ground motion intensity as well as considerations of uncertainty. The two main methods used for the development of PSDMs, which predict the seismic demand as a function of ground motion, are incremental dynamic analysis (IDA) and the "cloud" approach (Mackie and Stojadinovic, 2005b).

The procedure to formulate PSDMs using the "cloud" approach (Baker and Cornell, 2006) consists of four steps as outlined in Figure 2.13. The first step is to gather a suite of N ground motions that is appropriate and representative of the target geographic area and captures the uncertainty inherent in ground motions such as the magnitude and epicentral distances (Nielson, 2005). The next step is to generate N nonlinear finite element bridge models to be paired with each of the ground motion records selected. These models should be statistically different yet nominally identical and are generated by sampling on the probability distributions of uncertain structural parameters (e.g. material strength, soil stiffness, damping etc). The N bridge samples are subjected to the N ground motions and the maximum structural component responses are recorded. The PSDM is generated by carrying out a linear regression of the maximum structural response on the earthquake intensity. This approach gives the user a median value for structural demand as well as a standard deviation about that value for a

given ground motion intensity level. These probabilistic demand models can be convolved with probabilistic capacity models to develop fragility curves. This process is described further in Chapter 5. One of the main disadvantages of this method however is that an apriori assumption is made that the demand is lognormally distributed for a given earthquake intensity and that the distribution is independent of the earthquake intensity.

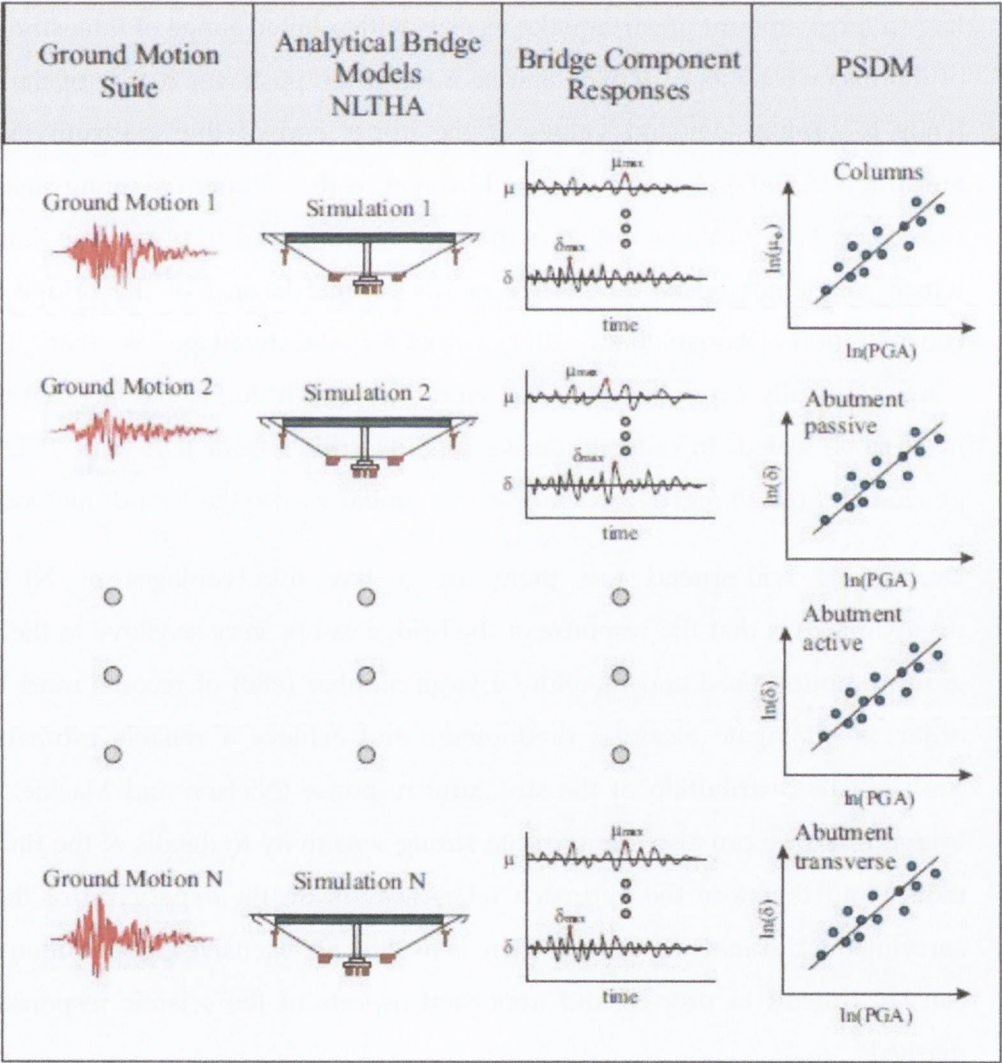


Figure 2.13 Schematic representation of the NLTHA procedure used to develop PSDMs (Ramanathan, 2012)

IDA was developed by Vamvatsikos and Cornell (2002) and uses a very similar approach to the "cloud" method. The only variance between the two is the first step involving ground motion selection. For IDA, a smaller number of ground motion records are chosen and the ground motions are scaled incrementally to the same level of intensity until significant strength reduction of the structural component occurs (collapse). In this sense, IDA generates a dynamic pushover curve comparable to the familiar static pushover curve. The method is useful for seismic regions that do not have a large amount of earthquake records with a broad range of intensity levels. One of the major benefits of IDA is that because of the pushover nature of the method, it tends to capture demand values at the upper end of the spectrum (Mackie and Stojadinovic, 2003). A major concern however, is the validity of scaling small intensity records by large scale factors. It is thought this may lead to unrealistic time histories, which might not be representative of the seismic hazard of the bridge site under consideration (Ramanathan, 2012). Another disadvantage is that it is more computationally expensive than the "cloud" approach due to the fact that the records need to be scaled. In order to derive fragility curves from IDA data, PSDMs can be generated through regression analysis in a similar way to the "cloud" method.

Despite its widespread use there are a few disadvantages of NLTHA. One disadvantage is that the response of the bridge can be very sensitive to the individual ground motion used and therefore a large number (>80) of records must be used in order to propagate aleatoric randomness and achieve a reliable estimation of the probabilistic distribution of the structural response (Nielson and Mackie, 2009). The bridge response can also demonstrate strong sensitivity to details of the finite element model and therefore the approach relies heavily on the experience of the engineer carrying out the analysis. Finally, there is usually an extensive amount of output which can be difficult to process and important aspects of the seismic response could be masked.

2.5 Review of Current Research on the Ageing Fragility of Bridges

In all of the current seismic risk assessment packages described in Section 2.4, fragility curves are developed based on bridges in their pristine or as-built condition. Recalling

Section 2.3, it was noted that the average bridge age in the US is 43 years, which is close to the average design service life of 50 years (ASCE, 2013). Since the deterioration mechanisms described in Section 2.3 affect the main force resisting components of a bridge, it is reasonable to assume that deterioration significantly affects the seismic fragility of a bridge. Therefore, the use of fragility functions based on bridges in their pristine condition may lead to unreliable loss estimates following the current risk assessment methods.

Very recently, a limited amount of research has been carried out attempting to quantify the effect of deterioration on the seismic performance of bridges. Most of this research has focussed on RC column deterioration, with some work focussing on how deterioration impacts the seismic capacity of the columns and others studying how deterioration impacts the seismic demand. In an attempt to estimate the corroded seismic capacity of columns, Choe et al. (2008) extended previously developed probabilistic drift and shear capacity models for pristine columns (Gardoni et al., 2002) to account for a loss of steel cross section. Using these deteriorated capacity models, seismic fragility estimates were developed for a RC corroded column in a single bent bridge. Choe et al. (2009) extended this work by developing novel probabilistic demand models for corroding RC bridges. By comparing these to the previously developed capacity models (Choe et al., 2008), the corroded bridge fragility was estimated. The only deterioration mechanism considered in these studies was chloride induced corrosion of the RC columns resulting in a loss of steel cross section. Both these studies found that the seismic fragility of RC columns increased over time and with deterioration level. Choe et al. (2010) then went on to develop fragility increment functions, which are used to obtain the fragility of a corroded column at any given time by multiplying the initial fragility of the pristine column by the corresponding increment function. This approach allows the fragility of deteriorated columns to be estimated without any extra reliability analyses, once the fragility of the pristine column is known.

Simon et al. (2010) investigated the impact of concrete cover spalling and a loss of steel cross section on the seismic fragility of a RC column in a single bent bridge. It was concluded that a 10% loss of steel area and a loss of stiffness due to a loss of concrete

cover only marginally influences the seismic fragility of the selected bridge. This can be seen in Figure 2.14 which shows fragility contours for the collapse prevention limit state, conditioning on the spectral acceleration in the longitudinal direction, $S_{a,x}$, and transverse direction, $S_{a,y}$. In the figure, FCFB represents the column with full concrete cover and reinforcement area, FCPB represents the column with full concrete cover and a 10% loss of reinforcement area, NCFB represents the column with no concrete cover and undeteriorated reinforcement and NCPB represents the column with no concrete cover and a 10% loss of reinforcement area. From the figure it can be seen that the FCFB consistently has the lowest probability of failure and the NCPB case almost consistently has the highest probability of failure. However, the authors of the study note that the overall differences in the fragilities among the four analysis cases are relatively insignificant.

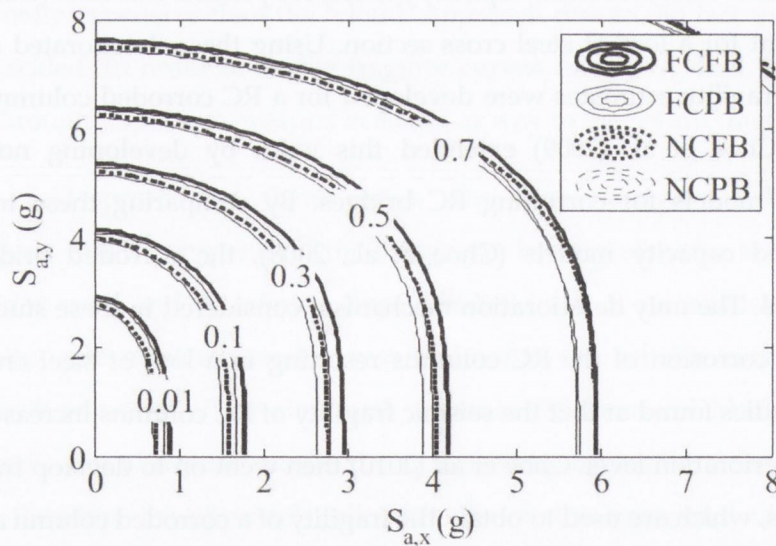


Figure 2.14 Fragilities at the complete prevention damage state (Simon et al., 2010)

Akiyama et al. (2011) integrated the probabilistic hazard associated with airborne chlorides into the seismic reliability of RC bridges and found that the seismic reliability of RC bridge piers can be dramatically affected by the presence of airborne chlorides. In this study, the displacement ductility capacity for a corroded RC column was

evaluated based on the buckling model of longitudinal rebars. The bridge pier under consideration was modelled as a single degree of freedom (SDOF) structure and therefore, the true demands on the RC column may not have been captured. In order to assess the impact corrosion has on the seismic demand of a bridge, a nonlinear dynamic analysis is required.

Ghosh and Padgett (2010) developed time dependent fragility curves for multi-span steel girder bridges, which broadened the focus to system fragility, considering the columns, abutments and steel bearings as components in the bridge reliability analysis. This study considered deterioration of both the RC columns and the steel bearings. This paper's findings are consistent with the previously mentioned papers, showing that seismic vulnerability increases throughout the lifetime of the bridge. This is illustrated in Figure 2.15, which shows the bridge system fragility curves at various points along the service life of the structure. From the figure it can be seen that the fragility of the bridge increases with time. The median fragility estimate shifts by 32% after 100 years. In this case, the median fragility estimate corresponds to the earthquake intensity level at which there is a 50% probability of bridge system failure.

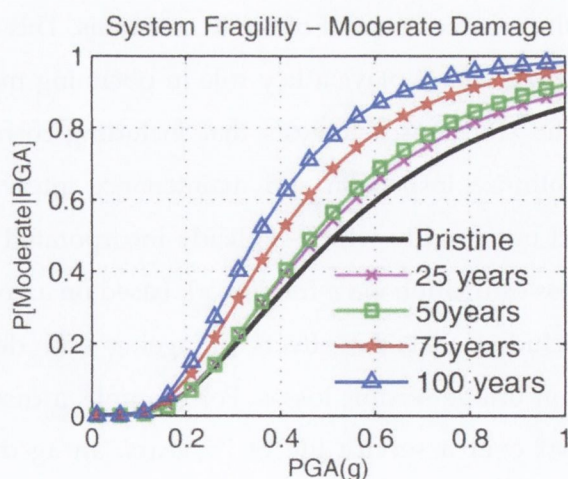


Figure 2.15 System fragility curves at the moderate damage state (Ghosh and Padgett, 2010)

Ghosh and Padgett (2012) extended this research by developing time dependent fragility curves for ageing reinforced concrete girder bridges, considering deterioration of the elastomeric bearings along with the RC columns. Additionally, this work assessed the relative sensitivity of the bridge fragility to the severity of the exposure to chlorides. The results show that exposure to de-icing salts over the 100 year time period results in a 44% shift in the median fragility estimate. However, it was also found that for the case of marine exposure, there was only a 5% shift in the median fragility. These results highlight the importance of accurate deterioration modelling, considering local environmental conditions at the bridge site. This topic will be discussed in Section 4.1. Both these studies found that the inclusion of multiple bridge components is necessary for the accurate estimation of seismic bridge fragility. In addition, both studies focus only on the increase in seismic demand as a result of deterioration; however, they significantly neglect the effect of deterioration on the seismic capacities of the considered components.

Since bridge design should take into account the cost effectiveness of investments throughout the bridge lifetime, several researchers have carried out life cycle cost analyses, accounting for the effects of deterioration. Alipour et al. (2011) carried out a life cycle cost analysis on a group of deteriorating bridges, focussing on the effect of deterioration on the seismic demand of the RC columns. This study concludes that the inclusion of corrosion effects plays a key role in obtaining more realistic estimates of the total costs. The study also suggests that including corrosion provides valuable information to optimize inspection and maintenance intervals. Ghosh and Padgett (2011) also carried out a study which explicitly incorporated time dependent seismic vulnerability in loss estimation via a framework based on a non-homogeneous Poisson process and concluded that the effects of ageing and deterioration can have a significant effect on bridge seismic losses. For example, a case study presented in the paper showed that over a service life of 75 years, an aged multi span continuous concrete bridge shows a 14% increase in total seismic losses, relative to the case where deterioration is neglected.

Although few earthquakes will occur over the lifetime of a bridge that will produce significant and noticeable damage requiring extensive repairs, in high seismic areas, a

given bridge may experience many more minor earthquakes. Although these earthquakes may go unnoticed in terms visible damage, they result in progressive degradation which causes an accumulation of damage over time. Furthermore, it must be acknowledged that columns that have been damaged and cracked by previous earthquakes may experience higher rates of chloride induced corrosion than fully intact columns. Several researchers have studied the combined effect of corroding columns as well as cumulative seismic damage. Kumar et al. (2009) presents a probabilistic approach to compute the life-cycle cost of bridges accounting for cumulative seismic damage and corrosion of the RC column reinforcement. The results of this study found that cumulative seismic damage affects the reliability of bridges over time more than corrosion of the columns. Although this type of progressive degradation is beyond the scope of this thesis, it presents an opportunity for future work in the area.

Although the papers mentioned provide a basis for the development of time dependent seismic risk assessment, more work is needed in the area. Most of the work to date has focussed on the effect of deterioration on either the seismic demand or the seismic capacity, whereas the fragility formulation herein is posed to consider the influence of ageing on both the component capacities and demands. This is important since conditions such as corrosion of reinforcing steel have been shown to not only affect the resistance of key components, but also the overall dynamic behaviour and demands on the structure. Furthermore, most of the work to date, with the exception of Ghosh and Padgett (2010, 2012), has focussed only on column fragility. However, the fragility formulation herein considers the entire bridge system, including the abutments, deck and columns. Additionally, there have been no studies to the author's knowledge on the ageing fragility of the integral bridge type. Finally, most of the work to date has focussed on the effect of chloride induced corrosion of the concrete columns on bridge fragility. This study also investigates the effect of deck corrosion and foundation settlement on the time dependent seismic fragility of integral bridges. Therefore, this thesis explores the relative need to account for such additional forms of ageing when evaluating bridge vulnerability to seismic loads.

CHAPTER 3 - ANALYTICAL BRIDGE MODELLING AND DETERMINISTIC BRIDGE COMPONENT RESPONSES

Recent advances in analytical modelling techniques coupled with a lack of earthquake damage data from previous earthquakes has resulted in an increase in the development of analytical or simulation based fragility curves. As described in the previous chapter, one of the most accurate and reliable methods of carrying out a bridge fragility analysis is through the use of nonlinear time history analysis. This thesis carries out nonlinear time history analyses using three dimensional finite element models, accounting for geometric and material nonlinearities. The finite element model is generated using the finite element platform OpenSEES (McKenna and Fenves, 2010). The OpenSEES model is used to simulate earthquake ground motions and the bridge response data is used to develop probabilistic seismic demand models (PSDMs). PSDMs provide a probabilistic model for the seismic demand on a bridge component for a given ground motion intensity level. These demand models are then convolved with capacity models to develop fragility curves.

This chapter presents a detailed description of the class of integral bridges under investigation as well as the analytical modelling procedures adopted for all of the bridge components. An eigen value analysis is then performed for each of the bridge geometries as well as a deterministic time history analysis to provide insight into the relative response of the various components and to provide a sanity check.

3.1 Three-Span Fully Integral Reinforced Concrete Bridges - Structural Details

As mentioned in Chapter 2, the ageing seismic performance of the integral bridge form is investigated in this thesis. In this context, integral refers to the fact that the bridge superstructure is cast monolithically with the substructure at the abutments and over

the column bents. The integral form is considered in this thesis because of its growing popularity, which can be partly attributed to the fact that it eliminates the need for maintenance associated with joints and bearings. In order to carry out a seismic risk assessment of a bridge, a finite element model of the bridge is generated using the finite element platform, OpenSEES (McKenna and Fenves, 2010). A three span fully integral bridge is modelled using engineering drawings provided by Roughan & O'Donovan Consulting Engineers and can be seen in Figure 3.1. The general layout can be seen in Figure 3.2. The total bridge length is 52 m and the column height is 6.9 m. The superstructure consists of a 10 m wide reinforced concrete deck slab supported on five prestressed U11 beams (Concast, 2012). Composite action of the slab and the girders is taken into account. As mentioned previously, the abutments are fully integral and consist of a single row of seven steel H piles. The bent foundation can be seen in Figure 3.2 and consists of a 1 m deep pile cap and two rows of six steel H piles each.



Figure 3.1 52m Three-span fully integral bridge

In order to investigate the effect of geometric variation on the seismic performance of the integral bridge, the finite element model is adapted to include two different bridge lengths and three column heights, resulting in six geometric bridge samples. The geometric dimensions of the six bridge samples can be seen in Table 3.1. In order to achieve a realistic range for bridge length, the maximum expected length for a 3-span integral bridge is determined based on maximum feasible spans for the prestressed concrete beams considered, based on manufacturers' recommendations (Concast, 2012). These recommendations suggest that the largest capacity U beam available, a U12 beam with a depth of 1.7 m, has a maximum span length of 35 m. This is taken as the centre span and the side spans are calculated based on the centre to side span ratio of the original bridge drawings. The total length of this bridge is 69 m and the individual span lengths can be seen in Table 3.1.

Several design parameters of the base bridge are maintained for all geometric configurations to maintain realistic samples, including the axial load ratio of 9% and the transverse and longitudinal reinforcement ratios of 1% and 3% respectively. This approach was deemed an acceptable approximation in the absence of design drawings. Although the bridge under consideration was not designed to resist seismic loading, the transverse reinforcement ratios are consistent with modern design practice in California according to Ramanathan (2012), which states that for typical box girder highway bridges built after 1990, the longitudinal reinforcement ratios vary from 1% to 3.5% and the transverse reinforcement ratios vary from 0.4% to 1.7%. This is due to the fact that the current columns have been designed to resist a high impact load of 1000 kN in the driving direction and 500 kN in the direction perpendicular to the driving direction, according to the Eurocode (Eurocode 1, 2006). Three column heights are considered for each of the bridge lengths which are shown in Table 3.1 along with other geometric dimensions of the six bridge samples. It is noted that other model parameters are varied for each geometric sample as discussed in Chapter 5.

It must be noted that additional foundation and abutment piles may be required for the 69 m bridge to accommodate the increase in axial load. In order to account for this potential design change, a further fragility analysis is performed in Chapter 6 for the 69 m bridge including an increase in the pile stiffness at the abutments and the

foundations. The results of this fragility analysis will demonstrate whether increasing the number of piles at the foundations and the abutments has a significant impact on the seismic fragility of the 69 m bridge and therefore, whether a potential change in the foundation or abutment design needs to be accounted for in this case.

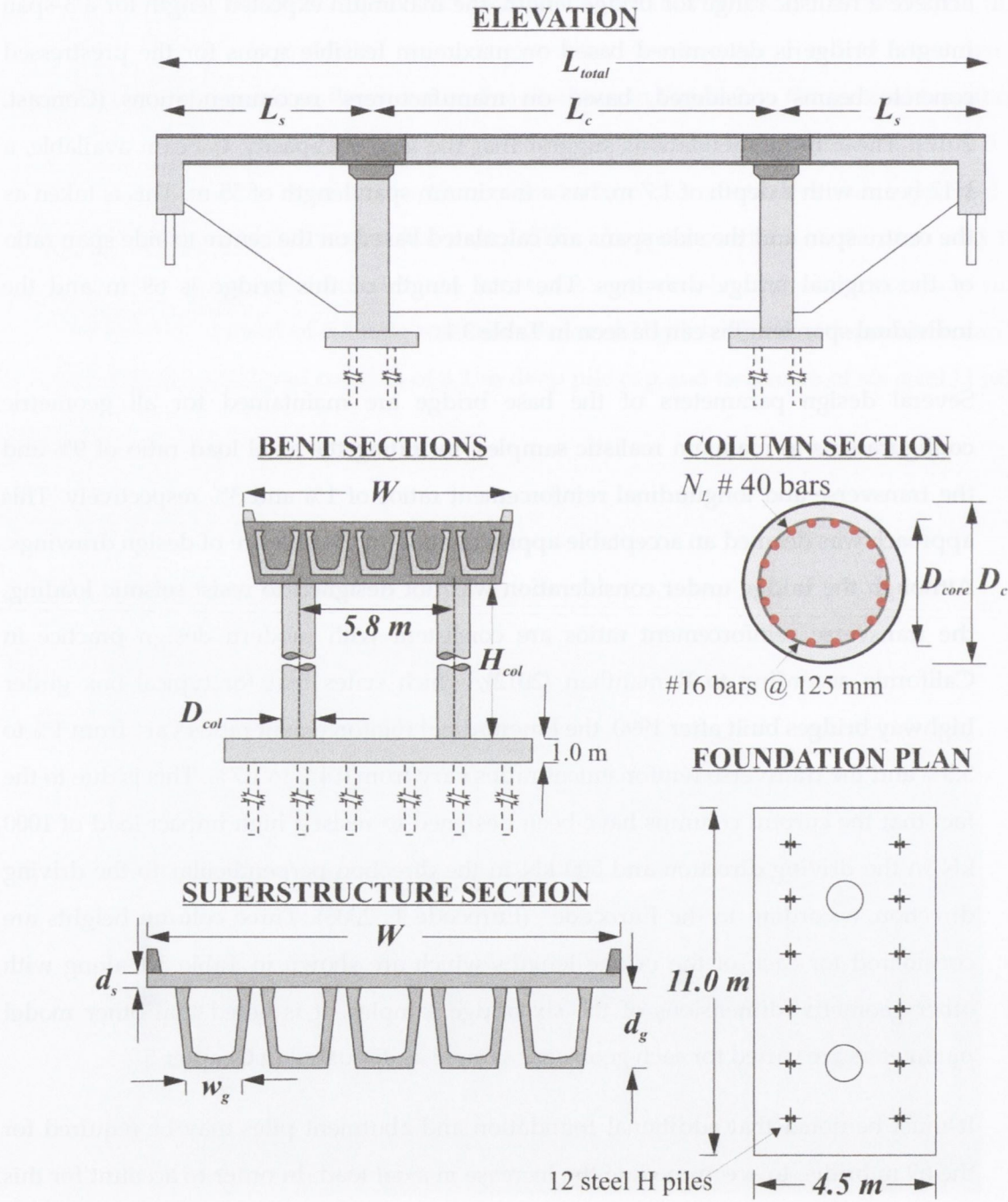


Figure 3.2 General layout of the three-span fully integral bridge class

Table 3.1 Geometric dimensions of 3-span bridges under study

Bridge Number	1*	2	3	4	5	6
Total length (m) (L_{total})	52.0	52.0	52.0	69.0	69.0	69.0
Centre span length (m) (L_c)	29.0	29.0	29.0	38.0	38.0	38.0
Side span length (m) (L_s)	11.5	11.5	11.5	15.5	15.5	15.5
Superstructure width (m) (W)	10.0	10.0	10.0	10.0	10.0	10.0
Slab depth (m) (d_s)	0.2	0.2	0.2	0.2	0.2	0.2
Girder depth (m) (d_g)	1.5	1.5	1.5	1.7	1.7	1.7
Girder width (m) (w_g)	0.97	0.97	0.97	0.97	0.97	0.97
Column height (m) (H_{col})	6.9	9.5	12.0	6.9	9.5	12.0
Column diameter (m) (D_{col})	1.0	1.0	1.0	1.1	1.1	1.1
Column core diameter (m) (D_{core})	0.9	0.9	0.9	1.0	1.0	1.0
Number of longitudinal bars (m) (N_L)	18	18	18	20	20	20

*Base case dimensions shown in bold

3.2 Analytical Bridge Component Modelling Strategies

This section presents the analytical modelling strategies adopted for this thesis. Figure 3.3 shows an overview of the finite element model developed using OpenSEES (McKenna and Fenves, 2010). From the figure it can be seen that a spine model is used with non-linear springs for the abutments and foundations. The following sections present modelling details for each of the bridge components.

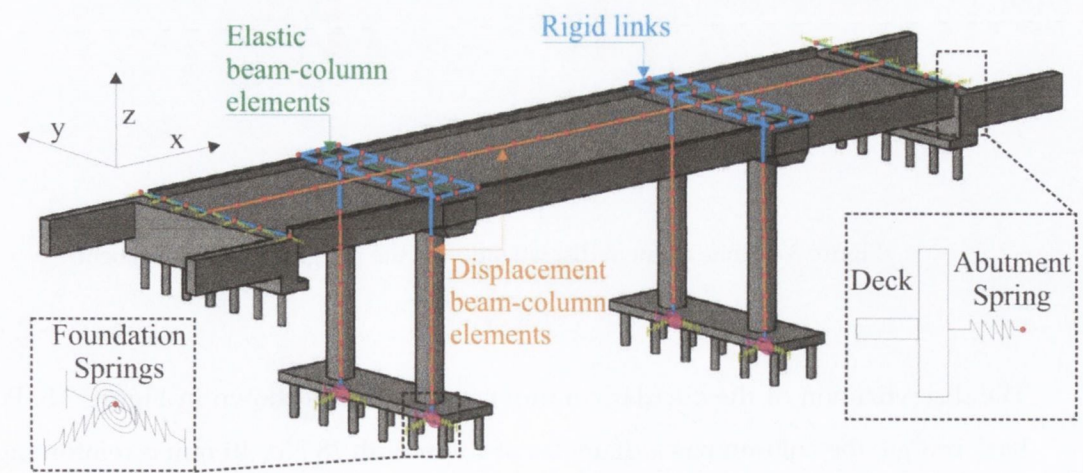


Figure 3.3 Three dimensional nonlinear finite element model of the 3-span integral bridge

3.2.1 Two Column Concrete Bents

The bents for this integral bridge type consist of a solid diaphragm and two circular RC columns. The vertical columns are modelled using displacement based beam-column elements with fiber defined cross sections. The transverse diaphragm is modelled using elastic based beam-column elements with the elastic section properties of the diaphragm assigned. The columns are connected to the diaphragm via rigid links to simulate force and moment transfer between the two. This modelling approach is adopted from Ramanathan (2012) who developed finite element models of typical Californian bridges with similar bent configurations to the bridge under consideration. Figure 3.4 presents the finite element discretization of the integral bridge bent.

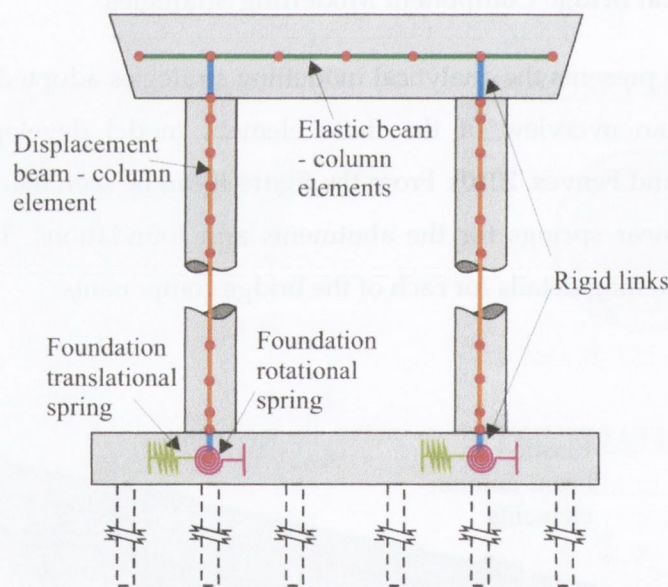


Figure 3.4 Finite element discretization of the integral two column bent

The discretization of the circular column cross section is shown in Figure 3.5. For the base bridge, the column has a diameter of 1.0 m with 18 No. 40 mm $\varnothing$ reinforcing bars and a 50 mm concrete cover. It is divided into 10 rings and 12 wedges following the discretization procedure used by Nielson (2005) to model columns of similar diameter.

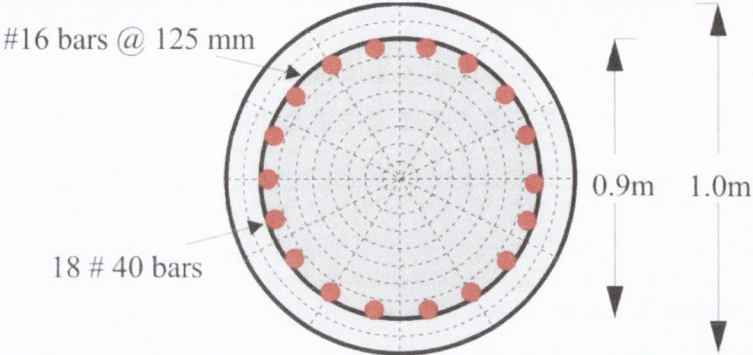


Figure 3.5 Fiber discretization of circular RC column

In order to verify the OpenSees RC column model, a moment curvature analysis of the RC section carried out using OpenSEES is compared with a moment curvature analysis carried out using Cumbia Octave, a program developed by Montjo (2007) using Matlab (The Mathworks Inc., 2010). The results in Figure 3.6 show that there is good agreement between the two moment curvature analyses, with less than a 5% difference between the moment capacities given by each program.

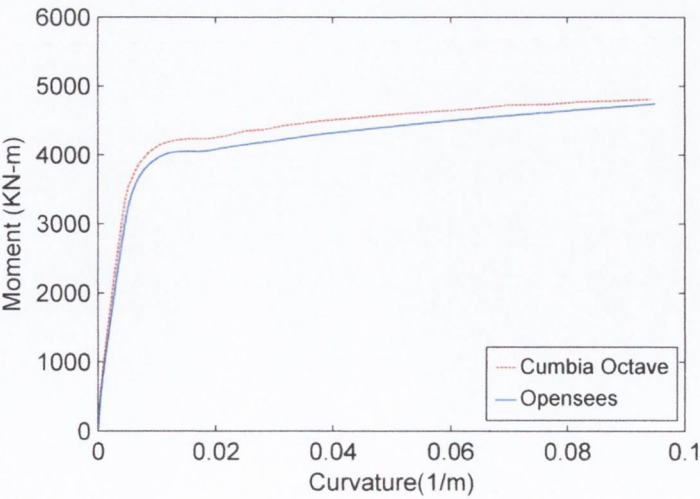


Figure 3.6 Moment curvature relationship of the RC column comparing OpenSEES and Cumbia Octave

It must be noted that the moment curvature plot shown Figure 3.6 does not demonstrate any reduction in strength before failure of the column. This is due to the fact that the *Steel 02* material used to model the reinforcement in the fiber model is bilinear and does not account for strain softening once the ultimate steel strength is reached.

3.2.1.1 Material Models

The major benefit of using fiber defined cross-sections to model the RC columns is that it allows for the specification of different material properties at different locations through the cross section. There are three material models required to generate the column cross-section. These include the confined concrete core, the unconfined concrete cover and the reinforcing steel. Once these materials are defined they can be applied to the precise locations of the concrete core, cover and longitudinal reinforcing bars.

The compressive stress-strain relationships for the confined and unconfined concrete are adopted from Mander et al. (1988) as shown in Figure 3.7. Researchers have shown previously that confinement of concrete by a suitable arrangement of transverse reinforcement provides a significant increase in strength and ductility of compressed concrete (Mander et al., 1988). The longitudinal compressive stress of the confined concrete, f'_c is given by:

$$f'_c = \frac{f'_{cc} x^r}{r - 1 + x^r} \quad \text{Eqn. 3.1}$$

where:

$$f'_{cc} = f'_{co} \left(-1.254 + 2.254 \sqrt{1 + \frac{7.94 f'_l}{f'_{co}}} - 2 \frac{f'_l}{f'_{co}} \right) \quad \text{Eqn. 3.2}$$

$$f'_l = 0.95 \left(\frac{2 f_{yh} A_{sh}}{D_{core} s} \right) \quad \text{Eqn. 3.3}$$

$$x = \frac{\varepsilon_c}{\varepsilon_{cc}} \quad \text{Eqn. 3.4}$$

$$\varepsilon_{cc} = 0.002 \left[1 + 5 \left(\frac{f'_{cc}}{f'_{co}} - 1 \right) \right] \quad \text{Eqn. 3.5}$$

$$r = \frac{E_c}{E_c - E_{sec}} \quad \text{Eqn. 3.6}$$

$$E_c = 5000\sqrt{f'_{co}} \quad (MPa) \quad \text{Eqn. 3.7}$$

$$E_{sec} = \frac{f'_{cc}}{\epsilon_{cc}} \quad \text{Eqn. 3.8}$$

In Eqn. 3.1 - Eqn. 3.8, f'_{cc} is the confined concrete strength; ϵ_{cc} is the concrete strain at the confined concrete strength; f'_{co} is the unconfined concrete strength; f'_l is the effective lateral confining stress provided by the transverse reinforcement; f_{yh} is the transverse steel strength; A_{sh} is the transverse steel area; D_{core} is the diameter of the concrete core between bar centres; s is the centre to centre spacing or pitch of spiral or circular hoops; E_c is the elastic modulus of concrete and E_{sec} is the secant modulus of elasticity. Both the confined and unconfined concrete behaviour are modelled using *Concrete 01* material, as provided in OpenSEES following an approach adopted in the literature (Nielson, 2005, Padgett, 2007, Simon et al., 2010, Cruz Noguez and Saiidi, 2012). This material uses the Kent-Scott-Park model (Scott et al., 1982), which utilizes a degraded linear unloading/reloading stiffness and a residual stress. This material neglects tensile strength. For the confined concrete material the ultimate compression strain is assumed to occur when fracture of the transverse confining steel initiates and is taken as:

$$\epsilon_{cu} = 0.004 + \frac{1.4\rho_s f_{yh} \epsilon_{su}}{f'_{cc}} \quad \text{Eqn. 3.9}$$

where ϵ_{su} is the ultimate reinforcement strain and $\rho_s = 4A_{sh}/D_{core}s$ is the volumetric ratio of confinement where A_{sh} is the transverse reinforcing steel area and D_{core} is the diameter of the concrete core. A value of 0.12 was assumed for ϵ_{su} following Caltrans (2006). The columns in this thesis have an unconfined concrete characteristic strength of 30 MPa and transverse reinforcement is provided by 16 mm bars spaced at 125 mm from centre to centre with 460 MPa steel. Using the above equations the maximum confined concrete strength and associated strain are estimated as 39.95 MPa and 0.0053 respectively. Figure 3.8 shows the stress strain relationship for the 1.0 m diameter RC

column using *Concrete 01*. From the figure it is apparent that the strength and ductility have both increased by the presence of confinement.

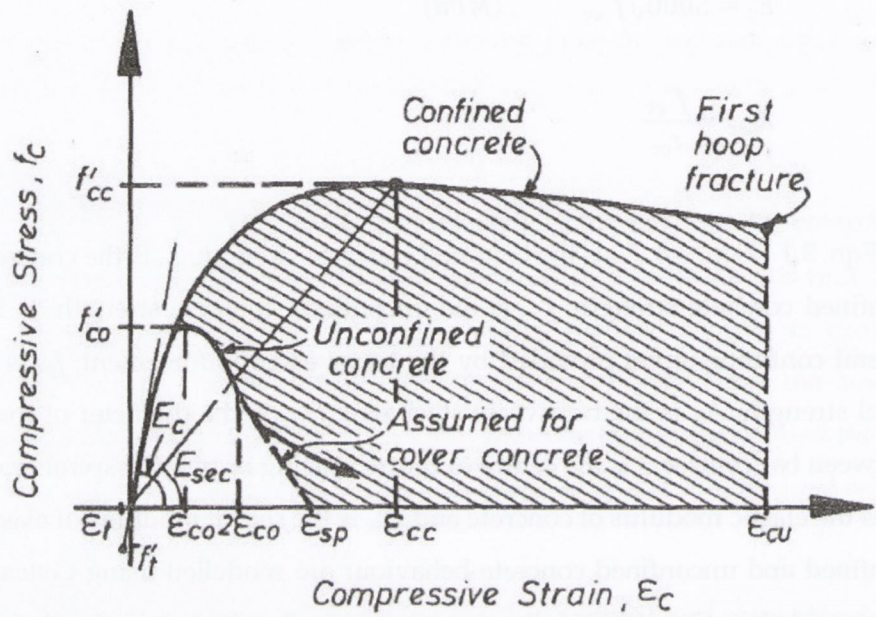


Figure 3.7 Stress-strain model for concrete in compression (Mander et al., 1988)

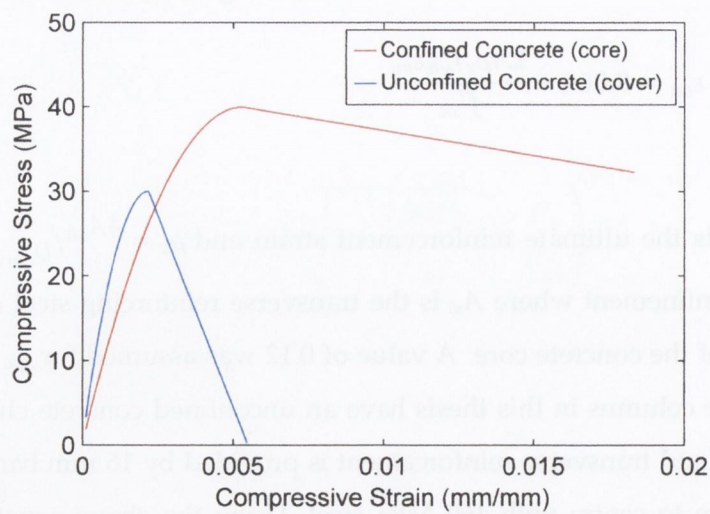


Figure 3.8 Stress-strain relationship for the concrete core and cover using *Concrete 01* material

Reinforcing steel is modelled using *Steel 02* material as provided on OpenSEES following Ramanathan (Ramanathan, 2012). This material uses the Menegotto and Pinto model (Menegotto and Pinto, 1973) which was later modified by Filippou et al. (1983) to include isotropic strain hardening. For the columns, the reinforcing steel is assumed to have a yield characteristic strength of 460 MPa and an elastic modulus of 200 GPa. The stress strain behaviour for the *Steel 02* material, given these properties, is shown in Figure 3.9. A strain hardening ratio of 0.018 was used following Nielson (Nielson, 2005).

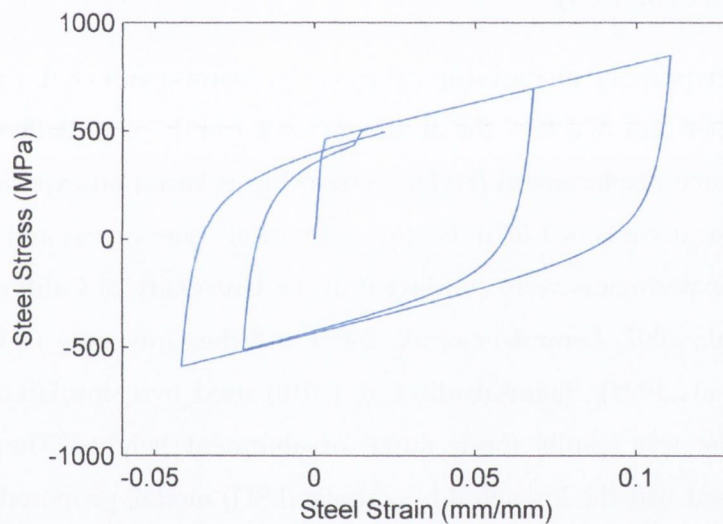


Figure 3.9 Stress-strain relationship for the reinforcing steel using *Steel 02* material

3.2.2 Abutments

The abutments for this integral bridge type are fully monolithic with the deck and are supported on a single row of seven steel H piles. Due to the absence of a system of joints and bearings, the potential for large seismic demands on the abutments is acknowledged. A method for modelling the abutments is adopted from Aviram et al. (2008). The abutment model consists of a rigid element the width of the superstructure, connected through a rigid joint to the superstructure centreline which can be seen in Figure 3.3. The longitudinal and transverse abutment behaviour is modelled using

nonlinear springs along the length of the abutment. Zero length elements provided by OpenSEES are used to model these springs. The response of the abutments in the longitudinal direction is different to the response of the abutments in the transverse direction. Furthermore, the response of the abutments in the passive longitudinal direction is different to the response in the active direction. In the passive direction, when the abutment is pushed towards the backfill, the contribution of the piles and the backfill soil are considered, whereas in the active direction, when the abutment is being pulled away from the backfill, only the pile contribution is considered. In the transverse direction the passive models are adopted with a modified backbone curve for the wing walls based on wing wall effectiveness and participation coefficients (Maroney and Chai, 1994).

A validated hyperbolic relationship proposed by Shamsabadi et al. (2010) between the lateral load per unit width of the abutment wall and the wall deflection is used. The hyperbolic force-displacement (HFD) relationship is based on experimental testing of bridge abutment walls of 1.67 m height and typical cohesionless and cohesive backfill soils. These experiments were conducted at the University of California, Los Angeles (Stewart et al., 2007, Lemnitzer et al., 2009) and the University of California, Davis (Romstad et al., 1995). Shamsabadi et al. (2010) used two simulation procedures to reproduce the test results for a range of abutment heights. The first simulation procedure used was the log-spiral hyperbolic (LSH) model, proposed by (Shamsabadi et al., 2007). The second simulation procedure used two- and three- dimensional finite element analyses using PLAXIS software (Vermeer and Brinkgreve, 1998).

Using the results from these simulations, simple HFD relationships were developed for walls of varying height with cohesive or granular backfills. Figure 3.10 shows the proposed abutment force displacement backbone curve, where F_{ult} is the maximum abutment force developed at a maximum displacement, y_{max} . K_{50} is the average abutment stiffness estimated as $K_{50} = F_{ult}/2y_{50}$ and y_{50} is the displacement at half of the maximum abutment force.

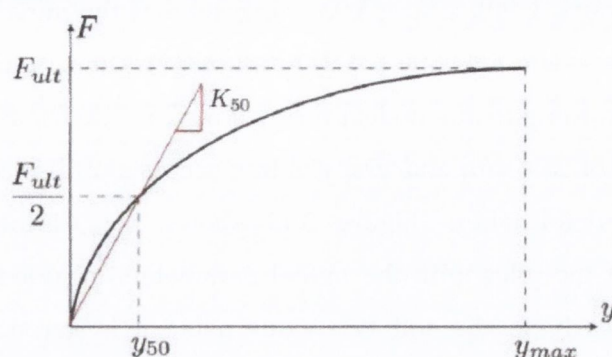


Figure 3.10 Force-Displacement response of the abutment backfill (Shamsabadi et al., 2007)

Following Shamsabadi et al. (2010), the HFD relationship can be estimated as follows:

$$F(y) = \begin{cases} \frac{410.6y}{\left(\frac{H}{1\text{ m}}\right) + 1.867y} \left(\frac{H}{1\text{ m}}\right)^{1.56}, & y \leq 0.05H \text{ (granular soils)} \\ \frac{249.1y}{\left(\frac{H}{1\text{ m}}\right) + 0.8405y} \left(\frac{H}{1\text{ m}}\right)^{1.05}, & y \leq 0.1H \text{ (cohesive soils)} \end{cases} \quad \text{Eqn. 3.10}$$

where $F(y)$ is the lateral force per unit width of the backwall at a lateral displacement of y and H is the abutment height. $F(y)$ is expressed in kN/m, y is expressed in cm and H is expressed in m. The height of the abutments in this thesis is taken as 2.9 m, based on design drawings. Shamsabadi et al. (2010) noted that the maximum displacement of the backwall is $0.05H$ for granular soils and $0.1H$ for cohesive soils. The HFD for the granular and cohesive soils with an abutment height of 2.9 m is shown in Figure 3.11. A hysteretic material provided by OpenSEES is used to estimate the response of the backfill soil based on the HFD curve, which is also shown in Figure 3.11. It is noted that the backfill soil stiffness is considered as a random variable in this thesis and will be further discussed in Chapter 5.

As mentioned previously, the piles are considered to provide abutment resistance in the transverse and longitudinal directions. In order to model the pile behaviour an effective stiffness of 7 kN/mm/pile is assumed with an ultimate strength of 119

kN/pile (Caltrans, 1990). Choi (2002) suggests that the initial pile stiffness degrades with soil surface yielding. Choi (2002) proposed a tri-linear model which is adopted in this thesis. According to the model proposed by Choi (2002), the piles become plastic at a deformation of 25.4 mm and first yielding occurs at a displacement equal to 30% of the ultimate deformation. Figure 3.12 shows the tri-linear force displacement relationship of the piles with the model parameters included. The pile behaviour is modelled similarly to the soil behaviour using a hysteretic material provided by OpenSEES.

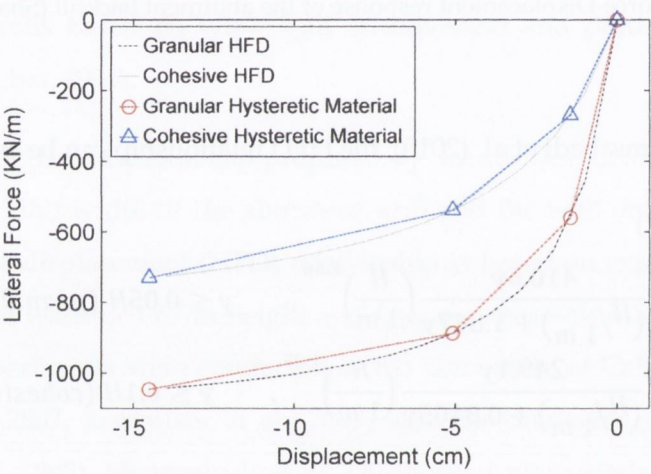


Figure 3.11 HFD relationship for granular and cohesive soils

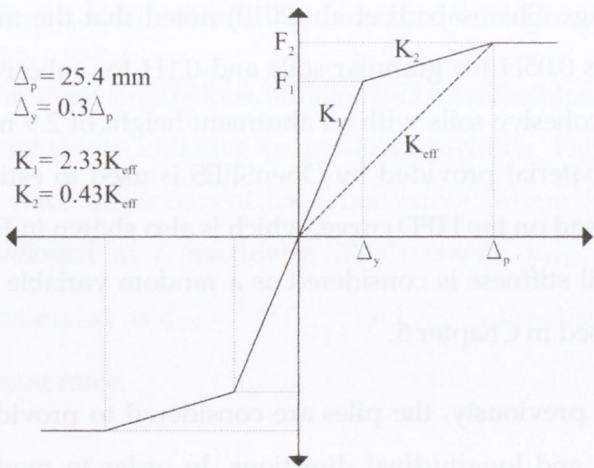


Figure 3.12 Analytical force displacement relationship of piles

The overall abutment behaviour in the longitudinal direction is modelled by placing the soil and pile materials in parallel and applying the material combination to zero length elements along the width of the abutment. Figure 3.13 shows the longitudinal hysteretic behaviour of a 1.65 m wide abutment section which has the equivalent of 1.67 piles. It can be seen from the figure that the behaviour in the passive direction is different to the behaviour in the active direction due to the extra resistance provided by the backfill soil.

In the transverse direction, the soil wing wall behaviour is modelled by altering the longitudinal passive backbone curve to include the wing walls based on wing wall effectiveness and participation coefficients. According to Maroney and Chai (1994), a wing wall effectiveness, C_L , of $2/3$ was taken and a participation coefficient, C_W , of $4/3$ was taken. Nielson (2005) suggested that the effective stiffness contributed by the piles is approximately the same whether the abutment is loaded in the longitudinal or transverse direction, therefore, the same material used to model the pile behaviour in the longitudinal direction is used in the transverse direction. Figure 3.13 shows the transverse hysteretic behaviour of a 1.65 m wide abutment section which has the equivalent of 1.67 piles. From the figure it can be seen that the abutment behaviour is the same in both transverse directions.

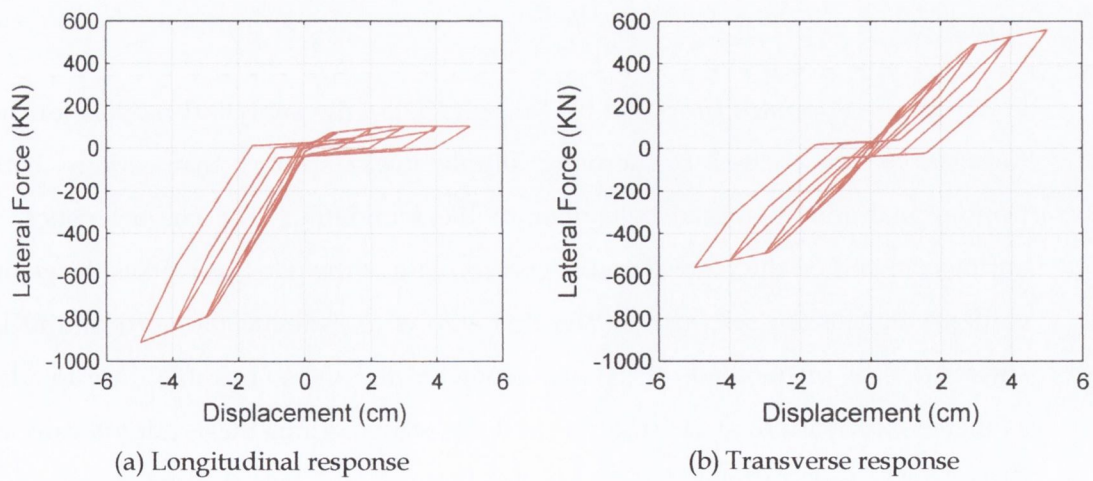


Figure 3.13 Hysteretic behaviour of abutment material in the longitudinal and transverse directions

3.2.3 Foundations

This integral bridge design consists of a pile foundation system with an $11 \times 4.5 \times 1$ m pile cap on two rows of six steel H piles, which can be seen in Figure 3.2. Although traditionally the effect of the foundation on the seismic response of a bridge is neglected, Ma and Deng (2000) have acknowledged the importance of considering the characteristics of the bridge foundation appropriately in an analytical model. Numerical methods used for seismic soil-foundation-structure interaction of highway bridge structures can be classified into the direct approach and substructure approach (Shamsabadi et al., 2006). The direct approach is the most accurate method and involves modelling the entire soil-pile structure system which explicitly treats soil structure interaction. However, this approach is very computationally expensive and Ma and Deng (2000) suggest that it is only advisable for critical structures. The substructure approach provides an alternative and less computationally expensive approach which models the foundation system using a set of simple translational and rotational springs. Owing to its computational efficiency the substructure method is a popular approach to address the soil-foundation-superstructure problem (Tseng and Penzien, 2000). Due to the large number of simulations required for this thesis, a substructure approach has been adopted following previous studies on the seismic risk assessment of highway bridges (Choi, 2002, Nielson, 2005, Padgett, 2007, Ramanathan, 2012).

Following an approach proposed by Nielson (2005), the analytical models for the pile foundations are created by deriving simple linear springs that capture both the translational and rotational behaviour of the foundation. The characteristics of the springs depend on the vertical and lateral stiffness of the piles, as well as the geometric configuration of the pile group. The first step is to estimate the vertical and lateral stiffness of the piles based on the pile geometry and the soil profile. Due to a lack of available information on the soil type and pile stiffness, this thesis adopts values from Choi (2002) and Caltrans (1990) for the vertical and lateral stiffness of the piles respectively. For the vertical stiffness of the piles, which depends on their frictional and tip resistance, a value of 175 kN/mm is adopted from Choi (2002), which is considered as a random variable and will be further discussed in Chapter 5. The lateral

stiffness of the piles is taken as 7 kN/mm/pile (Caltrans, 1990). It is acknowledged that given additional information regarding the soil type and pile stiffness these values may change, however, since the main aim of this thesis is to demonstrate the change in seismic fragility with bridge age, it is thought that a change in these values will not affect the conclusions of the thesis. The foundation pile force deformation behaviour is assumed to be the same as the abutment pile force deformation behaviour, following Nielson (2005). This force deformation relationship can be seen in Figure 3.12. Similar to the vertical stiffness, the lateral stiffness is considered as a random variable and will be further discussed in Chapter 5.

Using the values for the horizontal and vertical pile stiffness, the translational and rotational spring constants for each foundation pile group can be estimated following Ma and Deng (2000):

$$K_{G,h} = \sum_{i=1}^N K_{hh,i} \quad \text{Eqn. 3.11 (a)}$$

$$K_{G,r} = \sum_{i=1}^N K_{vv,i} \cdot x_i^2 \quad \text{Eqn. 3.12 (b)}$$

where $K_{G,h}$ is the group translational spring constant which is the same in the x and y directions and $K_{G,r}$ is the group rotational spring constant, which is different in the x and y directions. $K_{hh,i}$ and $K_{vv,i}$ are the horizontal and vertical stiffness's of the i^{th} pile in the group and x_i is the distance from the centroid of the pile group, measured in the direction perpendicular to the axis of rotation. N is the total number of piles in the group. A plan view of the pile foundation was shown in Figure 3.2. The foundation translational behaviour is modelled using the hysteretic material provided by OpenSEES. The foundation moment rotation relationship is assumed to be linear following the approach proposed by Nielson (Nielson, 2005) and therefore modelled using the elastic material provided by OpenSEES.

3.2.4 Deck Elements

For the first part of this study, which looks at the effect of deterioration on the seismic fragility of the integral bridge type, the deck is modelled using displacement based

beam-column elements with fiber defined cross sections. This is a departure from the usual approach in the literature which adopts elastic beam column elements assuming the superstructure generally remains elastic during seismic events (Aviram et al., 2008). The new approach is adopted in order to investigate whether corrosion leads to the superstructure reaching reduced limit state capacities during seismic events. This approach greatly increases the computational expense of the finite element model and therefore, an aim of this thesis is to investigate whether nonlinear modelling of the superstructure is necessary in cases where deck corrosion exists.

The two geometries considered for this bridge type consist of five U11 or U12 beams with a 200 mm concrete slab poured on top. The U beams and deck are constructed so that they operate in composite action. Similar to the column models, the deck is discretized into fiber sections and material properties are assigned to each fiber. Figure 3.14 shows the fiber discretization of the deck cross section for the centre span of the 52 m bridge. For ease of modelling, the U beams are estimated as being rectangular, as shown in Figure 3.14. This is due to the fact that OpenSEES requires the user to define the fiber section in terms of quadrilateral patches and by estimating the U beams as rectangular, the modelling effort is reduced significantly. The section properties of the approximate section were compared with the actual section properties and there was a negligible difference between the two sets of properties, therefore, this approach was deemed acceptable. The methods adopted for modelling the slab concrete, slab reinforcement and beam concrete are similar to those used for modelling the reinforced concrete columns. For the deck slab, the unconfined concrete material shown in Figure 3.8 is adopted with a maximum strength of 50 MPa taken from design drawings. The deck slab is reinforced at the top and the bottom with 69 No. 20 mm $\varnothing$ steel bars, with a cover to all reinforcement of 40 mm. The reinforcing steel is modelled using the *Steel 02* material on OpenSEES, with a maximum strength of 460 MPa, whose stress-strain relationship can be seen in Figure 3.9. The U beams are fabricated using concrete with a maximum strength of 60 MPa, which is also modelled using the unconfined concrete material with *Concrete 01*.

An approach for modelling the prestressing strands is adopted from Seraj et al. (1992) and is illustrated in Figure 3.15. The prestressing strands for this bridge are 15.7 mm

diameter superstrands with a yield strength, f_{py} , of 1500 MPa and an ultimate strength, f_{pu} , of 1770 MPa. They are stressed initially to 75% of the ultimate strength giving an initial prestress, f_{pi} , of 1328MPa. Total losses are assumed to be 15%, giving an effective prestress, f_{pe} , of 1129 MPa. Following Seraj et al. (1992) the yield strength of the steel model in OpenSEES, $f_{py, model}$, is taken as $(f_{py} - f_{pe})$, which is 371MPa for this thesis. Using this approach, the effects of the prestressing force in the tendon can be represented without a more formal inclusion of initial stresses and strains. *Steel 02* material is used to model the prestressing tendons with a yield stress of $f_{py, model}$.

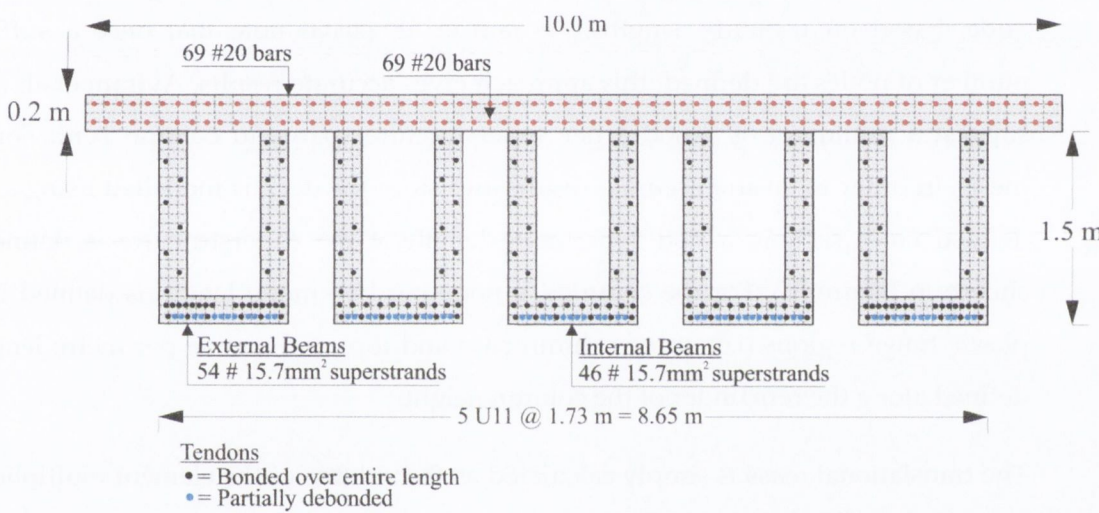


Figure 3.14 Fiber discretization of deck cross section with reinforcing details

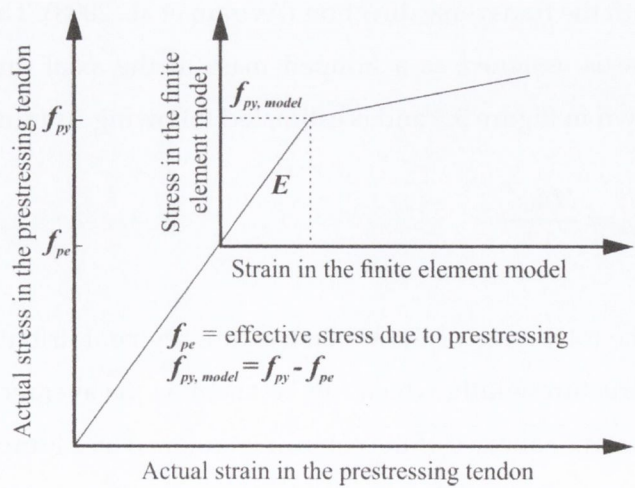


Figure 3.15 Stress-strain model for prestressing tendons

3.2.5 Mass Properties

It is widely acknowledged that the seismic response of a bridge system is sensitive to the mass of the deck (Nielson, 2005). Therefore, it is important that the mass of the bridge components, including the deck, diaphragms, columns and foundations, are quantified accurately. Ideally, all bridge elements should be approximated with a distributed mass along their length (Aviram et al., 2008). However, OpenSEES, as well as other finite element software packages, require the user to define the translational and rotational masses of all the elements and assign them as lumped masses at each node, based on tributary lengths. Aviram et al. (2008) note that once a sufficient number of nodes are defined, this approach gives accurate results. Aviram et al. (2008) suggest a minimum of 5 nodes per superstructure span and column bent. For this thesis, in order to ensure accurate results and since the deck is modelled using a fiber defined cross section, a node per metre length of the superstructure is defined as shown in Figure 3.3. For the columns, a node per half metre length is defined in the plastic hinge regions (i.e. at the column base and top) and a node per metre length is defined along the remainder of the column height.

The translational mass is simply calculated as the volume of the element multiplied by the density of the concrete. The additional assignment of rotational mass is required for the superstructure and column bents for a spine bridge model in order to more accurately represent the dynamic response and fundamental modes of the bridge associated with the transverse direction (Aviram et al., 2008). The rotational mass of the superstructure is assigned as a lumped mass in the axial direction or the global X direction shown in Figure 3.3 and is estimated following Aviram et al. (2008):

$$M_{xx} = \frac{M d_w^2}{12} \quad \text{Eqn. 3.13}$$

where M is the total mass of the superstructure segment, tributary to the node and d_w is the superstructure width, which can be taken as the average of the top and bottom flanges. The rotational mass of the column is assigned as a lumped mass in the global Z direction shown in Figure 3.3 and can be estimated following Aviram et al. (2008):

$$M_{zz} = \frac{1}{2}MR_{col}^2 \quad \text{Eqn. 3.14}$$

where M is the total mass of the column segment, tributary to the node and R_{col} is the radius of the circular column.

3.3 Modal and Deterministic Time History Analysis of the Base Bridge

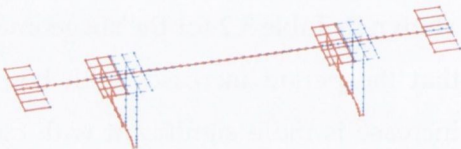
The first step in carrying out a dynamic analysis is to perform a modal or eigenvalue analysis to determine the natural periods and mode shapes of a structure. Eigenvalue analyses of the different bridge geometries are performed using OpenSEES and the first three periods are shown in Table 3.2 for the six geometric configurations. From the table it can be seen that the period increases with both column height and bridge length, although the increase is more significant with column height. The first three mode shapes for the different geometric configurations are similar, therefore, only the mode shapes for the 52 m bridge with 6.9 m columns, hereafter referred to as the base bridge, are shown in Figure 3.16. The first two mode shapes are in the longitudinal and transverse directions respectively, while the 3rd mode invokes a torsional response of the bridge.

A time history analysis is then performed on the base bridge by subjecting it to the X and Y components of a ground motion record in the longitudinal and transverse directions respectively. The records were chosen from a suite of near fault ground motions identified by Krawinkler et al. (2003) from the SAC project database (2013). The ground motions applied in the longitudinal and transverse directions have peak ground accelerations (PGA) of 0.99g and 1.1g respectively. These records were chosen due to their large PGAs in order to demonstrate the nonlinear behaviour of the various bridge components in the deterministic analysis. For the fragility analysis, the ground motion components will be applied to the bridge simultaneously, however, for clarity, they are also applied to the bridge separately for the deterministic analysis. It must be noted that for this thesis, ground motion records are not applied to the finite element models in the vertical direction, following previous studies on the seismic fragility of highway bridges (Nielson, 2005, Padgett, 2007, Ghosh and Padgett, 2010, Ramanathan

et al., 2010). However, it is important to note that the inclusion of vertical ground motions in the fragility analysis may lead to higher deck demands and therefore provides an opportunity for future work.

Table 3.2 First three periods of the integral bridge configurations considered

Column height	6.9 (m)		9.5 (m)		12.0 (m)	
Bridge length	52 (m)	69 (m)	52 (m)	69 (m)	52 (m)	69 (m)
1st mode	0.39	0.42	0.46	0.50	0.50	0.55
2nd mode	0.31	0.35	0.34	0.38	0.35	0.40
3rd mode	0.18	0.24	0.18	0.24	0.18	0.24



(a) Longitudinal response



(b) Transverse response



(c) Torsional response

Figure 3.16 First four mode shapes for base bridge

The acceleration time histories and response spectra with 4.5% damping for both ground motions are shown in Figure 3.17. 4.5% Rayleigh damping is used in the time history analyses since this value of damping is typical for concrete bridges (Aviram et

al., 2008). These response spectra are plots of the maximum spectral acceleration response of a single degree of freedom structure with varying natural period, subjected to the ground motions shown in Figure 3.17 (a) & (b). From Figure 3.17 (c) & (d), it can be seen that there is a 'dip' in the spectral acceleration response at 0.5 s. This is of interest in Section 3.4.1, when the responses of the various geometric bridge configurations are compared. Since the 52 m bridge with 12.0 m columns has a natural period of 0.50 s, it has a lower response to the chosen ground motion records than expected.

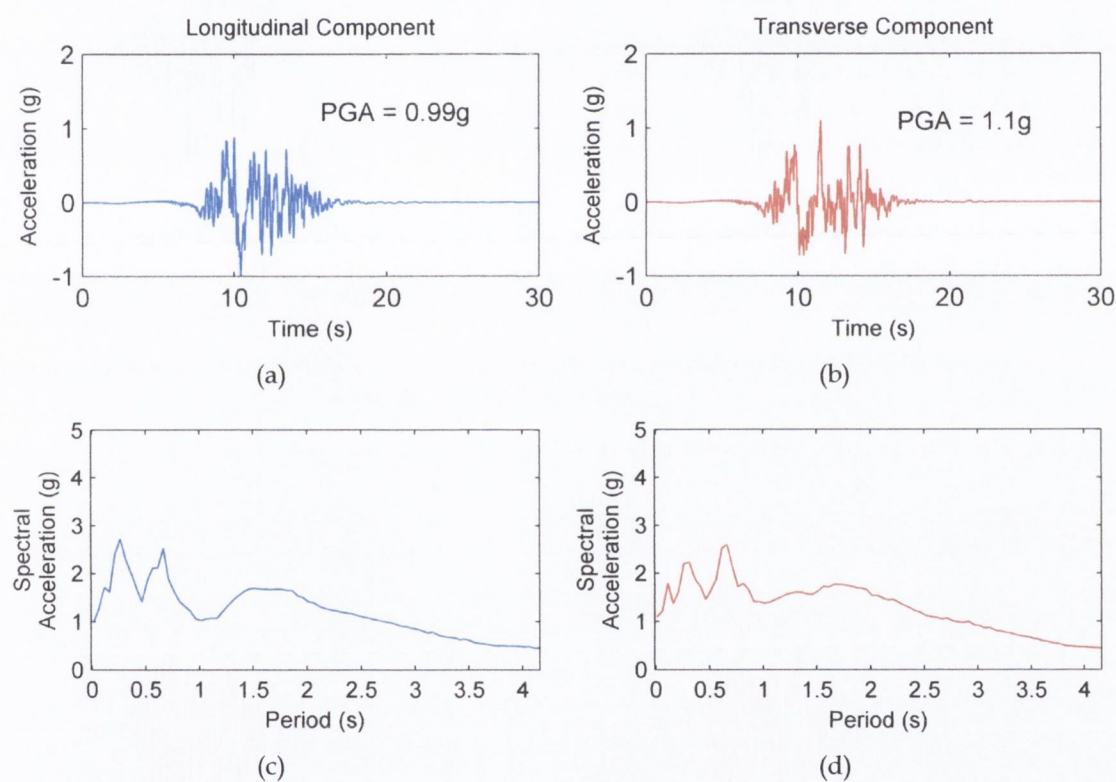


Figure 3.17 Longitudinal and transverse ground motions used to illustrate seismic response of bridge components (a) & (b) Longitudinal and transverse time histories (c) & (d) Longitudinal and transverse response spectra with 4.5% damping

Figure 3.18 shows the displacement response in the longitudinal and transverse directions of the right bent and abutment. It can be seen that the bent and abutment responses in the longitudinal direction are identical, which is expected since the superstructure is continuous and monolithic with the abutments. In the transverse

direction the maximum bent displacement is just less than 8% larger than the abutment displacement. This is expected due to the fact that the abutments provide more resistance to transverse movement at the level of the deck through the piles and wing walls.

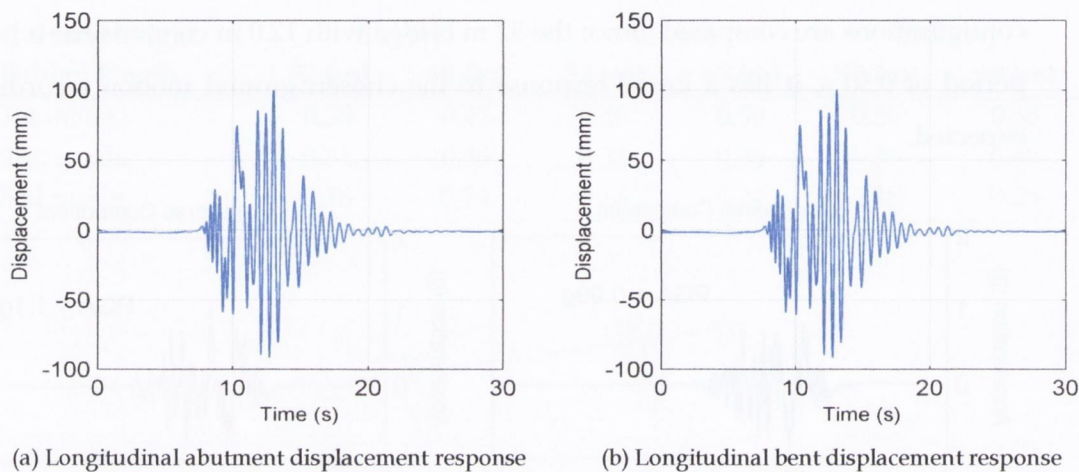


Figure 3.18 (a) & (b) Displacement response for the right abutment and bent in the transverse and longitudinal directions

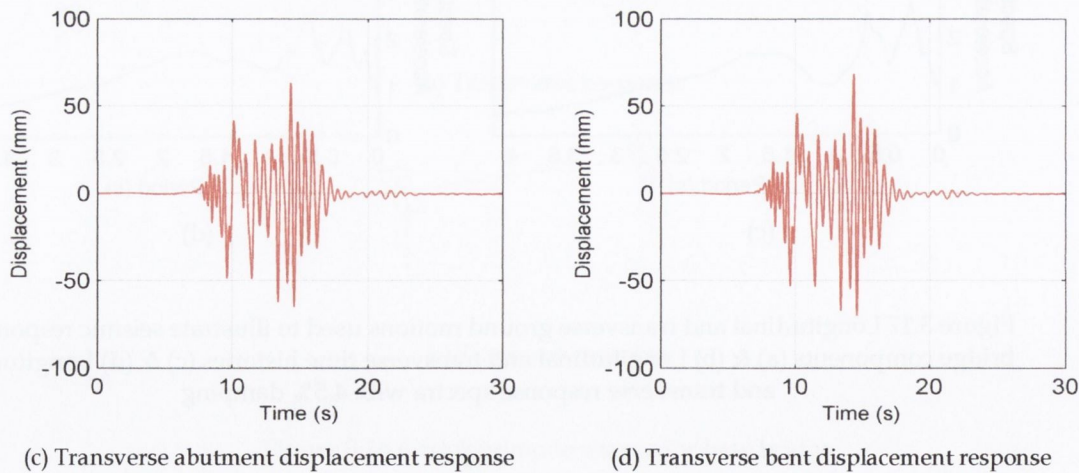


Figure 3.18 (c) & (d) Displacement response for the right abutment and bent in the transverse and longitudinal directions

Figure 3.19 shows the foundation displacement which is significantly smaller than the abutment and bent response since the superstructure mass causes greater movement at the level of the deck. Figure 3.20 shows the foundation lateral force deformation relationship in the longitudinal direction as well as the moment rotation relationship in the longitudinal direction. The lateral force deformation relationship shown in Figure 3.20 (a) appears reasonable based on the expected trilinear foundation force deformation relationship described in Section 3.2.3. In Figure 3.20 (b) the linear moment rotation relationship can be seen, which was also described in Section 3.2.3.

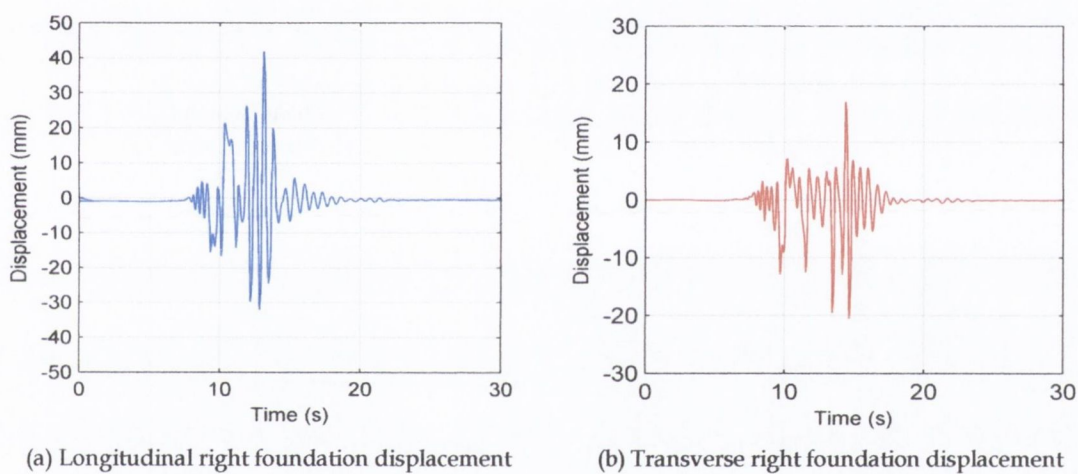


Figure 3.19 Displacement response for the right foundation in the longitudinal and transverse directions

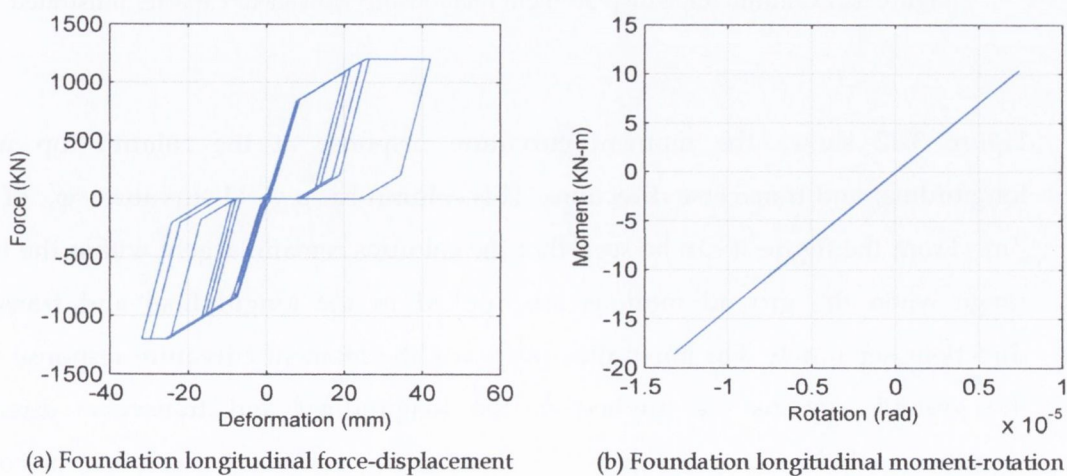


Figure 3.20 Foundation response in the longitudinal direction

The RC section analysis program, Cumbia Octave (Montjo, 2007), was used to estimate the base shear force displacement relationship of the column section as well as the shear capacity envelope. This analysis was performed in order to establish whether the columns under consideration would experience a shear, shear-flexure or flexure failure. This program estimated the shear strength envelope for the member using the analytical shear strength model presented by Kowalsky and Priestley (2000). The shear force demand and shear strength envelope are shown in Figure 3.21. From the figure, it can be seen that the columns under consideration are not shear critical and therefore a flexural failure will be assumed.

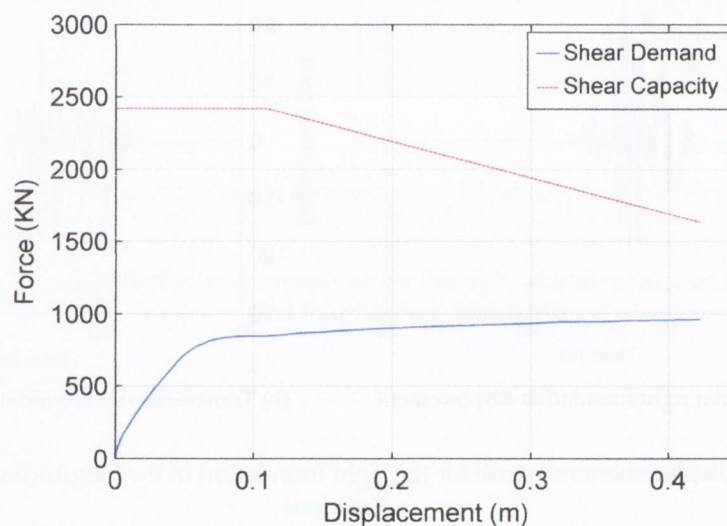


Figure 3.21 Column force-displacement relationship with shear capacity illustrated

Figure 3.22 shows the moment curvature response at the column top in the longitudinal and transverse directions. This column has a yield curvature, ϕ_y , of 0.007 /m. From the figure it can be seen that the columns remain largely within the elastic range when the ground motions are applied in the longitudinal and transverse directions separately. For illustrative purposes, the moment curvature response when the ground motions are applied in the longitudinal and transverse directions simultaneously is shown in Figure 3.23. From the figure it can be seen that the column

response begins to exhibit nonlinear behaviour when the ground motions are applied simultaneously.

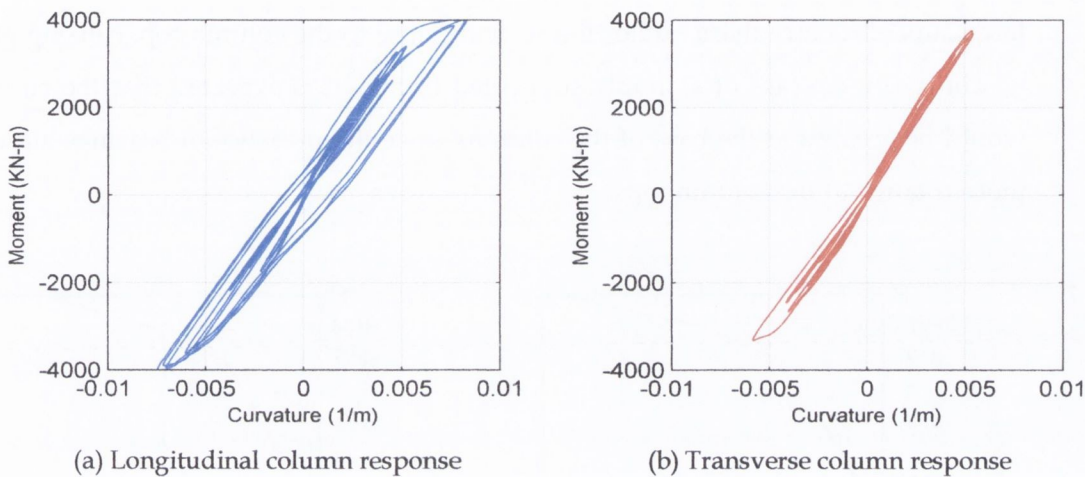


Figure 3.22 Longitudinal and transverse column moment curvature response

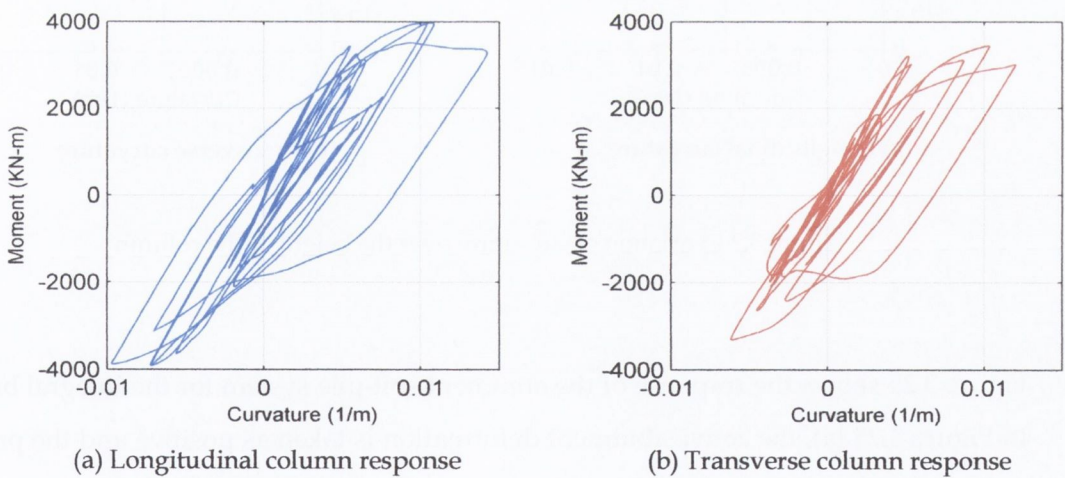


Figure 3.23 Longitudinal and transverse column moment curvature response to ground motions applied in the longitudinal and transverse directions simultaneously

Figure 3.24 shows a plot of the maximum curvature response over the height of the column when the orthogonal ground motions are applied simultaneously. From the figure it can be seen that the curvature is greatest at the column top and bottom due to

the fact that the column is close to fixed at both the top and base. Therefore, it is likely that the plastic hinges will form at the column base or top for this integral bridge type. It is also noted that the curvature is greater at the column top due to the fact that the large superstructure mass excites the sections close to the column top, causing greater yielding. For the case of a simply supported bridge, it is expected that the curvature would be greatest at the base of the columns since the presence of bearings allows for more rotation at the column tops.

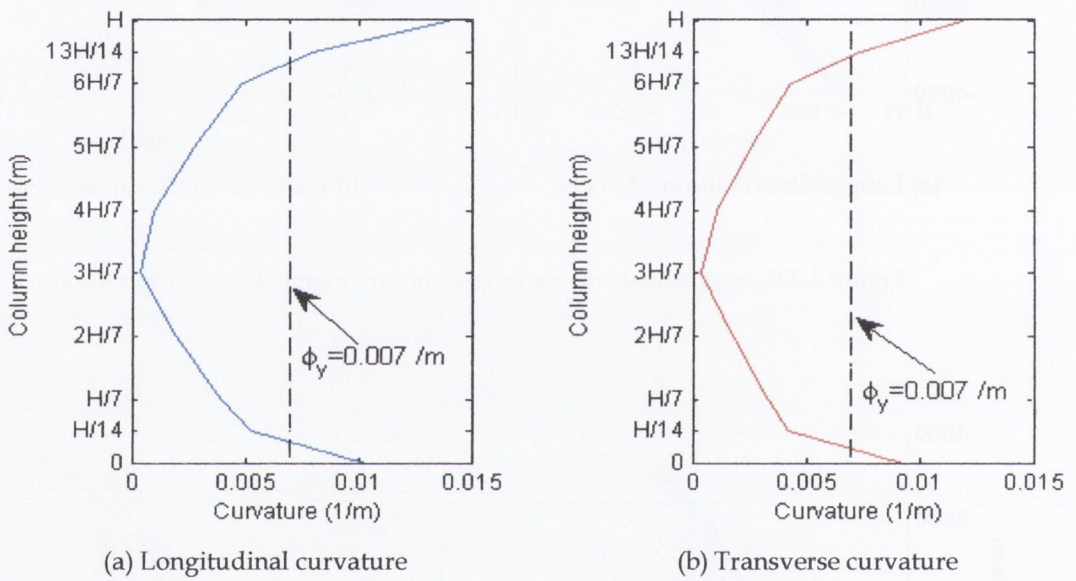


Figure 3.24 Variation of curvature over the height of the column

Figure 3.25 shows the response of the abutment soil-pile system for the integral bridge. In Figure 3.25 (a), the active abutment deformation is taken as positive and the passive deformation is taken as negative. As mentioned previously, the longitudinal response of the abutments is characterized by the contribution of the backfill soil and the piles in the passive direction but only the piles in the active direction. This can be seen in Figure 3.25 (a), where lower forces are developed in the active direction. In the traverse direction, both the soil and the piles contribute, resulting in similar behaviour in both transverse directions which can be seen in Figure 3.25 (b). Figure 3.25 also illustrates the effect of the backfill soil type on the abutment response by including the abutment

response with a sandy or clayey backfill. The sandy backfill is stiffer than the clayey backfill and therefore larger forces develop for this abutment-backfill configuration, which can be seen in Figure 3.25 (a) & (b). It is noted that the backfill type has a more significant impact in the longitudinal passive direction than the transverse direction. This is due to the fact that the wing walls are smaller than the abutment backwall and therefore the type of soil has a more significant effect longitudinally than transversely. The maximum displacement of the abutment is smaller for the sandy backfill compared to the clayey backfill, with maximum displacements of 58 mm and 106 mm occurring for the sandy and clayey backfills respectively. Figure 3.26 shows the moment curvature response for the sandy and clayey backfills, which shows that the sandy backfill results in smaller curvature demands due to its higher stiffness. It is noted that soil type is an uncertain parameter when modelling force displacement relationship of the abutment backfill due to a lack of site specific data from ground investigations. These plots demonstrate that the response of the abutment-backfill system is sensitive to the soil type and therefore the soil stiffness is considered a random parameter in the probabilistic analysis performed in Chapter 6.

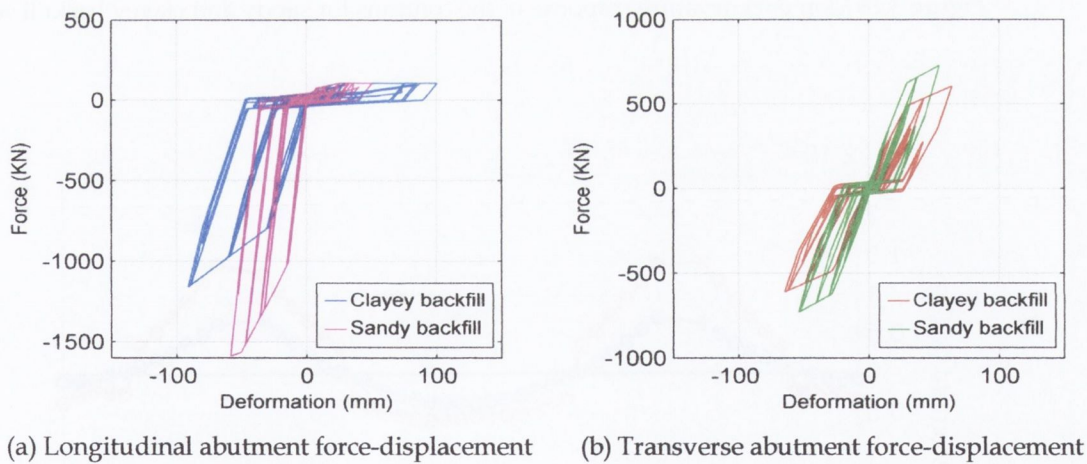


Figure 3.25 Abutment force-deformation response with granular and clayey backfill soil

Figure 3.27 shows the envelope of the maximum deck bending moments when the 0.99g and 1.1g components of the ground motion are applied simultaneously in the longitudinal and transverse directions respectively. Also shown on the figure is the

deck bending moment diagram under the bridge self weight. From the figure is can be seen that the maximum hogging moment under seismic loading is almost twice the hogging moment under the bridge self weight. The maximum sagging moment under seismic loading is around 1.75 times the sagging moment under the bridge self weight. From the figure it can be seen that the critical locations for this bridge type are at the intermediate supports.

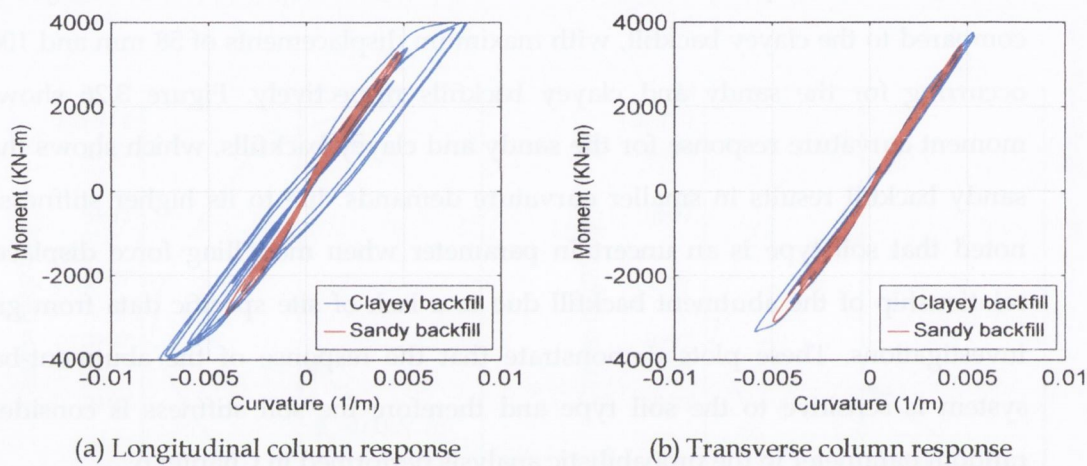


Figure 3.26 Moment curvature response of the columns for sandy and clayey backfill soil

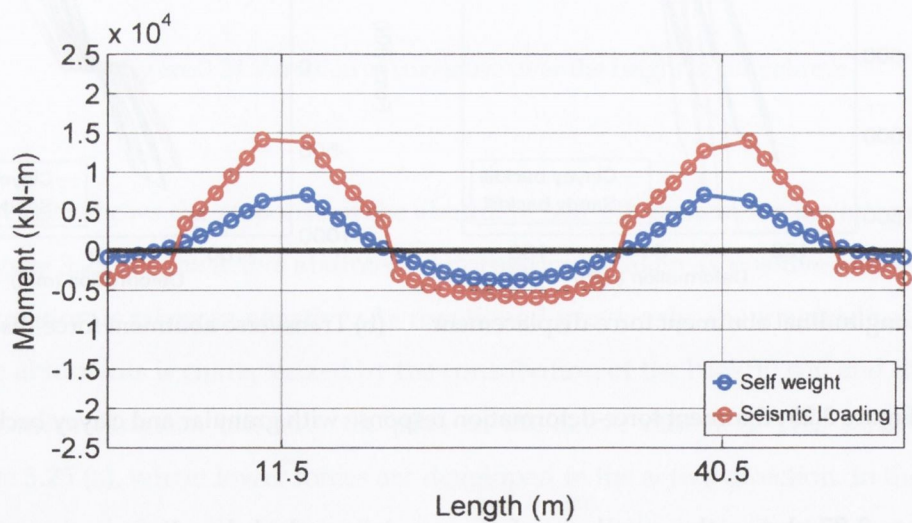


Figure 3.27 Envelope of maximum deck bending moment over bridge length

3.4 Effect of Geometric Variation on the Deterministic Time History Response

As mentioned previously, this study investigates the effect of bridge length and column height variation on the seismic response of integral bridges. As a preliminary investigation, the effect of geometric variation on the deterministic bridge response, when the 0.99g and 1.1g components of the ground motion are applied in the longitudinal and transverse directions separately, is shown. As mentioned previously, for the fragility analysis the two components of the ground motion will be applied to the bridge simultaneously, however, for clarity, they are applied to the bridge separately for the current deterministic analysis.

3.4.1 Column Height

As seen in Section 3.1, the three column heights considered in this study are 6.9 m, 9.5 m and 12.0 m. Figure 3.28 (a) shows the longitudinal displacement response of the 52 m bridge's right column bent, to the 0.99g earthquake applied in the longitudinal direction, given three column heights. From the figure it can be seen that the maximum displacement increases with column height. Figure 3.28 (b) shows the longitudinal moment curvature response of a column for the same earthquake. From the figure it can be seen that the 12 m column experiences the smallest curvature and the 9.5 m column experiences the largest curvature.

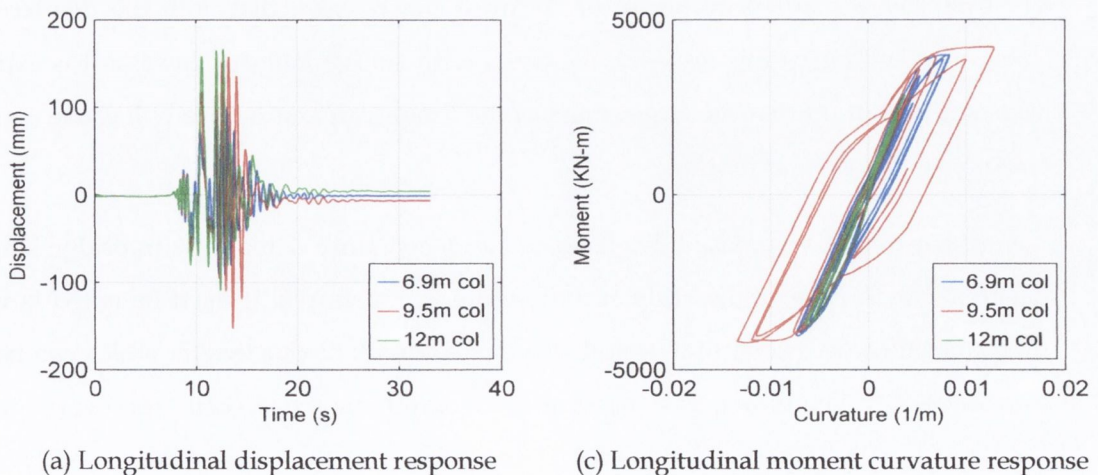


Figure 3.28 Effect of column height on the longitudinal column response to the 0.99g ground motion applied in the longitudinal direction

In order to understand this result it must be noted that for a given level of displacement, the curvature experienced by a column decreases with column height. From Figure 3.28 (a), it can be seen that although the maximum displacement increases with column height, the 9.5 and 12.0 m columns have very similar maximum responses. A possible reason for this may be found by referring to Table 3.2 and Figure 3.17 (c). From the table it can be seen that the fundamental period of the 52 m bridge with 12.0 m columns is 0.5 s. From Figure 3.17 (c), which shows the response spectrum of the longitudinal ground motion, it can be seen that there is a 'dip' in the spectral acceleration response at 0.5 s, which may explain the lower response for that column height. A similar trend is seen in the transverse direction and therefore the graphs for the transverse displacement and moment curvature response are not shown. It is important to note from the results of this deterministic analysis that the response of a structure can be very sensitive to the ground motion used, therefore, in order to gain an understanding of the overall seismic vulnerability of a bridge type, a probabilistic approach is necessary, where many ground motions are used.

3.4.2 Bridge Length

As seen in Section 3.1, the two bridge lengths under investigation are 52.0 m and 69.0 m. Figure 3.29 shows the transverse displacement response of the right column bent to the 1.1g earthquake applied in the transverse direction for the two bridge lengths with column heights of 6.9 m. From the figure it can be seen that both the displacement response and curvature response increases with bridge length. This result is expected since it is thought that the larger mass of the longer superstructure will cause a greater response to a ground motion.

A similar trend of increased displacement and curvature demand with bridge length is seen in the transverse direction and therefore is not shown. It must be noted however that the stiffness and natural period also increase with bridge length, which can be seen in Table 3.2. Therefore, this increase in demand may not occur for every ground motion, depending on the dynamic characteristics of the earthquake. A probabilistic seismic analysis is necessary in order to draw conclusions on the change in seismic performance with a change in bridge geometry. The probabilistic approach developed

in this thesis to estimate the seismic vulnerability of the integral bridge type is discussed in Chapter 5, with the results presented in Chapter 6.

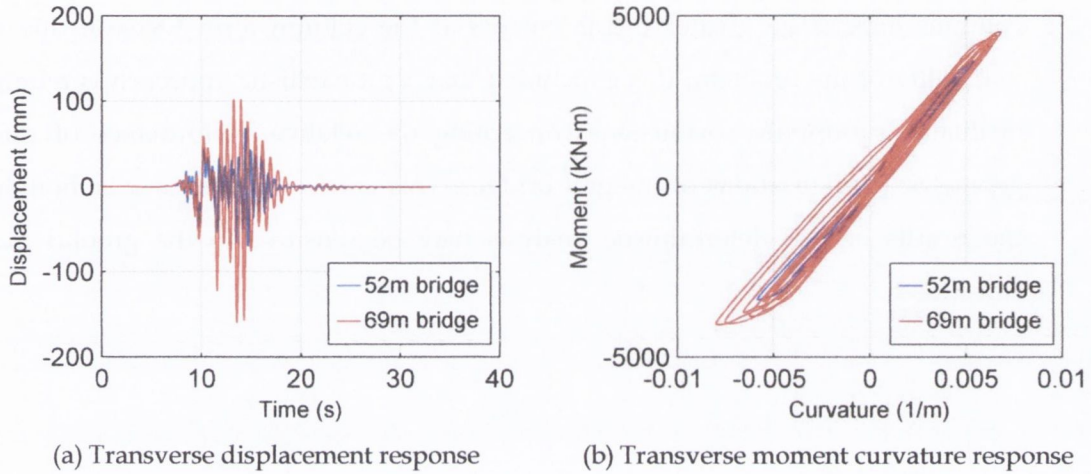


Figure 3.29 Effect of bridge length on the transverse column response to the 1.1 g ground motion applied in the transverse direction

3.5 Conclusion

This chapter presents extensive details about the modelling strategies adopted for each of the bridge components: superstructure, columns, abutments and foundations. The models are developed based on experimental data for the components and knowledge of their past performance during actual earthquakes. Three-dimensional finite element models of the six geometric configurations considered are generated using OpenSEES. A deterministic analysis is performed on each of the geometric configurations using a pair of orthogonal time histories from the SAC database (SAC, 2013) with PGAs of 0.99g and 1.01g. The purpose of this chapter is not to facilitate drawing conclusions on the seismic performance of this bridge type, but rather to gain insight into the relative seismic response of the different bridge components of the integral bridges under consideration, as well as providing a sanity check. From the results of this chapter, it was seen that plastic hinges tend to form at the column tops and bases when the bridge is subjected to the ground motions considered. Abutments with sandy backfills exhibit larger forces and lower displacements when compared to abutments with clayey backfills. This result is expected since the sandy backfill is stiffer and therefore

provides more resistance to movement. The columns in bridges with clayey backfills experience greater levels of curvature. The column curvature increases with bridge length due to the larger mass of the superstructure. Finally, it is found that longer columns experience greater displacements at the column tops. Overall, due to the variability of the problem, it is concluded that a probabilistic approach is required to facilitate appropriate conclusions concerning the relative performance of different geometric configurations of integral bridges. This is necessary since it is thought that the results of this deterministic analysis may be sensitive to the ground motions considered.

CHAPTER 4 - DETERIORATION MODELLING AND MODIFIED DETERMINISTIC BRIDGE COMPONENT RESPONSES

Depending on local conditions at the bridge site, certain bridge components may experience significant levels of strength degradation over time. Some of the main deterioration mechanisms that effect bridges are steel corrosion, concrete cracking and spalling, chemical attack, fatigue, scour and foundation settlement (FHWA, 2006). The next step in this thesis is to update the OpenSEES finite element bridge models to include various levels of deterioration, which is modelled as a function of time. The attempt to quantify the associated loss of strength with time is not a trivial task, due to vast uncertainties in structural and environmental parameters. This chapter describes the corrosion models adopted for this study and the approach used to quantify foundation settlement.

As described in Chapter 2, carbonation and chloride induced corrosion are the two main types of steel corrosion for RC structures. Chloride induced corrosion has been chosen for this thesis since there is general agreement (Saassouh and Lounis, 2012, Saraswathy and Song, 2007, Angst et al., 2009) that it is the most widespread and serious cause of deterioration of RC bridges worldwide. Tilly (2007) claims that it is responsible for more than 55% of concrete repairs carried out. For this thesis, it is assumed that chloride induced corrosion leads to a loss of steel cross section and strength in the RC columns and the deck. The loss of cross section and strength is modelled over time using a probabilistic approach described in this chapter. The finite element models are then updated and the pristine and corroded deterministic seismic responses are compared. This approach is adopted in order to gain insight into the effect of corrosion on the seismic response of this integral bridge type.

In order to quantify foundation movement, discrete levels of settlement are chosen based on a survey carried out on the foundation movement of 314 bridges across the US and Canada (FHWA, 1985), which is also further discussed in this chapter. The finite element models are modified to include the discrete levels of foundation settlement considered and the deterministic seismic responses are compared to the case of no foundation settlement. For the deterministic analysis performed in this chapter, the effect of settlement on a corroded bridge is investigated; however, for the full fragility analysis carried out in Chapter 6, the effect of settlement alone as well as the combined case of corrosion and settlement will be investigated.

4.1 Probabilistic modelling of Chloride Induced Corrosion

A vast amount of research has been undertaken to develop predictive models for chloride induced corrosion. However, there are significant levels of variation between these models depending on the environmental exposure conditions assumed and model parameters adopted. The exposure conditions modelled in this section include exposure to chlorides from de-icing salts and marine exposure in a coastal zone. Chapter 2 described the two phases of the corrosion process according to the model developed by Tutti (1979), which is illustrated in Figure 2.6. These phases are considered to be the same regardless of the source of the chlorides. The two phases are known as the initiation phase and the propagation phase. During the initiation phase, chlorides from the atmosphere or from within the concrete penetrate the concrete cover until they reach and destroy the passive film protecting the steel. The propagation phase involves the actual corrosion of the steel which results in a reduction in the cross sectional area and strength of the steel reinforcing bars. Corrosion will then occur if moisture and oxygen are present at the level of the reinforcement. The following sections discuss the probabilistic modelling of the initiation and propagation phases for the case of de-icing salts and marine exposure.

4.1.1 Corrosion Initiation Phase

There have been numerous studies which have found that the penetration of chlorides is best represented by a diffusion process once the concrete is relatively moist (Vu and

Stewart, 2000). Following previous studies which have attempted to model the corrosion process (Vu and Stewart, 2000, Enright and Frangopol, 1998, Thoft-Christensen et al., 1996), the ingress of chloride ions from the concrete surface to the level of the reinforcing steel is modelled using Fick's 2nd law of diffusion represented as:

$$\frac{\delta C(x, t)}{\delta t} = -D_c \frac{\delta^2 C(x, t)}{\delta x^2} \quad \text{Eqn. 4.1}$$

where $C(x, t)$ is the chloride ion concentration at a distance, x , from the concrete surface, after t years of exposure to the chloride source and D_c is the chloride diffusion coefficient. Given that the concentration of chlorides near the surface and the diffusion coefficient are relatively constant, as typically assumed for corrosion due to de-icing salts or marine exposure (Val and Stewart, 2003, Vu and Stewart, 2000, Hoffman and Weyers, 1996), the solution to the differential Eqn. 4.1 is written as:

$$C(x, t) = C_s \left[1 - \operatorname{erf} \left(\frac{x}{2\sqrt{D_c t}} \right) \right] \quad \text{Eqn. 4.2}$$

where erf is the Gaussian error function. For a RC column or deck slab, $C(x, t)$ is assumed to be equal to C_{cr} , the critical chloride concentration that causes dissolution of the protective passive film around the reinforcement and x is the thickness of the concrete cover. Therefore, the corrosion initiation time, T_i , can be estimated as follows:

$$T_i = \frac{x^2}{4D_c} \left[\operatorname{erf}^{-1} \left(\frac{C_s - C_{cr}}{C_s} \right) \right]^{-2} \quad \text{Eqn. 4.3}$$

From Eqn. 4.1 -Eqn. 4.3, it can be seen that the corrosion initiation time depends on the concrete cover, x , which can be read from the bridge design drawings, as well as three random parameters (D_c , C_s , C_{cr}). These parameters can vary considerably depending on the bridge location and exposure conditions. In order to estimate the corrosion initiation time for the bridge under consideration accurately, appropriate distributions for the random variables must be chosen. The following sections discuss each of the variables and their chosen distributions for this study.

4.1.1.1 Surface Chloride Concentration, C_s

The surface chloride concentration varies dramatically depending on the bridge location and the source of the chlorides. As mentioned previously, this section presents models for corrosion initiation due to the application of de-icing salts as well as marine exposure in a coastal zone. Vu and Stewart (2000) carried out a literature review on available data for surface chloride concentrations due to the application of de-icing salts. This study found that the data from Hoffman and Weyers (1993), comprising of samples taken from 321 concrete bridge decks in the US, was the most comprehensive of the literature reviewed. From this data, the mean value for C_s is taken as 3.5 kg/m^3 . Although Hoffman and Weyers (1993) proposed a coefficient of variation (COV) of 0.5, it is thought that this is too large since using this COV to estimate the distribution of the corrosion initiation time leads to unstable results even for very large sample sizes. Therefore, a COV of 0.2 is adopted following Enright and Frangopol (1998).

This study also models the corrosion process for regions $< 0.1 \text{ km}$ from the coast, referred to as an atmospheric zone in the literature (Val and Stewart, 2003). In atmospheric zones, salt laden water carried by the wind from the sea inland, accumulates on exposed concrete surfaces. Similar to the case of de-icing salts, the concentration of chlorides on the surface of a concrete member in a marine environment is highly variable and dependent on environmental conditions, topography, orientation of the concrete surface and distance from the coastline (Berke and Hicks, 1992). Val and Stewart (2003) suggest that data reported by McGee (2000) is one of the most comprehensive available for bridges in an atmospheric zone. This study suggests that the mean value for the surface chloride concentration for zones $< 0.1 \text{ km}$ from the coast is 2.95 kg/m^3 . As for the case of de-icing salts, a COV of 0.2 is adopted. For both the atmospheric zone and the case of de-icing salt exposure, it has been found that the surface chloride content is well described by a lognormal distribution (Val and Stewart, 2003, Vu and Stewart, 2000).

4.1.1.2 Critical Chloride Concentration, C_{cr}

The critical chloride concentration, C_{cr} , is the concentration at the level of the steel which causes dissolution of the protective passive film around the reinforcement, thus initiating corrosion. Stewart and Rosowsky (1998) suggest that numerous studies

(Mechta, 1991, Cady and Weyers, 1983) have found that the critical chloride concentration tends to lie within the range of 0.6 - 1.2 kg/m³. The critical chloride content is influenced by the water/cement ratio, cement type, water and oxygen content, pH, fly ash content and silica fume content (Stewart and Rosowsky, 1998). It is therefore expected to be highly variable. For this study, a uniform distribution within the range 0.6 - 1.2 kg/m³ is assumed for both marine and de-icing salt exposure, following Stewart and Rosowsky (1998).

4.1.1.3 Diffusion Coefficient, D_c

The chloride diffusion coefficient, D_c , represents the concrete permeability and can be influenced by both concrete and environmental parameters. A mean value of 1.29 cm²/year is chosen from Enright and Frangopol (1998), which was identified from field data on existing bridges in the US exposed to de-icing salts (Whiting et al., 1990). Vu and Stewart (2000), suggest that the chloride diffusion coefficient is not significantly affected by the source of the chlorides and therefore the same value is adopted for the case of marine exposure. A COV of 0.1 was chosen following Enright and Frangopol (1998).

4.1.1.4 Concrete Cover, x

The concrete cover for the bridge under investigation is taken as 50 mm for the columns and 40 mm for the deck following bridge design drawings. The corrosion initiation time increases with cover depth and it is therefore recommended to provide adequate cover depths for increased durability. Figure 4.1 shows the influence of the concrete cover depth on the corrosion initiation time. From the figure it can be seen that the initiation time increases from less than 3 years to almost 11 years with a 30 mm increase in cover, when the all the other parameters are taken as the mean values described in Sections 4.1.1.1 - 4.1.1.3.

The probabilistic models adopted for the corrosion variables for each exposure class can also be seen in Table 4.1. The distribution of the corrosion initiation time derived from Eqn. 4.3 and random variables in e

Table 4.1 can be found through Monte Carlo simulation with a sample size of 10,000. Figure 4.2 shows the simulated data for the corrosion initiation time for the case of

deicing salt exposure and marine exposure. For both cases, a lognormal distribution was found to be a good fit to the simulated data and is shown in the figures. For the case of deicing salt exposure, a lognormal distribution with a logarithmic mean of 8.9 years and standard deviation of 5.3 years was found to be a good fit. For the case of marine exposure, a lognormal distribution with a mean of 11.3 years and standard deviation of 7.2 years was found to be a good fit. These values will be used as input variables for the next step in corrosion modeling, which predicts the loss of steel cross sectional area once corrosion has initiated. As expected, marine exposure leads to a longer corrosion initiation time due to lower surface chloride concentrations.

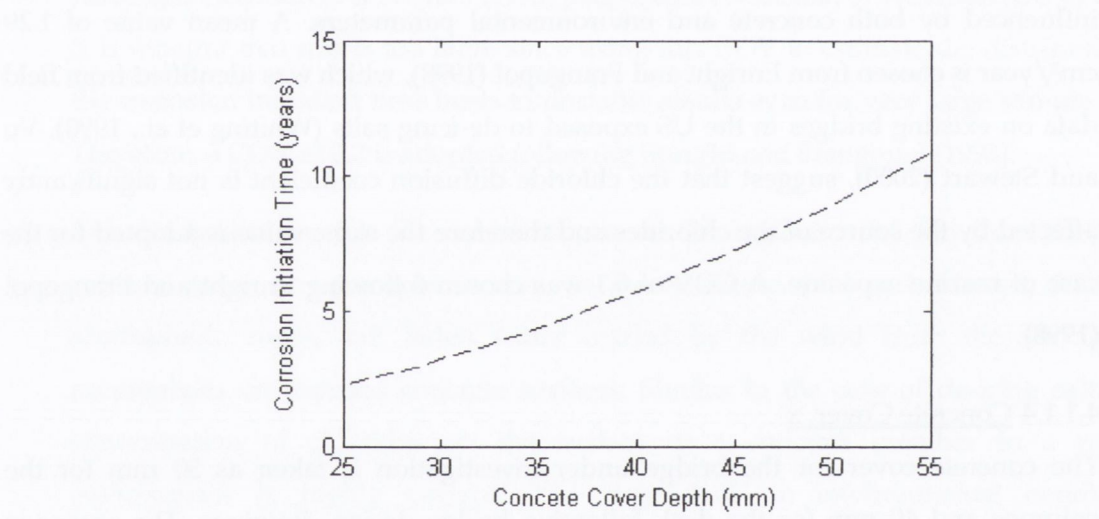


Figure 4.1 Influence of concrete cover depth on corrosion initiation time

Table 4.1Statistical parameters for corrosion variables

Parameter	Mean	COV ^a	Distribution
Cover depth	50 mm	0.20	Lognormal
Diffusion coefficient (D_c) ^b	1.29 cm ² /yr	0.10	Lognormal
Surface chloride concentration (De-icing) (C_0) ^c	3.5 kg/m ³	0.20	Lognormal
Surface chloride concentration (Marine) (C_0) ^c	2.95 kg/m ³	0.20	Lognormal
Critical chloride concentration (C_c) ^d	0.9 kg/m ³	-	Uniform (0.6-1.2)

^aCOV = Coefficient of Variation

^bAdopted from Enright and Frangopol (Enright and Frangopol, 1998)

^cAdopted from Vu and Stewart (Vu and Stewart, 2000)

^dAdopted from Hoffman and Weyers (Hoffman and Weyers, 1996)

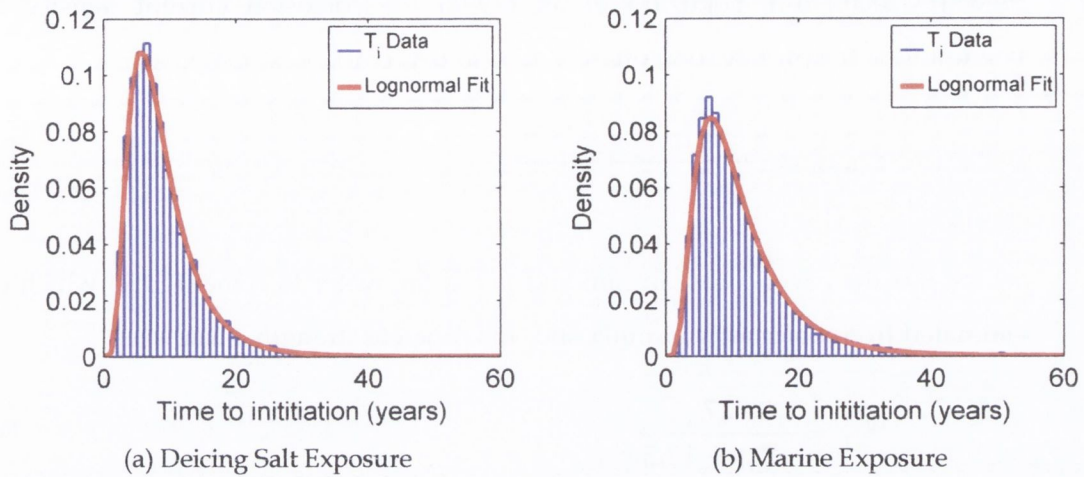


Figure 4.2 Distributions for the corrosion initiation time

4.1.2 Corrosion Propagation Phase

Once corrosion has initiated due to de-icing salts or marine exposure, the loss of steel diameter over time can be estimated using the diameter of the pristine reinforcing bar and the rate of steel cross section loss due to corrosion. Although it was noted in Chapter 2 that chloride induced corrosion is characterised by pitting corrosion, uniform corrosion is assumed in this thesis as a simplification following previous studies on the ageing fragility of bridges (Choe et al., 2008, Choe et al., 2009, Simon et al., 2010, Ghosh and Padgett, 2010, Alipour et al., 2011). An opportunity for future research would be to consider the effects of pitting corrosion in the model.

In many studies, the corrosion rate is assumed to be constant over the service life of the structure due to the lack of available data to develop an accurate time dependent corrosion rate (Thoft-Christensen et al., 1996, Enright and Frangopol, 1998, Stewart and Rosowsky, 1998, Akgul and Frangopol, 2004). However, Vu and Stewart (2000) have suggested that the formation of rust products on the steel surface will reduce the diffusion of the iron ions away from the steel surface, thus slowing the corrosion rate with time. Furthermore, the area ratio between the anode and the cathode is reduced, which also suggests that the corrosion rate will reduce with time. Following Vu and

Stewart (2000) and Yokozaki et al. (1997) the corrosion current density at the beginning of the propagation phase, $i_{corr 0}$ (in $\mu\text{A}/\text{cm}^2$), is estimated as:

$$i_{corr 0} = \frac{37.8(1 - w/c)^{-1.64}}{x} \quad \text{Eqn. 4.4}$$

where x is the cover depth in mm and w/c is the water to cement ratio which can be estimated from Bolomey's formula once the concrete strength is known:

$$w/c = \frac{27}{f'_{cyl} + 13.5} \quad \text{Eqn. 4.5}$$

where f'_{cyl} is the concrete compressive strength of a standard test cylinder in MPa. Vu and Stewart (2000) developed the following empirical relationship between the time since initiation and the corrosion rate using data reported by Liu and Weyers (1998a):

$$i_{corr}(t) = 0.85i_{corr 0}(t - T_{i_{marine}})^{-0.29} \quad \text{Eqn. 4.6}$$

where $i_{corr}(t)$ is the corrosion current density after t years of exposure and $T_{i_{marine}}$ is the time of corrosion initiation given marine exposure. $i_{corr}(t)$ is then multiplied by a conversion factor of 0.0232 following Vu and Stewart (2000) to find the time dependent corrosion rate $r_{corr}(t)$ in mm/year. The loss of steel diameter is estimated as shown:

$$D(t) = D_i - r_{corr}(t)(t - T_i) \quad \text{Eqn. 4.7}$$

For the case of de-icing salts, this time dependent corrosion rate is considered inappropriate since it was developed based on limited marine exposure data. Instead, a constant corrosion rate of 0.076 mm/year was adopted from Enright and Frangopol (1998), which was identified from field data on existing bridges exposed to de-icing salts. The loss of diameter was also estimated following Eqn. 4.7 with a constant corrosion rate. Once the time variant reduction in diameter is known, the reduction in area over time can be found as (Enright and Frangopol, 1998, Thoft-Christensen et al., 1996):

$$A(t) = \begin{cases} nD_i^2 \frac{\pi}{4} & \text{for } t \leq T_i \\ n[D(t)]^2 \frac{\pi}{4} & \text{for } T_i < t < T_f \\ 0 & \text{for } T_i > T_f \end{cases} \quad \text{Eqn. 4.8}$$

where n is the number of reinforcement bars; D_i is the initial diameter of steel reinforcement; t is the elapsed time in years; r_{corr} is the rate of corrosion; $D(t)$ is the reinforcement diameter at the end of t years; and T_f is the time when $D(t)$, in theory, reaches zero. Figure 4.3 shows the time dependent loss of steel area, normalized by the initial bar area, and the distributions of the longitudinal bar areas in the columns at three discrete points in time for the base bridge with both marine and de-icing salt exposure. From the figure it can be seen that de-icing salt exposure is more critical and is therefore adopted as the main exposure class for this thesis. Although this exposure class may be more applicable to regions where there is intensive de-icing salt use and high seismic risk such as Japan or Italy, it was chosen for this thesis due to the fact that it is the most critical exposure class.

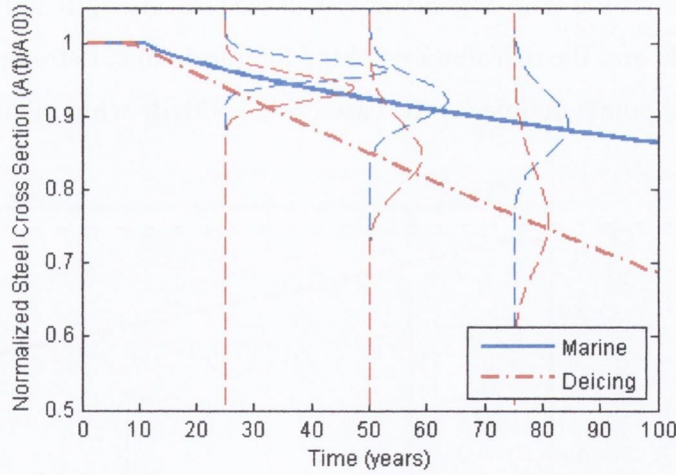


Figure 4.3 Distribution of normalized time variant steel cross sectional area for 40mm diameter column reinforcement

4.2 Loss of Steel Strength due to Corrosion

In addition to a loss of cross sectional area, it has also been suggested in the literature that reinforcing bars experience a reduction in strength with time (Andrade et al., 1991,

Lee et al., 1996, Saifullah, 1994, Morinaga, 1996, Zhang et al., 1995). Andrade et al (1991) reported a 4.5% loss in yield stress with a 10% loss in mass due to corrosion, a conclusion that is supported by the experimental results in the literature cited above. The most comprehensive study to date on the effect of corrosion on the strength of steel bars was carried out by Du et al. (2005b). This study tested the residual strength of corroding bars by performing both accelerated and simulated tests on bare bars and bars embedded in concrete. From the experimental results, the following empirical equation was proposed to predict the residual capacity of corroded reinforcing bars in practice:

$$f_y(t) = [(1 - 0.005A(t))f_{y0}] \tag{Eqn. 4.9}$$

where $f_y(t)$ is the yield strength of corroded reinforcement at time t ; f_{y0} is the yield strength of non-corroded reinforcement; t is the time in years elapsed since corrosion initiation and $A(t)$ is the percentage of steel area loss over time calculated from the consumed area of steel per unit length divided by the original steel area. This model for strength reduction was adopted for this study considering f_{y0} as a random variable. Figure 4.4 shows the time dependent loss of the steel strength, normalized by the initial steel strength, and the distributions of the longitudinal bar strengths in the columns at three discrete points in time for the base bridge with de-icing salt exposure.

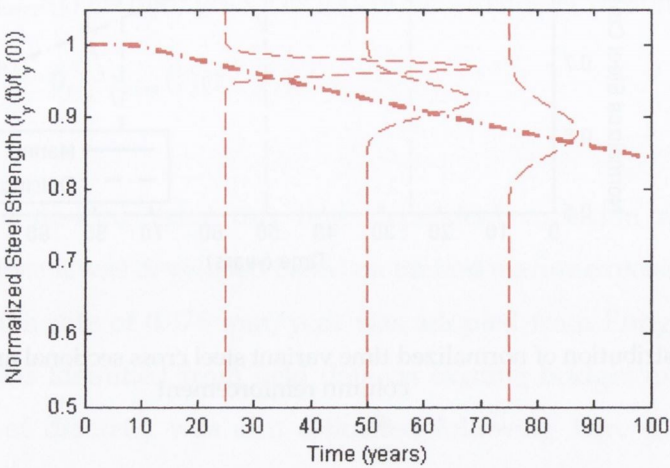


Figure 4.4 Distribution of normalized time variant steel strength for 40mm diameter column reinforcement

4.3 Limitations of Corrosion Model

It must be acknowledged that other possible deterioration mechanisms and effects of component ageing are possible which are outside the scope of this thesis. It has been noted that corrosion of the reinforcing steel in the RC columns and deck may lead to secondary effects such as concrete cracking and spalling, as well as a loss of bond strength between the steel and the concrete (Ghosh and Padgett, 2010). These additional effects of ageing provide an opportunity for future study, however a review of the existing literature suggests that some of these effects may have a limited impact on bridge fragility (Fang et al., 2004, Ghosh and Padgett, 2010).

In terms of the impact of concrete cracking and spalling, Ghosh and Padgett (2010) carried out a preliminary sensitivity analysis which revealed that the effect of concrete cover loss on bridge vulnerability is insignificant. The results of the sensitivity analysis suggest that complete cover loss results in only a 2-6% shift in the median PGA value when compared to the case of no cover loss. In this case, the median PGA value is defined as the PGA at which there is a 50% probability of bridge failure. This is due to the fact that the analytical model for the unconfined concrete described in Section 3.2.1.1 predicts little resistance from the concrete cover at high curvatures due to its low ductility. Additionally, Ghosh and Padgett (2010) note that time dependent modelling of concrete spalling is not a trivial task and it would be expected to increase the uncertainty of the fragility estimation. It must also be noted however that concrete cracking may have a negative impact of concrete durability by reducing the corrosion initiation time which has not been accounted for in the current corrosion model.

A loss of bond strength between the concrete and the steel due to corrosion is also noted as a potential deterioration mechanism. Fang et al. (2004) and Aquino and Hawkins (2007) have shown that for RC members with insufficient splice lengths and no confinement, the loss of bond strength due to corrosion is significant. However, Fang et al. (2004) also suggest that for members with reinforcement confinement, the loss of bond strength due to corrosion is not substantial. As mentioned in Chapter 3, the bridge columns considered in this thesis have transverse reinforcement ratios that are consistent with current design practice in California (Ramanathan, 2012). This is due to the fact that the columns have been designed to resist high impact loads

according to the Eurocode (Eurocode 1, 2006). Therefore, it is assumed that the transverse reinforcement in these columns is adequate to adopt the assumption proposed by Fang et al. (2004). However, it is noted that the lack of literature available on the effect of bond loss on the seismic fragility of RC columns highlights the need for future research in the area.

Finally, some studies have suggested that corrosion of reinforcing steel may lead to a loss of ductility (Zhu et al., 2013, Kashani et al., 2013, Du et al., 2005a, Palsson and Mirza, 2002, Almusallam, 2001). The inclusion of ductility loss due to corrosion provides a very important opportunity for future work since it could potentially lead to brittle failures of concrete members under seismic loads (Coronelli and Gambarova, 2004). However, it remains beyond the scope of the current study.

4.4 Corroded Bridge Models and Updated Deterministic Responses

For this thesis, corrosion of the longitudinal and transverse column reinforcement and the longitudinal deck reinforcement is considered. From Section 4.1, it was seen that exposure to de-icing salts, given the current models adopted from the literature, leads to the most significant amount of steel corrosion. For this reason, de-icing salt exposure is chosen as the exposure class for this study. The reinforcement in the deck slab corrodes from the direct application of de-icing salts, whereas column corrosion can be attributed to traffic spray from the roads (HETEK, 1997). It is assumed the steel in the prestressed girders remains unchanged. The reasoning behind this assumption is twofold. Firstly, the underside of the deck is not directly exposed to chlorides from de-icing salts due to the lack of deck joints for this bridge type and secondly, the girders are fabricated under factory conditions and designed to withstand corrosion. For these reasons, the girders are less vulnerable to chloride ingress.

Corrosion of the longitudinal steel in the RC columns and deck slab is implemented in the OpenSEES model in two ways. Firstly, the cross sectional area of the reinforcing steel bars in the fiber defined cross section described in Section 3.2.1, is reduced according to Eqn. 4.8. Secondly, the yield strength of the *Steel 02* material described in Section 3.2.1.1 is reduced according to Eqn. 4.9. Corrosion of the transverse

reinforcement is accounted for by updating the confined concrete parameters described in Section 3.2.1.,1 to account for a loss of cross sectional area and strength of the transverse (confining) reinforcement.

As a preliminary investigation, the OpenSEES models are updated to include corrosion and the deterministic component responses are compared to the pristine responses. A modal analysis is also carried out on the corroded bridge and compared to the pristine modal analysis. As mentioned previously, it is assumed in this thesis that there is a uniform loss of steel cross section and strength over the length of the reinforcement. For this thesis, the bridge fragility is evaluated at four discrete points along the service life of the structure, namely, 0, 50, 75 and 100 years. 100 years is assumed to be the design life of the bridge and therefore is the maximum age at which the fragility is evaluated. It was found that corrosion did not have a significant effect on the seismic response of the bridge until after 50 years and therefore this is the second point in time at which the fragility is evaluated. The fragility is also evaluated at 75 years, half way between the second and final points. The steel cross section and strength is evaluated at each of these points in time using Eqns. 4.7 - 4.9. The updated steel areas and strengths for the 52 m and 69 m bridges can be seen in Table 4.2.

Table 4.2 Updated steel cross sections and strengths for the 52 m and 69 m bridges

Years	0		50		75		100	
	D (mm)	f _y (MPa)	D (mm)	f _y (MPa)	D (mm)	f _y (MPa)	D (mm)	f _y (MPa)
52 m Bridge								
Column longitudinal	40	460.0	36.9	425.6	35.0	406.2	33.1	388.0
Column transverse	16	460.0	12.9	379.9	11.0	341.2	9.4	323.6
Slab	20	460.0	16.6	408.5	14.7	356.6	12.9	330.1
Slab @ support	28	460.0	24.6	389.8	22.7	382.7	20.9	359.3
70 m Bridge								
Column longitudinal	40	460.0	36.9	425.6	35.0	406.2	33.1	388.0
Column transverse	16	460.0	12.9	379.9	11.0	341.2	9.4	323.6
Slab	32	460.0	28.6	414.4	26.7	391.2	24.8	369.8
Slab @ support	40	460.0	36.6	423.1	34.7	403.8	32.8	385.7

A modal analysis is performed for each bridge geometry in its pristine condition and after 100 years. It was found that there was an insignificant shift in natural period with

age. This result is expected since we are reporting the fundamental elastic period of the pristine and 100 year old bridges and therefore do not expect a change with corrosion.

A time history analysis is then performed on the pristine and 100 year old base bridge by subjecting it to the 0.99 g and 1.1 g ground motion components described in Chapter 3 in the longitudinal and transverse directions separately and simultaneously. A reduction in the reinforcing steel area and strength in the columns and deck is expected to affect the dynamic behaviour and demands on the structure; by comparing a deterministic time history analysis of the pristine and 100 year old bridges, insight will be gained into this effect.

Figure 4.5 shows the moment curvature response at the column top for the pristine and 100 year old bridges when the ground motions are applied in the longitudinal and transverse directions separately. From the figure it is clear that the maximum column curvature increases with bridge age. For example, the maximum curvature when the ground motion is applied in the longitudinal direction increases by over 90% from 0.008 /m to 0.016/m after 100 years. When the ground motion is applied in the transverse direction, the maximum curvature increases by 53%, from 0.005 /m to 0.008 /m.

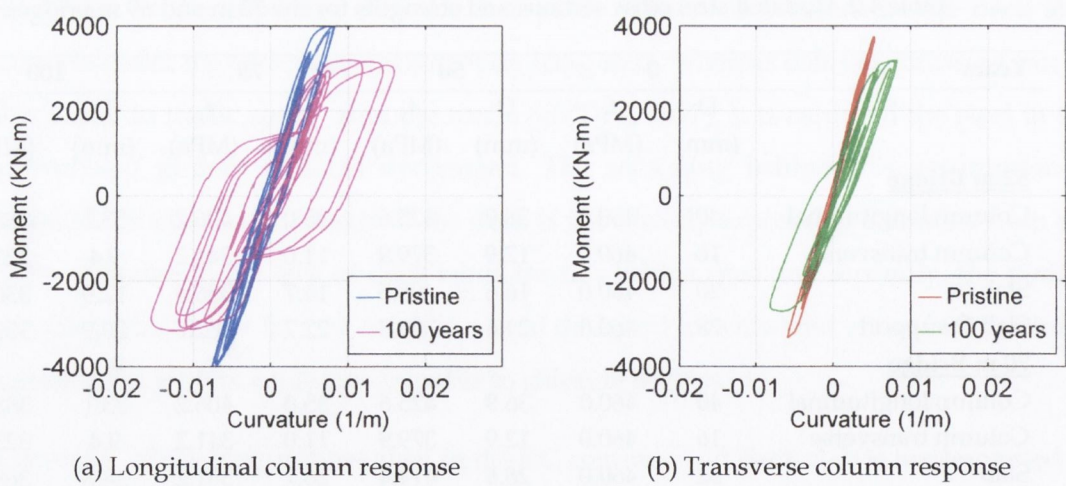


Figure 4.5 Moment curvature response to ground motions applied in the longitudinal and transverse directions separately for the pristine and corroded columns

The moment curvature response when the ground motions are applied in the longitudinal and transverse directions simultaneously is shown in Figure 4.6. From the figure it can be seen that the maximum curvature increases by over 60% from 0.0143 /m to 0.0228 /m in the longitudinal direction and by 48% from 0.012 /m to 0.018 /m in the transverse direction after 100 years.

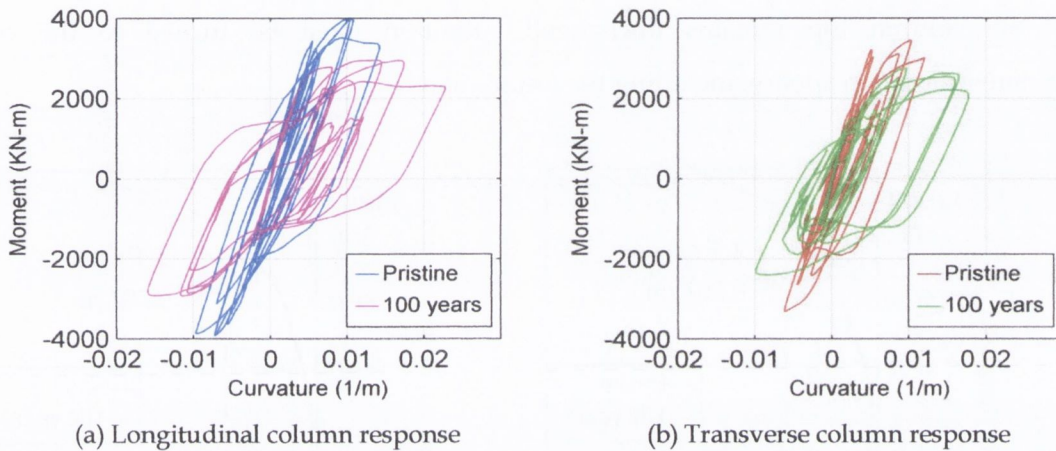


Figure 4.6 Moment curvature response to ground motions applied in the longitudinal and transverse directions simultaneously for the pristine and corroded columns

It must be noted that as well as affecting the dynamic behaviour of the bridge, corrosion and the subsequent loss of steel area and strength, leads to a reduction in the load carrying capacity and yield curvature of a column. This phenomenon is further discussed in Chapter 5 but for illustrative purposes, a plot of the maximum curvature response over the height of the column is shown in Figure 4.7, with the change in yield curvature highlighted. From the figure, it can be seen that the yield curvature is reduced by almost 35% from 0.007 /m to 0.004 /m after 100 years of chloride induced corrosion. These values for yield curvature were estimated through moment curvature analysis and are further described in Section 5.5.1.

Next, the response of the corroded abutments is examined. Figure 4.8 (a) & (b) show the longitudinal and transverse displacement response of the integral abutments. From the figures it can be seen that the displacement time histories for the corroded and pristine abutments are almost identical, with less than a 3% difference between the maximum displacements. This suggests that the effect of corrosion may have a

negligible impact on the abutment fragility; however, a full probabilistic analysis is needed to draw a more definite conclusion, which will be shown in Chapter 6. From Figure 3.18 in the previous chapter, it was seen that displacement response at the abutment is almost identical to the displacement response column top due to the integral design. In order to understand why the maximum curvature response in the column increases with column corrosion but the maximum displacement response at the column top remains unchanged, attention must be turned to the overall substructure response, including the foundations.

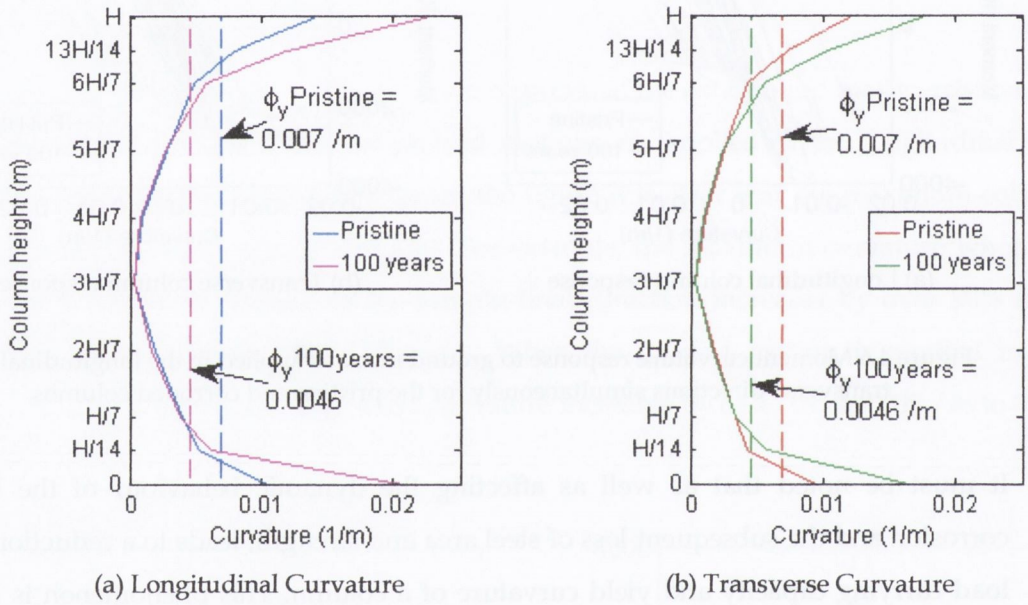


Figure 4.7 Variation of curvature of the height of the column for the pristine and corroded bridges

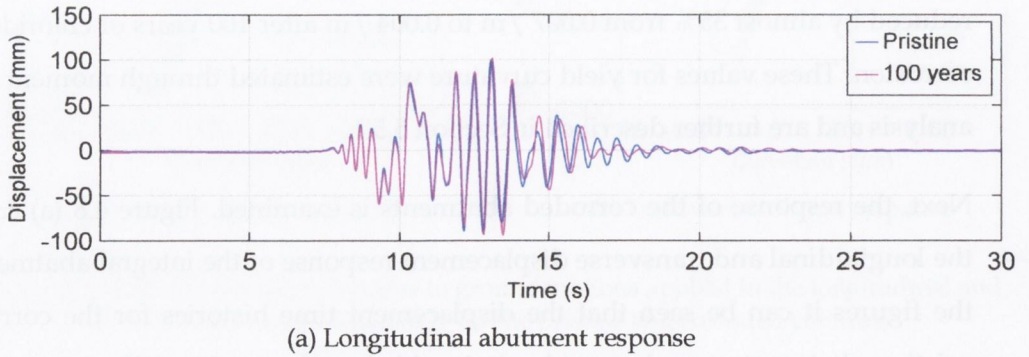
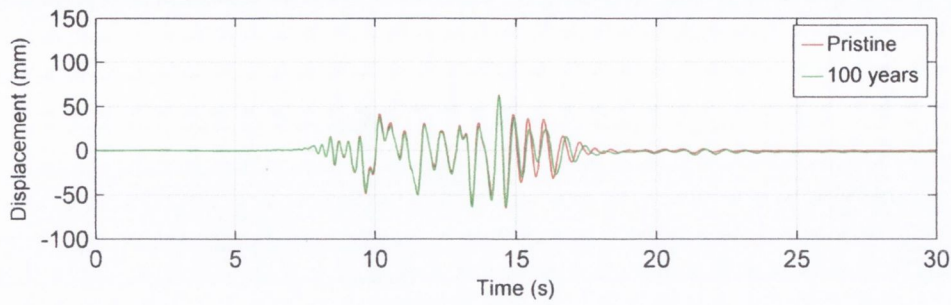


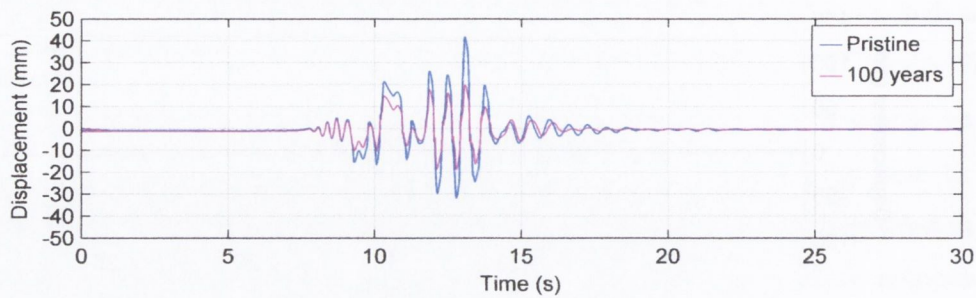
Figure 4.8 (b) Displacement response of the right abutment in the longitudinal and transverse directions for the pristine and corroded bridges



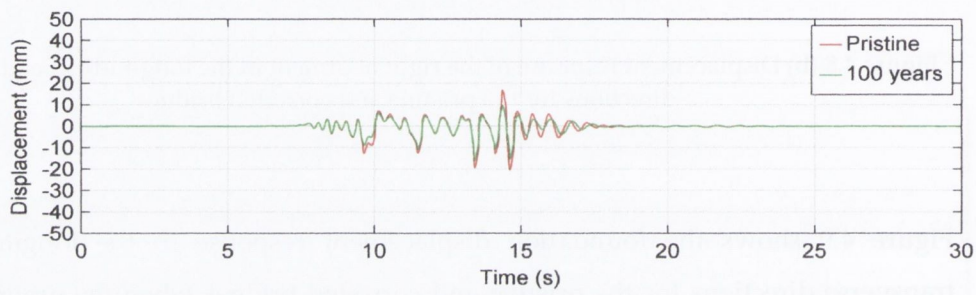
(a) Longitudinal abutment response

Figure 4.8 (b) Displacement response of the right abutment in the longitudinal and transverse directions for the pristine and corroded bridges

Figure 4.9 shows the foundation displacement response in the longitudinal and transverse directions for the pristine and corroded bridges when the ground motions are applied separately. From the figure, it can be seen that the maximum foundation displacement response is lower for the corroded bridge in both the longitudinal and transverse directions. This is further highlighted in Figure 4.10 which shows the foundation force displacement response for the pristine and corroded bridges. From the figure it can be seen that lower forces are developed in the foundation for the corroded bridge. An explanation for this situation may be due to the reduced stiffness in the column as a result of corrosion. With the reduction of steel cross section and strength in the column, it becomes more flexible and therefore less force may be transferred to the foundations during an earthquake. In other words, although the displacement response of the superstructure remains unchanged, the columns are subjects to greater deformation and the foundation deformation is reduced. It must be noted that this result is specific to this particular earthquake and the seismic response may change for a different set of ground motion records. Therefore, a probabilistic analysis is vital before any formal conclusions are drawn.

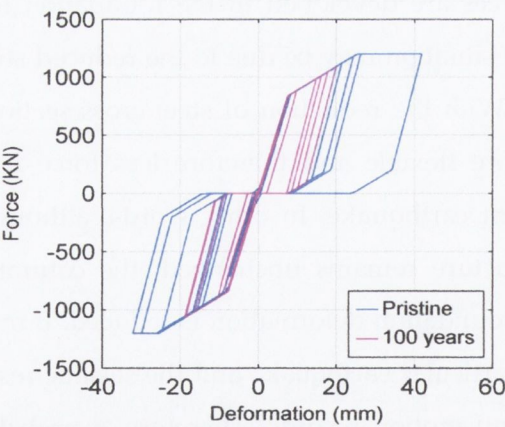


(a) Longitudinal foundation response

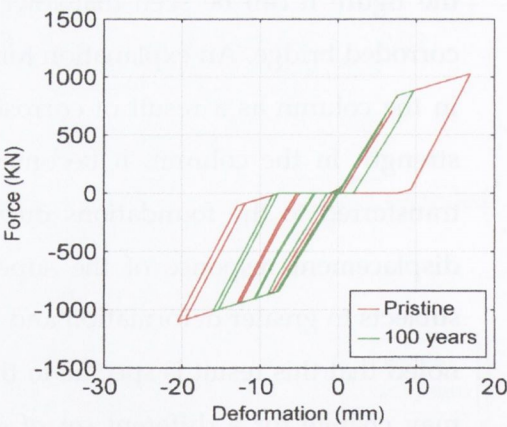


(b) Transverse foundation response

Figure 4.9 Displacement response of the right foundation for the pristine and corroded bridges



(a) Longitudinal foundation response



(b) Transverse foundation response

Figure 4.10 Force deformation response of the right foundation for the pristine and corroded bridges

Finally, the demands on the deck are compared for the pristine and corroded bridges. An envelope of the maximum bending moment along the length of the bridge is shown in Figure 4.11 for the pristine and 100 year old bridges. From the figure it can be seen that there is a negligible ($< 3\%$) decrease in the maximum hogging bending moment with corrosion, whereas the maximum sagging bending moment increases by roughly 20% with corrosion. Despite the fact that corrosion has negligible impact on the hogging deck demands, it has a significant effect on the deck ultimate limit state capacities which are developed in Chapter 5. Therefore, it is anticipated that corrosion may still have an impact on deck fragility.

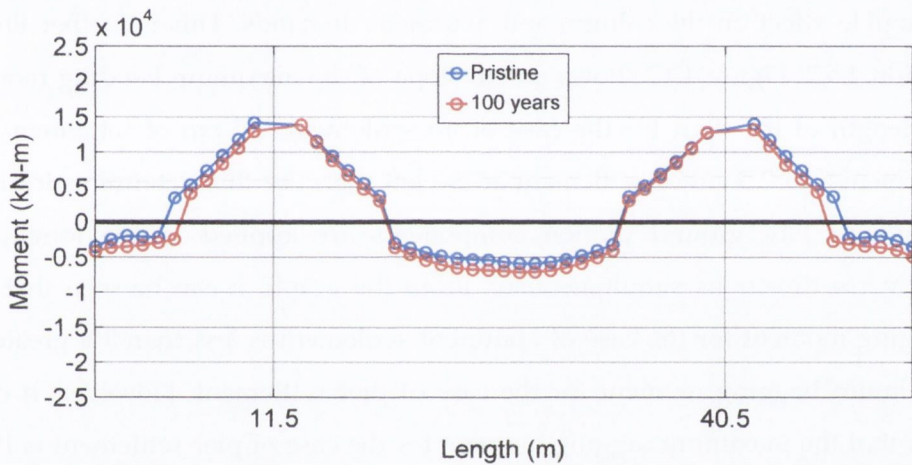


Figure 4.11 Envelope of maximum deck bending moments over bridge length for the pristine and corroded bridges

4.5 Foundation Settlement

4.5.1 Quantification of Foundation Settlement

This section describes the approach used to account for long term foundation settlement in the seismic assessment of an integral bridge. As mentioned previously, foundation settlement has been cited as a main form of deterioration of highway bridges (FHWA, 2006). The task of predicting settlement with time is a very complex problem dependent on site specific conditions and uncertain parameters that remains unaddressed in the literature. Therefore, in order to get a realistic estimate of the maximum feasible settlement levels for the highway bridge under consideration, a

survey on the foundation movement of 314 bridges across the US and Canada is consulted (FHWA, 1985). The survey recorded the vertical movements of 580 abutments and 1068 piers for the 314 bridge included in the survey. This study found that 65% of the abutments surveyed experienced vertical movement and 22% of the piers surveyed experienced vertical movement.

In order to decide whether settlement of the piers or the abutments should be considered, a preliminary deterministic analysis is carried out to investigate the effect of abutment and pier settlement on the deck demand. The effect of settlement on the deck demand is investigated because it was found that foundation settlement has a negligible effect on the column and abutment demands. This is further illustrated in Section 4.5.2. Figure 4.12 shows an envelope of the maximum bending moment along the length of the deck for the case of no settlement, 2.5 cm of settlement at the left abutment and 2.5 cm of settlement at the left pier. For this deterministic analysis, the 0.99g and 1.1g ground motion components are applied in the longitudinal and transverse directions simultaneously. From the graph, it can be seen that maximum hogging moment for the case of abutment settlement is less than 3% greater than the maximum hogging moment for the case of pier settlement. However, it can also be seen that the maximum sagging moment for the case of pier settlement is 19% greater than the maximum sagging moment for the case of abutment settlement. For this reason, foundation settlement of an intermediate pier is considered for this study.

From the results of the survey, two discrete levels of settlement are identified corresponding to a low and high level of settlement. These values are taken as 2.5 cm and 5 cm for the low and high levels respectively. It is noted that the allowable level of differential settlement considered in design, as specified by codes of practice, is generally taken as 2 cm (WSDOT, 2013). These discrete levels of settlement are applied as forced displacements to the left hand pier foundation as shown in Figure 4.13, using the single point constraint command provided on OpenSEES.

As mentioned previously, it was found that 22% of piers surveyed experienced vertical movement. It is noted that although this finding suggests that the problem of settlement is much less widespread than the problem of reinforcement corrosion, its effect on the seismic response of a bridge remains a worthwhile study yet to be

explored. It is important to note that this study does not attempt to predict how likely these levels of settlement are to occur, but is instead concerned with evaluating the effect a given level of settlement has on the seismic fragility of the bridge. This will help determine whether foundation settlement should be included in future seismic risk assessments of bridges in regions where it is found to be an issue.

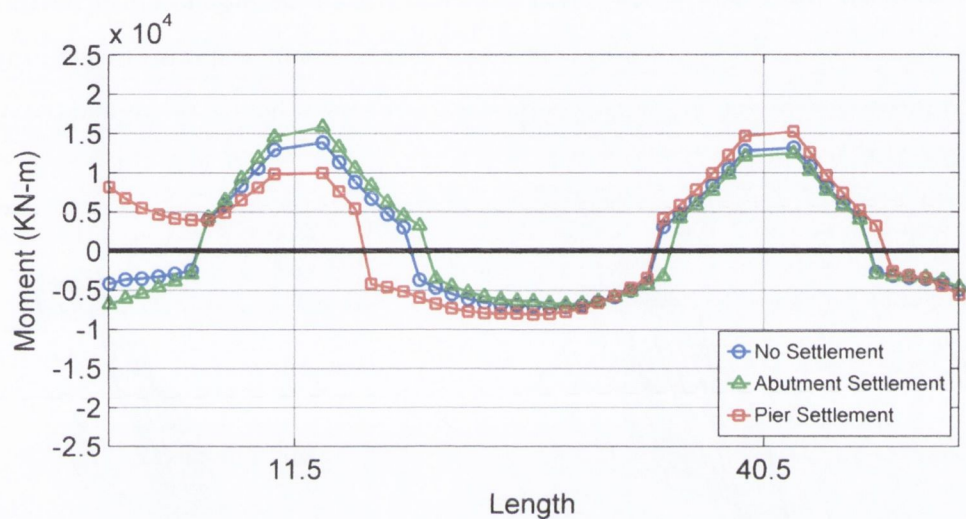


Figure 4.12 Envelope of maximum deck bending moments over bridge length for the case of no settlement, 2.5 cm of settlement at the left abutment and 2.5 cm of settlement at the left pier

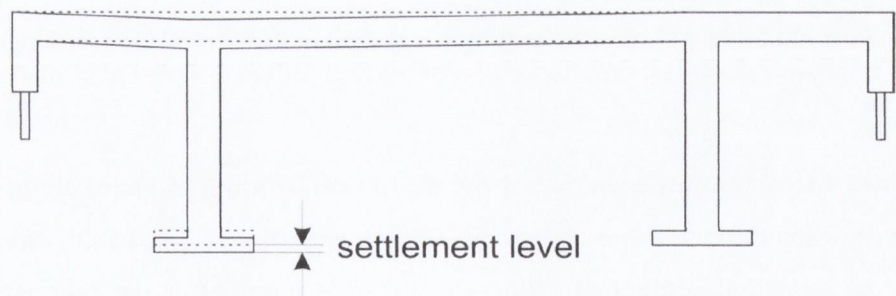


Figure 4.13 Foundation settlement applied to intermediate pier

4.5.2 Updated Deterministic Component Responses with Settlement

As mentioned previously, the three components considered in the current study are the abutments, columns and deck. According to intuition, it is expected that foundation

settlement will have the greatest impact on the deck's seismic response. However, all three component responses, given settlement levels of 2.5 cm and 5 cm are compared with the case of no settlement. For the current comparison, the 0.99 g and 1.01 g ground motion records are applied in the longitudinal and transverse directions simultaneously. Figure 4.14 shows the moment curvature response of the column top. From the figure it can be seen that settlement has a negligible effect on the column response. Figure 4.15 shows the abutment displacement response in the longitudinal and transverse directions. Again, settlement appears to have a negligible impact on the abutment response.

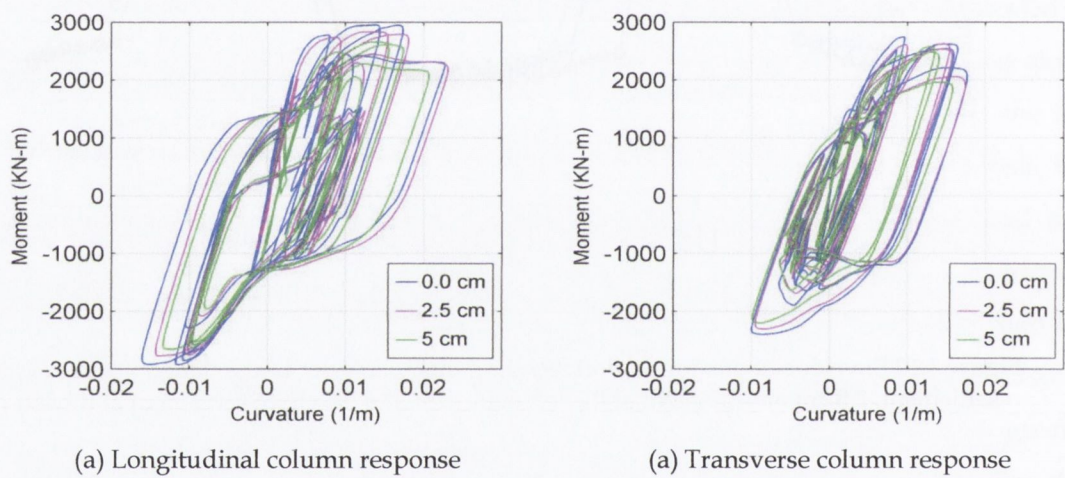
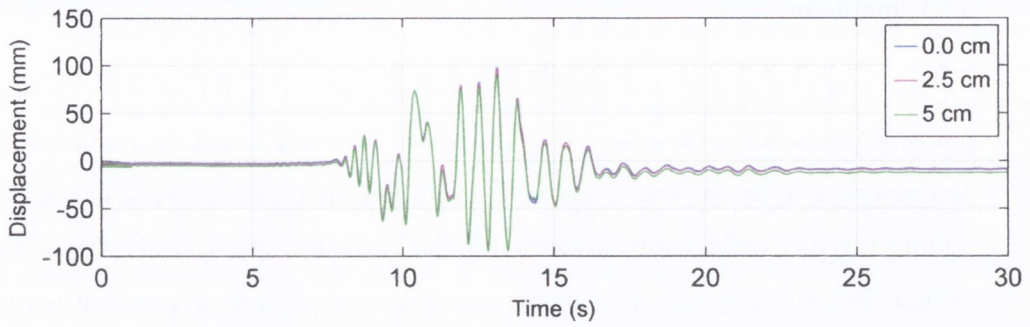
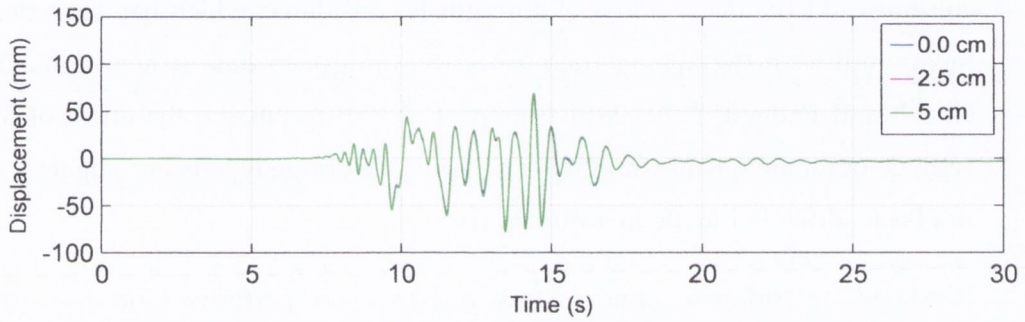


Figure 4.14 Moment curvature response to ground motions applied in the longitudinal and transverse directions simultaneously for the case of 0 cm, 2.5 cm and 5 cm of settlement

Figure 4.16 shows an envelope of the maximum bending moment along the length of the deck for the case of no settlement, 2.5 cm of settlement and 5 cm of settlement. It can be seen that settlement induces a negative moment at the pier where it occurs which reduces the bending moment demand at that location. At the other pier however, the deck experiences an increase in bending moment demand with settlement. For this particular earthquake, the bending moment in the deck experiences a 10% and 25% increase in demand for the low and high levels of settlement respectively. Once again, this result is specific to this particular set of ground motions and a full probabilistic analysis is needed to draw conclusions on the effect of settlement on the seismic vulnerability of this bridge type.



(a) Longitudinal abutment response



(b) Transverse abutment response

Figure 4.15 Displacement response of the right abutment for the case of 0 cm, 2.5 cm and 5 cm of settlement

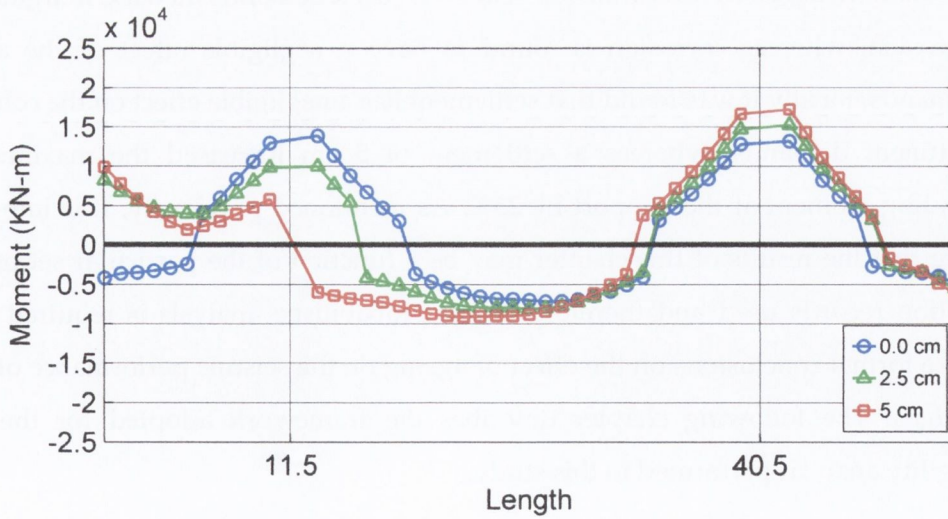


Figure 4.16 Envelope of maximum deck bending moments over bridge length for the case of 0 cm, 2.5 cm and 5 cm of settlement

4.6 Conclusion

This chapter describes the probabilistic approach adopted in this thesis to model chloride induced corrosion, as well as the approach used to quantify foundation settlement. It is noted that this thesis improves on previous work carried out on the ageing fragility of highway bridges. Firstly, corrosion of the deck slab is investigated which has not been included in previous studies on time dependent seismic bridge fragility. Secondly, as well as considering a loss of reinforcement cross section in the columns and the deck, a loss of strength is considered which has been neglected in most studies on the ageing fragility of RC bridges to date (Choe et al., 2009, 2008, Ghosh and Padgett, 2012, 2010, Simon et al., 2010). Finally, the effect of foundation settlement on the seismic fragility of this integral bridge type is investigated, which has not been addressed in the literature to date.

Updated deterministic time history analyses are performed on the pristine and deteriorated bridges and the results are compared in order to gain insight into the effect of ageing on the dynamic response of this integral bridge type. From the results of the deterministic analyses, it is shown that the column demands increase significantly with corrosion. The maximum column curvature in the longitudinal direction increases by 90% after 100 years. The deck demands increase marginally with corrosion, whereas corrosion is found to have a negligible effect of the abutment demands. Finally it was found that settlement has a negligible effect on the column and abutment demands, whereas a settlement of 5 cm increased the maximum deck bending moment at the support by 25%. As mentioned previously, it is important to note that the results of this chapter may be a function of the particular set of ground motion records used and therefore, a full probabilistic analysis is required to draw more formal conclusions on the effect of ageing on the seismic performance of integral bridges. The following chapter describes the framework adopted for the seismic fragility analysis performed in this study.

CHAPTER 5 - TIME DEPENDENT FRAGILITY ANALYSIS

FRAMEWORK

This chapter outlines the framework used to develop time dependent fragility curves. As mentioned previously, fragility curves give the probability of reaching a defined damage state for a given ground motion intensity. They are very useful tools for seismic risk assessment and in the case of this thesis, provide an effective approach to determine the effect of deterioration on the seismic performance of bridges. The multiphase fragility framework adopted herein consists of several steps including quantification of the hazard level, seismic demand and seismic capacity. This thesis adopts and adapts the fragility framework proposed by Nielson (2005) to include a time dependent estimation of both seismic demand and capacity. The following sections in this chapter provide an overview of the developed time dependent fragility method as well a detailed description of each of the individual steps.

5.1 Overview of the Time Dependent Fragility Analysis Method

Figure 5.1 shows a schematic of the time dependent fragility framework and its essential components. The main steps involved in a seismic fragility analysis are:

- Selection of a suitable ground motion suite
- Development of a group of stochastic finite element models representing the response at various points along the service life of the structure
- Generation of capacity models at various points along the service life of the structure
- Generation of a time dependent probabilistic seismic demand model (PSDM)
- Formulation of time dependent fragility curves

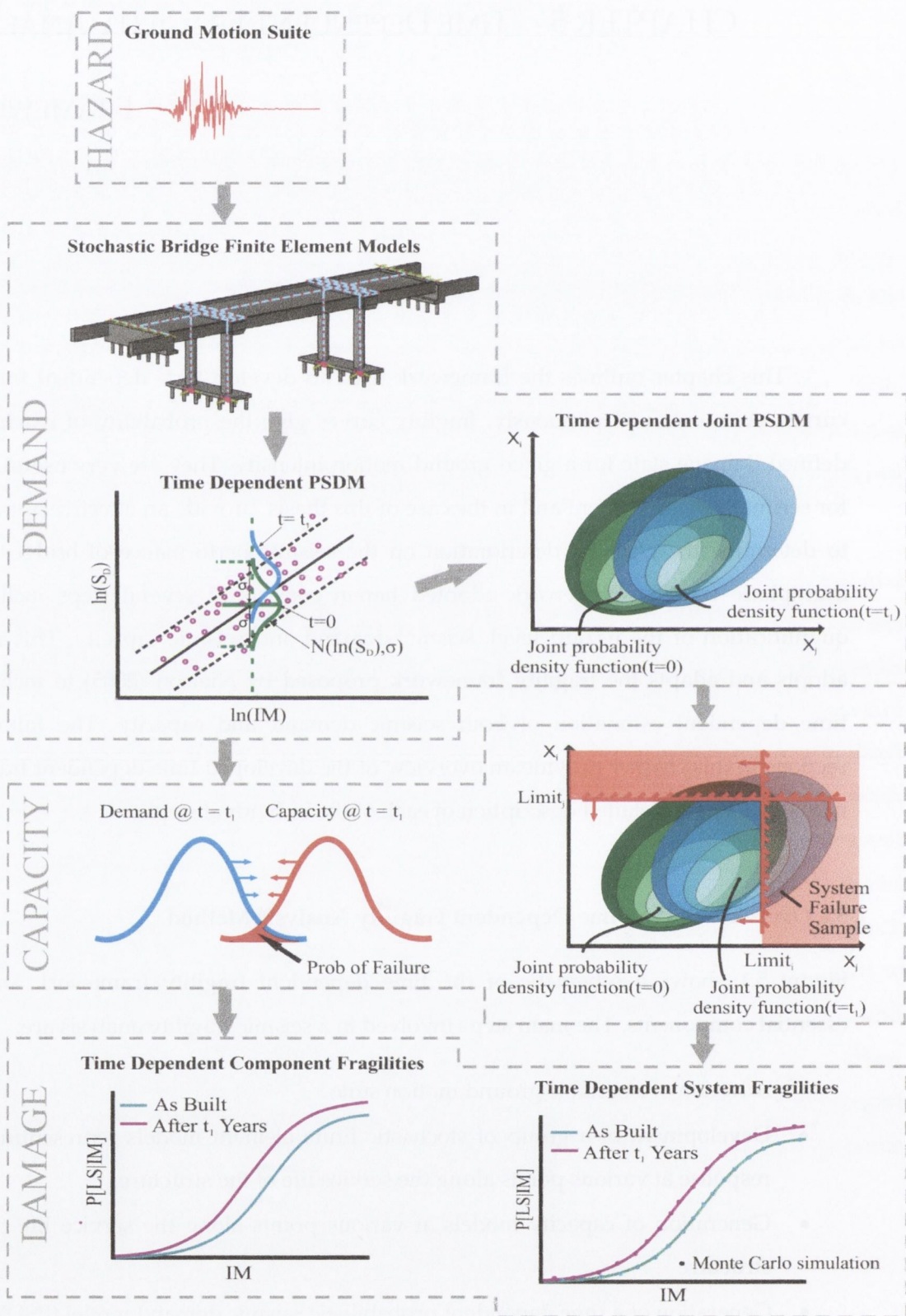


Figure 5.1 Schematic of time dependent fragility framework

The first step of the fragility analysis is to assemble a suite of N ground motions which are representative of seismicity in the geographic location of interest. The next step is to develop N nonlinear finite element bridge models, to be randomly paired with each of the ground motion records selected. These models should be statistically different yet nominally identical. They are generated in the context of this thesis using the OpenSEES finite element platform (McKenna and Fenves, 2010), by sampling on the probability distributions of uncertain structural parameters (e.g. material strength, soil stiffness, damping etc). A series of N nonlinear time history analyses are then carried out using the finite element model and ground motion pairs. The maximum responses of the components thought to contribute to the seismic vulnerability of the bridge system are recorded. The pristine finite element models are then modified to account for deterioration and the process is repeated at various points along the service life of the structure, which are taken as 0, 50, 75 and 100 years for this thesis. The justification for choosing these points was discussed in Section 4.4.

Probabilistic seismic demand models (PSDM) are developed for each of the components, at each of the discrete points in time considered in the analysis. This provides a probabilistic model for each of the component demands for a given ground motion intensity level. Once a model for seismic demand is established, attention is then turned to the seismic capacity of the components considered. Performance based seismic engineering (PBSE) requires that different levels of structural capacity are defined, referred to as limit states, each of them corresponding to an operational consequence. Probabilistic capacity models are developed for each of the component limit states based on experimental results or expert opinion based on observed damage during past earthquakes. Similar to the demand models, the capacity models are developed at each bridge age considered in the analysis. The demand and capacity are assumed to be lognormally distributed at each point in time along the bridge's service life (Ghosh and Padgett, 2012, Ghosh and Padgett, 2010, Cornell et al., 2002) and therefore, the time dependent component fragility can be expressed as (Melchers, 1999):

$$P\left(\frac{Demand(t)}{Capacity(t)} > 1.0|IM\right) = \Phi\left(\frac{\ln(S_D(t)/S_c(t))}{\sqrt{\beta_{D|IM}^2(t) + \beta_c^2(t)}}\right) \quad \text{Eqn. 5.1}$$

where $S_D(t)$ and $\beta_{DIM}^2(t)$ are the median and dispersion of the lognormally distributed seismic demand and $S_c(t)$ and $\beta_c^2(t)$ are the median and dispersion of the lognormally distributed component capacity.

Although a component fragility analysis offers valuable insight into the vulnerability of an individual bridge component, a system fragility analysis is required to gain an understanding of the overall bridge behaviour, as well as for loss estimation packages. In order to formulate system fragility curves, a joint probabilistic seismic demand model (JPSDM) is developed by combining the individual component PSDMs, accounting for correlation between the component demands, considering they are not independent. A joint capacity distribution is then developed which assumes the component capacities are statistically independent. Realizations of the JPSDM are compared with those from the joint capacity distribution using Monte Carlo simulation to derive system fragility curves. It should also be noted that assessment of the bridge system reliability is carried out by assuming the bridge as a series system. Therefore, it is important that component limit states are combined in such a way that they have similar operational consequences, as further described in Section 5.5. The subsequent sections of this chapter describe in more detail the various steps of the time dependent fragility framework.

5.2 Ground Motion Suite

As mentioned in the previous section, the first step of any fragility analysis is to assemble a suite of ground motions that accurately characterize the seismic hazard in the geographic location of interest. It is important for the ground motion suite to have sufficient samples to propagate aleatoric randomness inherent in the occurrence of seismic events. For this study, 100 ground motions are chosen following recommendations by Nielson and Mackie (2009). It is also important to have a suite of ground motion records that covers the range of intensity levels expected in the area where the bridge being assessed is located. Additionally, the suite must propagate uncertainty in the realization of other earthquake characteristics such as magnitude and epicentral distance (Ramanathan, 2012). This thesis employs a suite of 100 ground motions identified by Gupta and Krawinkler (2000) and Krawinkler et al. (2003) from

the PEER Strong Motion Database (PEER, 2013) and the SAC project database (SAC, 2013) for the typical site conditions of the California region. The moment magnitudes of the ground motion suite range from 4.7 to 7.0 and the epicentral distances range from 3.6 km to 60 km.

Each of the ground motion records consist of two orthogonal components which are applied to the finite element bridge models in the longitudinal and transverse directions. For this thesis, ground motion records are not applied to the finite element models in the vertical direction, following previous studies on the seismic fragility of highway bridges (Nielson, 2005, Padgett, 2007, Ghosh and Padgett, 2010, Ramanathan et al., 2010). However, it is important to note that the inclusion of vertical ground motions in the fragility analysis may lead to higher deck demands and therefore provides an opportunity for future work. The intensity measure chosen for this study is peak ground acceleration (PGA) based on findings of its validity for probabilistic seismic demand modelling for typical bridge classes (Padgett et al., 2008) and consistent efficiency found for the case study bridges herein. The ground motion suite is characterized by PGAs between 0.03g and 1.3g. The durations of the earthquakes range between 18.7 seconds and 99.96 seconds. Figure 5.2 shows histograms of the PGAs and durations of the chosen earthquake records.

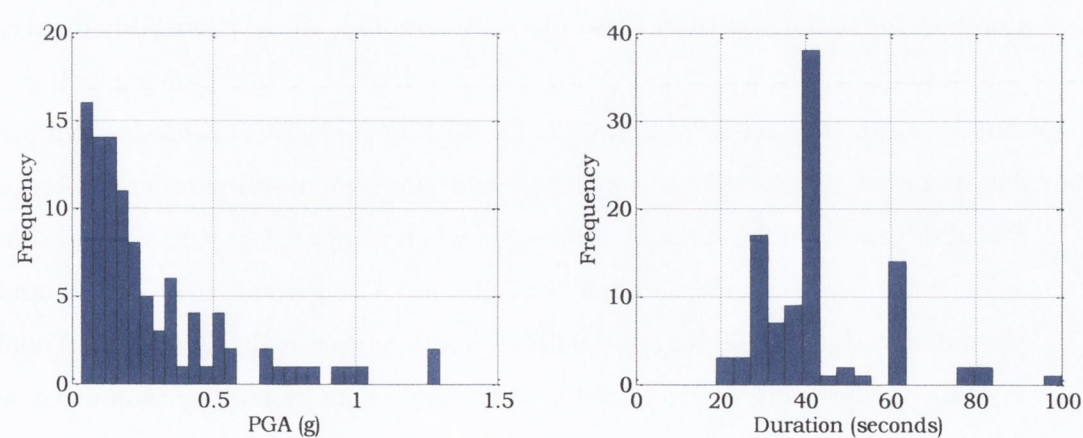


Figure 5.2 Histograms of the PGAs and durations of the ground motion suite

Also of interest are the response spectra of the earthquake records which show the response of a single degree of freedom structure, of varying natural period, to the

ground motion input. The response spectra of the X and Y components of the ground motion records are shown in Figure 5.3, with the mean response spectra for each direction also highlighted.

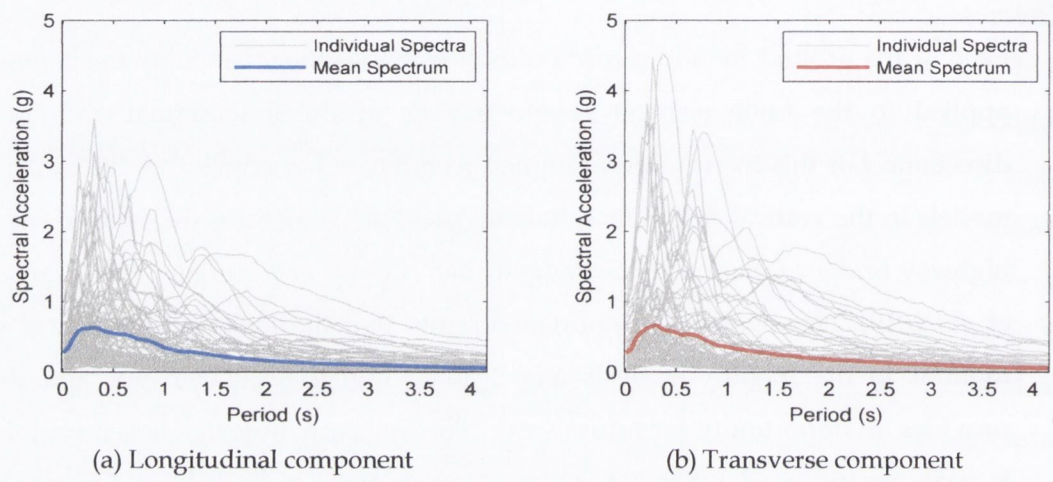


Figure 5.3 Spectral acceleration plots (4.5% damping) for the ground motion suite

5.3 Generation of Stochastic Bridge Models

The next step of the time dependent fragility framework is to generate 100 statistically significant yet nominally identical finite element bridge models, at each point in time along the life of the structure. This is done by sampling on the probability distributions of uncertain structural parameters, which are described in the following section of this chapter. Latin Hypercube Sampling (LHS) (McKay et al., 1979) is used to sample from the structural parameters as it provides an effective stratified scheme to cover the probability space of the random parameters when compared to pure random sampling using naive Monte Carlo Simulation (Celik and Ellingwood, 2010). Each randomly generated bridge sample is paired with a ground motion realisation and 100 nonlinear dynamic analyses are carried out for the bridge in its pristine condition (i.e. after 0 years) and after 50, 75 and 100 years.

Once the stochastic bridge models are generated, it is necessary to decide which bridge components may contribute to the overall bridge seismic vulnerability. For this integral bridge type, the deck, abutments and columns are included in the fragility analysis. It must be noted that the deck is typically neglected from seismic fragility analyses of

highway bridges, as it is assumed to remain elastic during earthquake events (Aviram et al., 2008). However, these assumptions were made based on the bridge in its as-built condition, therefore, it needs to be investigated whether the inclusion of deck deterioration results in demands exceeding reduced limit state capacities. An engineering demand parameter (EDP), or a measure of the structural response, is then chosen for each of the components. Bending moment, displacement and curvature are the chosen EDPs for the deck, abutments and columns respectively.

5.4 Uncertainty in Modelling Parameters

A number of sources of uncertainty exist in the seismic fragility assessment of bridges. It is generally acknowledged that the two main types of uncertainty present in the seismic reliability assessment of bridges are known as aleatoric or epistemic. Aleatoric uncertainty refers to that which is completely random whereas epistemic uncertainty is associated with a lack of knowledge and assumptions in the modelling techniques. The inherent variability associated with the occurrence of seismic events or in the material properties of the bridge such as the steel strength, can be referred to as aleatoric. Epistemic uncertainties, however, are associated with a lack of knowledge of the actual bridge parameters such as the pile and soil stiffness and can be reduced with the acquisition of additional information and understanding (Ellingwood and Wen, 2005). This section presents the probabilistic models adopted for the uncertain parameters used in this study. Previous studies that have performed fragility analyses on groups of bridges have included geometric uncertainty in the analyses (Nielson, 2005, Padgett, 2007, Ramanathan, 2012), however, since this study treats geometric variation in separate analyses, this source of uncertainty is avoided here.

5.4.1 Concrete Strength

The cast-in-place concrete used to construct the columns and deck slab for this integral bridge has a characteristic strength of 30 MPa. Following Ramanathan (2012), it is assumed the concrete strength is normally distributed with a mean of 30 MPa and coefficient of variation (COV) of 0.12. The prestressed U beams are modelled using 60MPa concrete with a COV of 0.12. Following previous studies on the seismic

performance of ageing bridges (Choe et al., 2009, Ghosh and Padgett, 2010, Alipour et al., 2011, Akiyama et al., 2011), it is assumed the concrete strength remains unchanged with deterioration.

5.4.2 Steel Strength

For this bridge, grade 460 steel reinforcement bars are used in the concrete columns and deck slab. For the prestressed concrete beams, 15.7 mm superstrands are used with a specified yield strength, f_y , of 1500 MPa. Ellingwood and Hwang (1985) proposed that the yield strength, f_y , of grade 60 (414 MPa) steel is lognormally distributed with a mean of 465.79 and COV of 0.08. In the absence of data on the distribution of grade 460 steel, a lognormal distribution with a mean of 460 MPa and a COV of 0.08 is assumed conservatively. For the superstrands with a yield strength of 1500 MPa, a lognormal distribution is assumed with a mean of 1500 MPa and a COV of 0.08. The distribution of the reinforcing steel strength is also time dependent and changes with bridge age according to Figure 4.3 in Chapter 4.

5.4.3 Steel Cross Sectional Area

Following Enright and Frangopol (1998), the initial diameter of the steel reinforcement is assumed to follow a lognormal distribution with a COV of 0.02. The mean bar areas vary depending on the location of the steel and the bridge geometry. The distribution of the reinforcing steel area is also time dependent and changes with bridge age according to Figure 4.2 in Chapter 4.

5.4.4 Concrete Cover

The concrete cover is assumed to follow a lognormal distribution with a COV of 0.2, adopted from Enright and Frangopol (1998). The distribution of the cover depth is assumed to remain constant with time due to the lack of information in the literature to develop a time dependent model.

5.4.5 Abutment Active Stiffness

As shown in Chapter 4, the abutment active stiffness is defined by the horizontal stiffness of the piles. This thesis assumes an initial horizontal stiffness of 7

kN/mm/pile, following recommendations by Caltrans (1990). Following Nielson (2005), a uniform distribution with bounds which are 50% and 150% of this recommended value is assumed to incorporate uncertainty in the initial pile stiffness. This gives bounds of 3.5 kN/mm/pile and 10.5 kN/mm/pile for the uniform distribution.

5.4.6 Abutment Passive Stiffness

This thesis uses a validated hyperbolic relationship proposed by Shamsabadi et al. (2010) between the lateral load on the abutment wall and the wall deflection, as described in Chapter 3. This relationship is based on experimental testing of abutment walls with a cohesionless backfill, with an average abutment stiffness of 29 kN/mm/m and a cohesive backfill, with an average abutment stiffness of 14.5 kN/mm/m. It is therefore assumed for this thesis that the abutment stiffness follows a uniform distribution with these bounds following Nielson (2005).

5.4.7 Translational Foundation Stiffness

From Chapter 3, it is recalled that the translational stiffness of the pier foundations is linearly related to the horizontal stiffness of the piles, which have a uniform distribution with bounds of 3.5 kN/mm/pile and 10.5 kN/mm/pile. Since each foundation of this bridge has 12 piles, the bounds of the translational foundation stiffness are taken as 42 kN/mm and 126 kN/mm.

5.4.8 Rotational Foundation Stiffness

From Chapter 3, the rotational stiffness of the foundations is calculated directly from the vertical stiffness of the individual piles. A value of 175 kN/mm is assumed for the vertical stiffness of the piles following Choi (2002). Following Nielson (2005), the distribution is assumed to be uniform with bounds being 50% and 150% of the suggested value. This gives bounds of 87.5 kN/mm and 262.5 kN/mm for the uniform distribution.

5.4.9 Mass

The mass of the superstructure is considered variable due to a number of sources including the presence of parapets and barriers, variation in deck slab thickness, electricity poles and other equipment, re-pavement procedures and variation in the material densities etc. (Ramanathan, 2012). Therefore, it is assumed that the variation in the mass can be accounted for by a mass factor with a uniform distribution with bounds of 1.1 and 1.4, following Ramanathan (2012). Ramanathan (2012) estimated the additional mass from a review of Californian bridge plans.

5.4.10 Damping Ratio

Fang et al. (1999) recommended that the uncertainty in the damping ratio for tall buildings is modelled using a normal distribution. The recommendation is extended to bridges for this study following Nielson (2005). Bavirisetty et al. (2003) estimated the 2nd and 98th percentile of the damping ratio in bridges to be 0.02 and 0.07 respectively and using these values, the damping ratio is sampled from a normal distribution with a mean of 0.045 and a COV of 0.278.

5.4.11 Loading Direction

Although previous studies have considered uncertainty in the direction from which the main component of an earthquake strikes a bridge (Nielson, 2005, Padgett, 2007), a recent study by Mackie et al. (2011) demonstrated that the angle of incidence had a negligible effect on the mean ensemble response of bridge components. Following this finding, the angle of incidence is not considered as a source of uncertainty in this thesis.

5.5 Component Limit States

The time dependent seismic fragility of a highway bridge component, as shown in Eqn. 5.1, requires knowledge of the time dependent seismic demand for a given ground motion intensity, as well as the time dependent seismic capacity for a given limit state. The predictive model for the time dependent seismic demand is described in the

following section. This section describes the approach used to develop capacity models for the individual bridge components at the limit states considered in this thesis.

The three main components considered in this thesis are the deck, columns and abutments. The definition of the component limit states is not a trivial task due to the fact that each limit state should be associated with a description of damage that can be identified by a bridge inspector in the aftermath of an earthquake. The qualitative limit states adopted for this thesis are those defined in the risk assessment package, HAZUS-MH (FEMA, 2005). The four damage states described in HAZUS are referred to as slight, moderate, extensive and complete and their associated damage descriptions shown in Table 5.1. The component limit state distributions are assumed to be lognormal and are characterized by representative values for the median, S_C , and the logarithmic standard deviation, β_C , referred to as dispersion herein.

Table 5.1 Damage state descriptions as defined by HAZUS-MH (FEMA, 2005)

Limit State	Description
Slight	Minor cracking and spalling to the abutment, cracks in shear keys at abutments, minor spalling and cracks at hinges, minor spalling at the column (damage requires no more than cosmetic repair) or minor cracking to the deck.
Moderate	Any column experiencing moderate (shear cracks) cracking and spalling (column structurally still sound), moderate movement of the abutment (<5 cm), extensive cracking and spalling of shear keys, any connection having cracked shear keys or bent bolts, keeper bar failure without unseating, rocker bearing failure or moderate settlement of the approach.
Extensive	Any column degrading without collapse – shear failure – (column structurally unsafe), significant residual movement at connections, or major settlement approach, vertical offset of the abutment, differential settlement at connections, shear key failure at abutments.
Complete	Any column collapsing and connection losing all bearing support, which may lead to imminent deck collapse, tilting of substructure due to foundation failure.

A further challenge arises when defining the limit states for system damage. The challenge this poses is that limit states for each of the components must be defined in

such a way that their consequences on the bridge functionality are equal. For example, it must be decided what level of curvature in the columns will affect the functionality of the bridge to the same degree as a chosen bending moment in the deck. This issue is dealt with by dividing components into primary and secondary categories based on their contribution to bridge system damage, following Ramanathan (2012). For this thesis, the deck and columns are defined as primary components since they affect the vertical stability and the load carrying capacity of the bridge. Extensive or complete damage of these components could therefore lead to closure of the bridge (Ramanathan, 2012). The abutments, however, are considered secondary components based on the findings of a past survey of bridge engineers and inspectors carried out by Padgett and DesRoches (2007). Although their failure may lead to restrictions on the travel speed and traffic restrictions on the bridge, it will generally not force closure (Ramanathan, 2012). Ramanathan (2012) developed this approach in close collaboration with Caltrans to ensure that the limit states were aligned with the inspection/maintenance closure decisions and training guides for post earthquake inspections.

A significant contribution of this thesis is the development of time dependent capacity models for the columns and the deck. Corrosion leads to a reduction in the column and deck capacities over time, due to the loss of steel cross sectional area and strength. Some previous studies on the ageing fragility of bridges have only focussed on the effect of corrosion on the seismic demand of the bridge (Ghosh and Padgett, 2010, Ghosh and Padgett, 2012, Alipour et al., 2011), whereas this thesis accounts for corrosion effects on both the seismic demand and capacity. For this thesis, abutment limit states are assumed to remain constant with time because the effect of deterioration on their displacement capacities is considered negligible, as field data suggests that corrosion and deterioration of the abutments is overshadowed by that of other bridge components (Simon et al., 2010).

5.5.1 Columns

For the columns, curvature is adopted as the engineering demand parameter (EDP). Curvature limits are found through moment curvature analysis and are characterized with respect to concrete compression and steel tension strain limits taken from the

literature. A moment curvature plot is shown in Figure 5.4, which identifies the curvatures adopted corresponding to the slight, moderate and complete damage states.

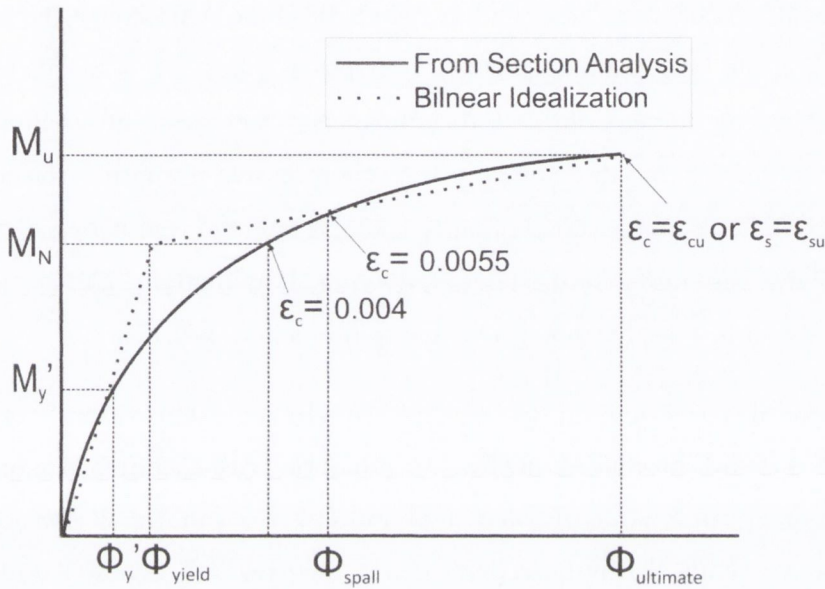


Figure 5.4 Definition of column limit states from moment curvature analysis

The slight damage state is defined as the yield curvature, which is found by extrapolating the first yield curvature, ϕ_y' , to the nominal flexural strength, M_n . M_n is taken as the flexural moment at a maximum concrete compressive strain of 0.004, which corresponds to the onset of concrete crushing (Elwood and Eberhard, 2009). The moderate damage state is defined as the curvature at which concrete spalling occurs. A review of values for ϵ_{spall} reported in the literature found that there was a significant level of variation in the values reported. It was found that the reported values ranged from 0.003 to 0.008 (Akkari and Duan, 2003, Berry and Eberhard, 2003, Caltrans, 2006). For this thesis, a value of 0.0055 was assumed for ϵ_{spall} . The complete damage state is taken as the curvature which corresponds to $\epsilon_c = \epsilon_{cu}$, where ϵ_{cu} is the compression strain at which fracture of the transverse confining steel initiates. This strain limit is estimated conservatively using the energy balance approach proposed by Mander et al. (1988) as shown:

$$\varepsilon_{cu} = 0.004 + \frac{1.4\rho_s f_{yh} \varepsilon_{su}}{f'_{cc}} \quad \text{Eqn. 5.2}$$

where $\rho_s = 4A_h/D's$ is the volumetric ratio of confining steel, in which A_h is the cross sectional area of transverse reinforcement; D' is the diameter of the confined concrete core; s is the longitudinal spacing of hoops or spirals; f_{yh} is the yield strength of the transverse reinforcement; ε_{su} is the transverse steel strain at maximum tensile stress taken as 0.12 for this study and f'_{cc} is the compressive strength of the confined concrete. The extensive damage state is simply taken as being $1/2$ way between the moderate and complete curvature limit states following Ramanathan (2012), which estimated capacity limit states for modern bridge columns in California.

A probability distribution for the slight, moderate, extensive and complete curvature limits is found through Monte Carlo Simulation with a sample size of 10000. The steel strength, concrete strength, steel area and cover depth are all considered as random parameters with distributions described in the previous section. Additionally, concrete and steel strain limits are considered to be random, with their variability kept consistent with the concrete and steel strengths, following Lu et al. (2005). Finally, the axial load is considered to be uniformly distributed in a similar manner to the bridge mass, as described in the previous section. By a hypothesis test at the 5% significance level, it is found that the curvature capacities at each of the limit states follow a lognormal distribution. However, the associated COVs are much smaller than those assumed in previous studies that have estimated column curvature limit states (Nielson, 2005, Padgett, 2007, Ramanathan, 2012). This is due to the fact that the Monte Carlo simulation only accounts for uncertainty in a number of material and geometric parameters but neglects additional sources of variation such as uncertainty in the models assumed or construction quality, etc. Due to the lack of information to quantify the logarithmic standard deviation or dispersion, β_C , a value of 0.35 is adopted across all damage states following Ramanathan (2012). Ramanathan (2012) states that this value is a particularly good estimate for columns as it is consistent with the test results documented in the PEER column structural performance database.

Deterioration is accounted for by reducing the longitudinal and transverse steel cross sectional area and strength in the moment curvature analysis. The moment curvature analysis for the column at 0, 50, 75 and 100 years is shown in Figure 5.5. From the figure it can be seen that there is a loss in load carrying capacity with deterioration. The column capacities at the four limit states are then recalculated using the modified values for steel cross section and strength. The values of dispersion adopted for the pristine case are also used for the deteriorated columns.

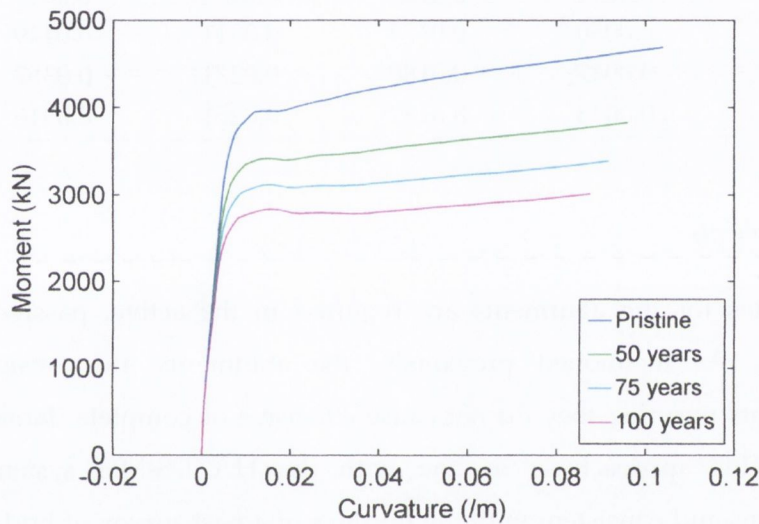


Figure 5.5 Moment curvature plots for the column at 0, 50, 75 and 100 years

The curvature limit states for each bridge length and age are shown in Table 5.2. From the table it can be seen that the complete damage curvature suffers the greatest reduction with age. This is mainly due to the fact that the ultimate concrete compression, ε_{cu} , decreases significantly with a loss of confinement. It is also noted that the moderate damage curvature, or the curvature at which concrete spalling occurs, increases slightly with deterioration. This result is consistent with work carried out by Kowalsky (2000), which found that greater curvatures are required to reach a given concrete strain, with a decrease in longitudinal steel ratio. In this case, deterioration leads to a reduction in reinforcing steel cross section which effectively results in a decrease in the longitudinal steel ratio.

Table 5.2 Curvature limit states for each bridge length and age

	Median Values, S_C (/m)				β_C
	Slight	Moderate	Extensive	Complete	
52 m Bridge					
0 years	0.0061	0.0172	0.0463	0.0753	0.35
50 years	0.0056	0.0186	0.0360	0.0533	0.35
75 years	0.0053	0.0194	0.0312	0.0430	0.35
100 years	0.0050	0.0202	0.0284	0.0367	0.35
69 m Bridge					
0 years	0.0055	0.0159	0.0398	0.0637	0.35
50 years	0.0050	0.0173	0.0311	0.0449	0.35
75 years	0.0047	0.0180	0.0271	0.0362	0.35
100 years	0.0045	0.0187	0.0251	0.0316	0.35

5.5.2 Abutments

Limit states for the abutments are required in the active, passive and transverse directions. As mentioned previously, the abutments are considered secondary components meaning they do not cause extensive or complete damage of the bridge system. This approach is in line with the HAZUS-MH system damage level descriptions and consistent with the findings of a past survey of bridge engineers and inspectors carried out by Padgett and DesRoches (2007). This survey found that abutment damage does not cause extensive or complete damage of the bridge system.

In the passive and transverse directions, the limit states are taken as a function of the maximum soil displacement capacity for a given abutment height following Choi (2002). Moderate damage is defined as the maximum soil displacement capacity, y_{max} , estimated as $0.05H$ and $0.1H$ for granular and cohesive soils respectively, where H is the abutment backwall height. Slight damage is taken $0.35y_{max}$. Limit states for active abutment deformations are adopted from Ramanathan (2012). Following Ramanathan (2012), the active abutment capacities are specified corresponding to the first yield and ultimate deformation of the underlying piles. The values for the abutment capacities at the slight and moderate damage states are shown in Table 5.3. The value for dispersion, β_C , is taken as 0.35 across all damage states following Ramanathan (2012).

Table 5.3 Abutment limit states

	Median Values, S_c (mm)				β_c
	Slight	Moderate	Extensive	Complete	
Active Abutment	25.4	100	N/A	N/A	0.35
Passive Abutment	76.1	217.5	N/A	N/A	0.35
Transverse Abutment	76.1	217.5	N/A	N/A	0.35

Abutment limit states are assumed to remain constant with time because the effect of deterioration on their displacement capacities is considered negligible as mentioned previously (Simon et al., 2010). However, further studies are recommended to explore the potential impact of abutment scour or backfill soil erosion on abutment capacity limits.

5.5.3 Deck

A preliminary investigation carried out by the authors showed that the deck was more vulnerable to failure in bending than in shear. Therefore, bending moment is adopted as the deck EDP. The ultimate flexural resistance of the prestressed girder and reinforced concrete slab composite section is estimated following the AASHTO LRFD bridge design specifications (AASHTO, 2012). It is acknowledged that if the results of this thesis show that the deck contributes significantly to the system fragility of the integral bridge, further work may be necessary to refine the deck limit states. However, as an initial investigation, it is thought that the use of bending moment as the deck EDP is acceptable. The ultimate positive moment capacity of the composite section with both bonded and unbonded prestressing strands, as shown in Figure 5.6 (a), is estimated by first assuming the neutral axis lies within the RC slab as shown in Figure 5.6 (b).

The depth of the neutral axis is estimated as:

$$c = \frac{A_{psb}f_{pu} + A_{psu}f_{pe}}{0.85f'_c\beta_1b + kA_{ps}\frac{f_{pu}}{d_{ps}}}$$

Eqn. 5.3

where

$$k = 2\left(1.04 - \frac{f_{py}}{f_{pu}}\right)$$

Eqn. 5.4

In Eqn. 5.3 - Eqn. 5.4, A_{psb} is the area of the bonded prestressing steel tendons; f_{pu} is the ultimate stress of the prestressing tendons; A_{psu} is the area of the unbonded prestressing steel tendons; f_{pe} is the effective stress of the tendons after losses; f'_c is the compressive strength of the concrete slab; β_1 is a stress block reduction factor; b is the width of the compression block; A_{ps} is the total area of prestressing steel tendons; d_{ps} is the depth to the centroid of the prestressing steel from the extreme compression fiber and f_{py} is the yield strength of the prestressing steel. If $c < h_f$, rectangular behaviour can be assumed and the nominal moment capacity can be estimated as:

$$M_n = A_{psb}f_{ps}(d_{ps} - a/2) + A_{psu}f_{pe}(d_{ps} - a/2) \quad \text{Eqn. 5.5}$$

where

$$f_{ps} = f_{pu} \left(1 - k \frac{c}{d_{ps}} \right) \quad \text{Eqn. 5.6}$$

and

$$a = \beta_1 c \quad \text{Eqn. 5.7}$$

In Eqn. 5.5 - Eqn. 5.7, f_{ps} is the stress in the prestressing steel at nominal flexural resistance and a is the depth of the equivalent stress block. Once the nominal moment of resistance, M_n , has been found, the factored flexural resistance is estimated as:

$$M_r = \phi M_n \quad \text{Eqn. 5.8}$$

where $\phi = 1.0$ is the resistance factor for flexure in prestressed concrete. If $c > h_f$, T section behaviour is assumed as shown in Figure 5.6 (c) and the neutral axis is estimated as:

$$c = \frac{A_{psb}f_{pu} + A_{psu}f_{pe} - 0.85f'_c h_f (b - 2b_w)}{0.85f'_c \beta_1 (2b_w) + k A_{ps} \frac{f_{pu}}{d_{ps}}} \quad \text{Eqn. 5.9}$$

The factored flexural resistance can be estimated using Eqn. 5.8 where the nominal moment of resistance is:

$$M_n = 0.85f_c 2b_w a \left(d_{ps} - \frac{a}{2} \right) + 0.85f_c (b - 2b_w) h_f \left(d_{ps} - \frac{h_f}{2} \right) \quad \text{Eqn. 5.10}$$

In the negative moment region above the support, the same approach is used however the bottom flange is in compression.

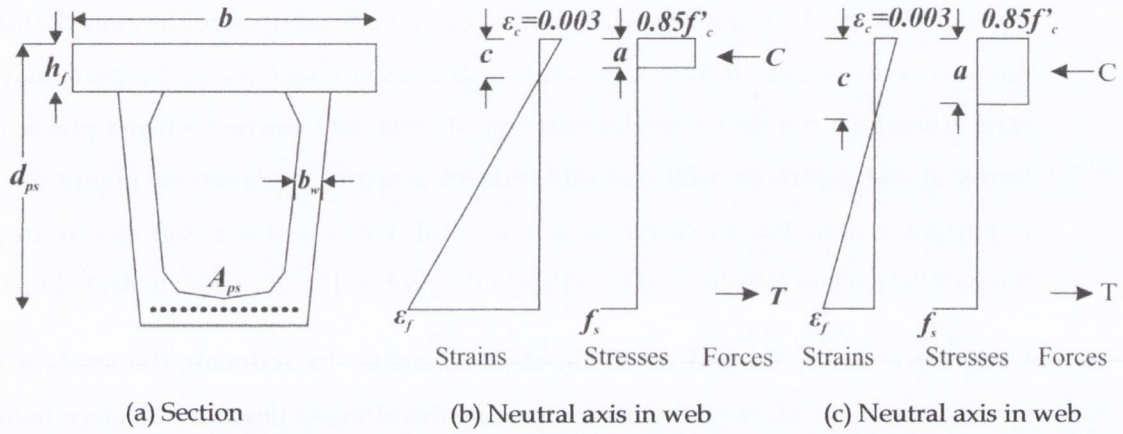


Figure 5.6 Stress and strain distribution of beam section in flexure (not actual location of prestressing steel)

It is assumed, following previous studies, that the bending moment capacity of the deck is lognormally distributed (Nowak et al., 1994). The above approach is used to estimate the median value for the ultimate flexural resistance and a dispersion, β_C , of 0.3 is adopted from Nowak et al. (1994) who developed probabilistic models for the resistance of prestressed concrete bridge girders. Since the deck is fully continuous at the abutments and over the supports, it is noted that the formation of a plastic hinge at one section does not lead to the collapse of the deck and therefore is considered extensive damage. Complete damage is defined as the formation of a collapse mechanism at one of the spans, as illustrated in Figure 5.7. The slight and moderate limit states are taken as 50% and 75% respectively of the ultimate flexural resistance. As mentioned previously, if the results of this thesis show that the deck contributes significantly to the system fragility of the integral bridge, further work may be necessary to refine the deck limit states. However, as a preliminary investigation into deck vulnerability and given the lack of available information on post earthquake

inspection of decks, these estimates for the slight and moderate damage states are deemed acceptable. The fragility of the deck is evaluated as a sub-system comprised of elements. For the slight, moderate and extensive damage states, if the capacity is exceeded in any element along the bridge deck, then the deck component limit state is reached (i.e. series system abstraction). For complete damage, in which the limit state is defined as the formation of a collapse mechanism at one of the spans, a parallel system is adopted. Parallel systems survive even if one component has failed and only fail to function when all the components of the system have reached a particular limit state. Therefore, the deck complete damage state is only reached when a plastic hinge forms at the centre as well as both ends of a span, as shown in Figure 5.7. It is important to note that the series assumption still stands for the bridge system fragility; the parallel system is only used to estimate the probability of complete deck damage.

Deterioration is accounted for in the deck capacities by reducing the steel area and strength in the RC deck slab when estimating the ultimate flexural resistance using the approach outlined above. This results in a reduction in the ultimate flexural resistance in the negative moment region above the support. As mentioned in Chapter 4, it is assumed that the steel in the prestressed girders remains unchanged. This is because the underside of the deck is not directly exposed to chlorides from de-icing salts due to the lack of deck joints for this bridge type. Additionally, the girders are fabricated under factory conditions and designed with greater durability, so are therefore less vulnerable to chloride ingress. Table 5.4 shows the positive and negative flexural limit states for each bridge length and age.

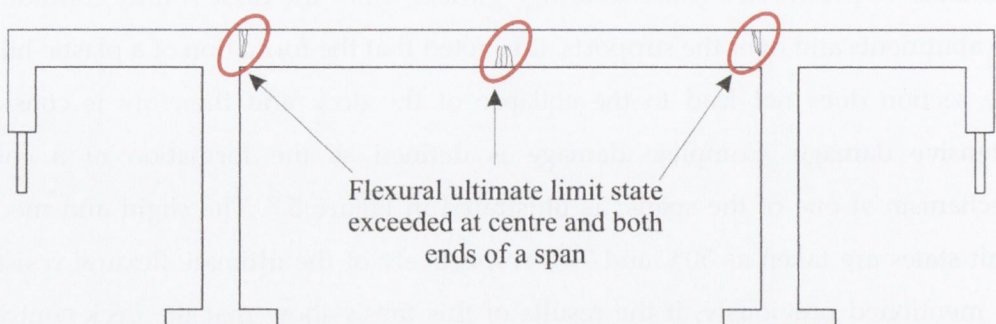


Figure 5.7 Formation of a collapse mechanism at the centre span

Table 5.4 Deck limit states for each bridge length and age

		Median Values (kN-m)			β_c
		Slight	Moderate	Extensive	
<u>52 m Bridge</u>					
Positive Flexural Resistance		7303.5	10955	14607	0.3
Negative Flexural Resistance	0 years	4684.3	7026.5	9368.6	0.3
	50 years	3309.6	4964.3	6619.1	0.3
	75 years	2615.1	3922.7	5230.3	0.3
	100 years	2111.3	3166.9	4222.5	0.3
<u>69 m Bridge</u>					
Positive Flexural Resistance		8935.0	13402.5	17870.0	0.3
Negative Flexural Resistance	0 years	9837.0	14755.5	19694.0	0.3
	50 years	7743.5	11615.3	15487.0	0.3
	75 years	6683.5	10025.3	13367.0	0.3
	100 years	5760.0	8640.0	11520.0	0.3

As a summary, Table 5.5 maps each of the component limit states considered to the relevant damage states. It can be seen from Table 5.5 that only the columns and deck contribute to the extensive and complete damage states.

Table 5.5 Component limit states

COMPONENT	LIMIT STATE			
	Slight	Moderate	Extensive	Complete
Columns (curvature)	Column yielding	Concrete Spalling	1/2 way between moderate and complete damage	Fracture of the transverse confinement
Deck (bending moment)	50% of the ultimate flexural resistance	75% of the ultimate flexural resistance	Ultimate flexural resistance	Formation of a collapse mechanism at one of the spans
Active Abutment (displacement in mm)	25.4	100.0	N/A	N/A
Passive Abutment (displacement in mm)	76.1	217.5	N/A	N/A
Transverse Abutment (displacement in mm)	76.1	217.5	N/A	N/A

5.6 Probabilistic Seismic Demand Models

5.6.1 Component Probabilistic Seismic Demand Models

Once the stochastic bridge models and probabilistic component capacity models are generated, the next step is to develop probabilistic seismic demand models for each component. In order to do this, 100 nonlinear time history analyses are carried out using the finite element model and ground motion pairs at 0, 50, 75 and 100 years. The maximum responses of the critical components are recorded for each ground motion. As mentioned previously, the component demands considered as a part of this study are the deck bending moment, the abutment deformations and the column curvature. The column demand is taken as:

$$\phi = \sqrt{\phi_{longitudinal}^2 + \phi_{transverse}^2} \quad \text{Eqn. 5.11}$$

where $\phi_{longitudinal}$ and $\phi_{transverse}$ are the curvature demands in the longitudinal and transverse directions respectively.

Cornell et al. (2002) proposed that the median seismic demand can be predicted as a function of the ground motion intensity via the following power law model:

$$S_D(t) = a(t)IM^{b(t)} \quad \text{Eqn. 5.12}$$

where $S_D(t)$ is the time dependent median seismic demand; IM is the ground motion intensity and $a(t)$ and $b(t)$ are unknown coefficients. By transforming Eqn. 5.12 into lognormal space as shown in Eqn. 5.13, the parameters $\ln(a(t))$ and $b(t)$ can be found through linear regression analysis.

$$\ln(S_D(t)) = \ln(a(t)) + b(t)\ln(IM) \quad \text{Eqn. 5.13}$$

For the case of the columns, this approach is adapted by developing models for the demand to capacity ratio, $S_D(t)/S_C(t)$, as a function of the ground motion intensity. This

approach is adopted in order to account for potential correlation between the demand and capacity when degradation of both the capacity and demand are simultaneously

accounted for. In the proposed framework, the peak column demands of the aged bridge are divided by the capacity realizations generated using the same set of deteriorated material strength and reinforcing steel area realizations used to generate the bridge samples at a given time t . Eqn. 5.14 shows the modified linear regression model.

$$\ln\left(\frac{S_D(t)}{S_C(t)}\right) = \ln(a(t)) + b(t)\ln(IM) \quad \text{Eqn. 5.14}$$

From the regression Eqn. 5.14, the median of the demand to capacity ratio is obtained as well as the variation about the median. Following previous work which has shown that the demand can be probabilistically modelled with a lognormal distribution (Shinozuka et al., 2000a, Mackie and Stojadinovic, 2005b), it is assumed the demand to capacity ratio can be modelled with a lognormal distribution. Therefore, in the transformed space, the demand to capacity ratio is normally distributed. A time dependent estimate of the dispersion, $\beta(t)_{D/C|IM}$, conditional on the intensity level, can be estimated as the standard deviation of the residuals as follows:

$$\beta(t)_{D/C|IM} \cong \sqrt{\frac{\sum_{i=1}^N (\ln(d/c_i) - \ln(a(t)IM^{b(t)}))^2}{N-2}} \quad \text{Eqn. 5.15}$$

In the above equation, d/c_i is the demand to capacity ratio for the i^{th} ground motion; $a(t)IM^{b(t)}$ is the estimated median demand to capacity ratio and N is the number of simulations carried out. Figure 5.8 shows a typical PSDM with the parameters described in this section illustrated. Although Figure 5.8 shows a linear regression, this thesis uses a combination of both linear and bilinear regression, depending on the component considered.

From Figure 5.8, it can be seen that an appropriate intensity measure needs to be adopted. For this study, peak ground acceleration (PGA) is chosen as the intensity measure for developing the PSDMs. PGA is chosen based on findings of past studies that have proven its superior performance on the basis of such measures as practicality, efficiency, sufficiency, proficiency and hazard computability for probabilistic seismic

demand modelling of typical bridge classes (Padgett et al., 2008). Furthermore, results of this thesis demonstrate the consistent predictive capability of PGA for the case study bridge herein, as indicated by the goodness-of-fit estimates. It is noted however that the framework presented herein is flexible to incorporate other intensity measures such as spectral acceleration, spectral velocity or spectral displacement.

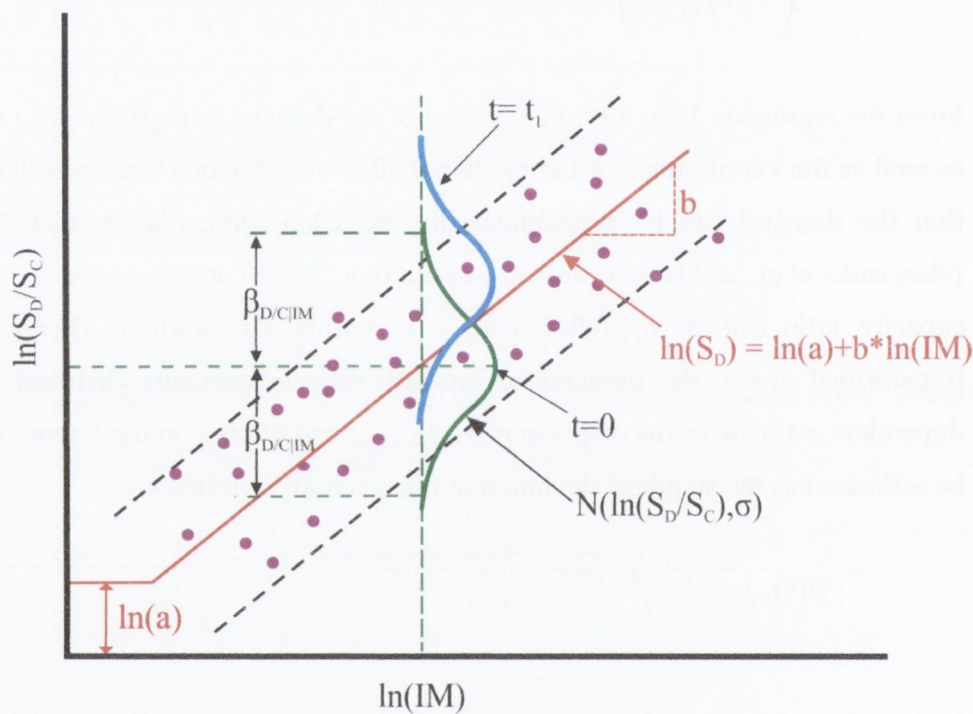


Figure 5.8 Illustration of a typical PSDM

For illustrative purposes, a selection of the PSDMs are shown graphically in Figure 5.9 -Figure 5.12. In Figure 5.9 - Figure 5.12, the points represent the maximum demand experienced by the component for a given ground motion. The blue circles represent the demands on the pristine bridge and the red triangles represent the demands on the 100 year old bridge. From Figure 5.9, it can be seen that the abutment demands in the transverse direction are predicted with reasonable accuracy using linear regression in the log transformed space. From the figure it can also be seen that corrosion has a negligible effect on the abutment transverse demand model. The passive and active

demand models are not shown but corrosion also has a negligible effect on these models. For the column and deck demands, a bilinear regression is found to be a good fit as shown in Figure 5.10 -Figure 5.12, reflecting typical stress-strain nonlinear behavior of the materials. Further discussion on the use of bilinear demand models can be found in Freddi (2012) and Ramamoorthy (2006). Figure 5.10 illustrates the increase in the column curvature demand to complete curvature capacity ratio after 100 years. From the values for σ , it can be seen that deterioration does not have a significant impact on the dispersion about the demand estimate. From Figure 5.11, it can be seen that the maximum negative moment demand in the deck, which occurs over the intermediate support, decreases with corrosion. However, maximum positive moment demand in the deck, which occurs at centre span, increases with corrosion as shown in Figure 5.12. Similar findings are revealed for the various geometric configurations considered. Appendix A presents the component PSDMs in tabular form for all the bridge ages and geometric configurations considered.

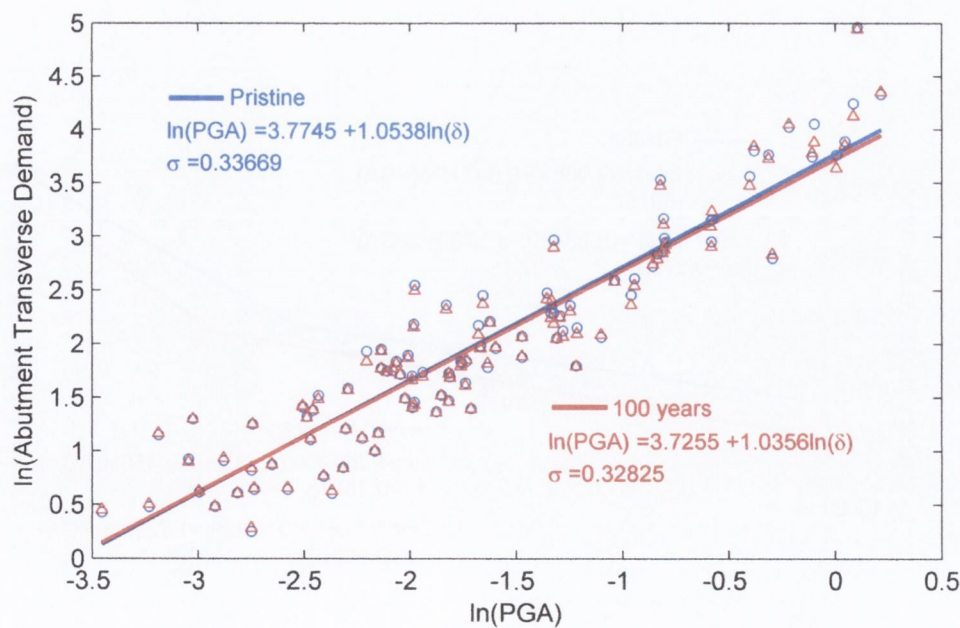


Figure 5.9 PSDM for the abutment transverse deformation at 0 and 100 years

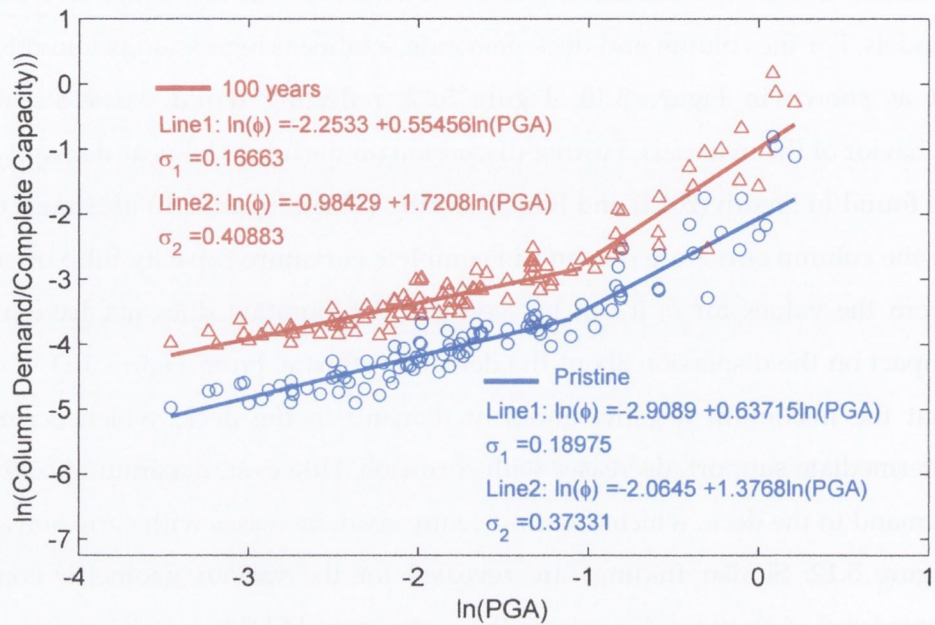


Figure 5.10 Probabilistic seismic demand to complete capacity model for the column curvature at 0 and 100 years

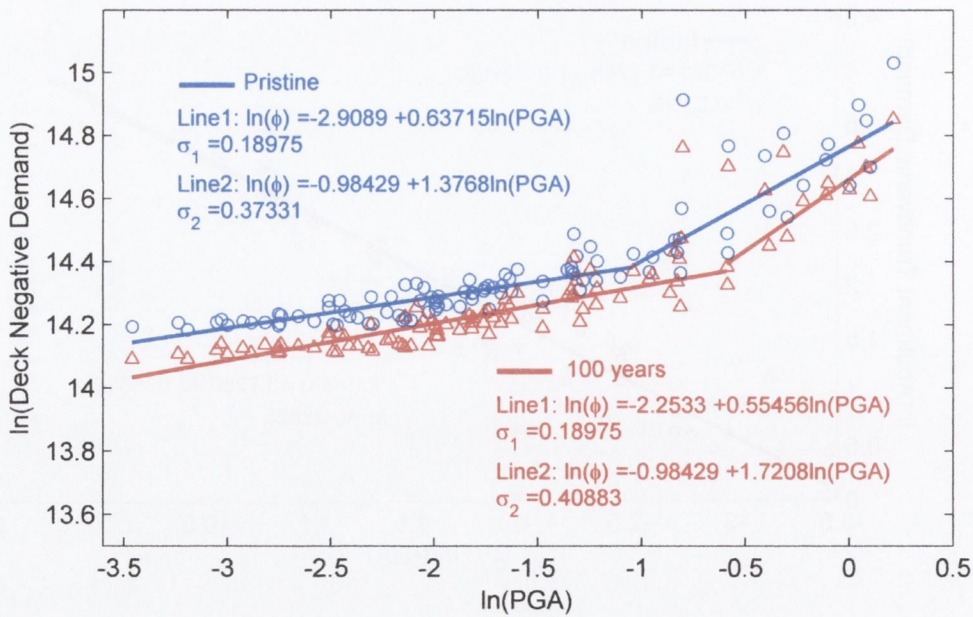


Figure 5.11 PSDM for the deck bending moment over the support at 0 and 100 years

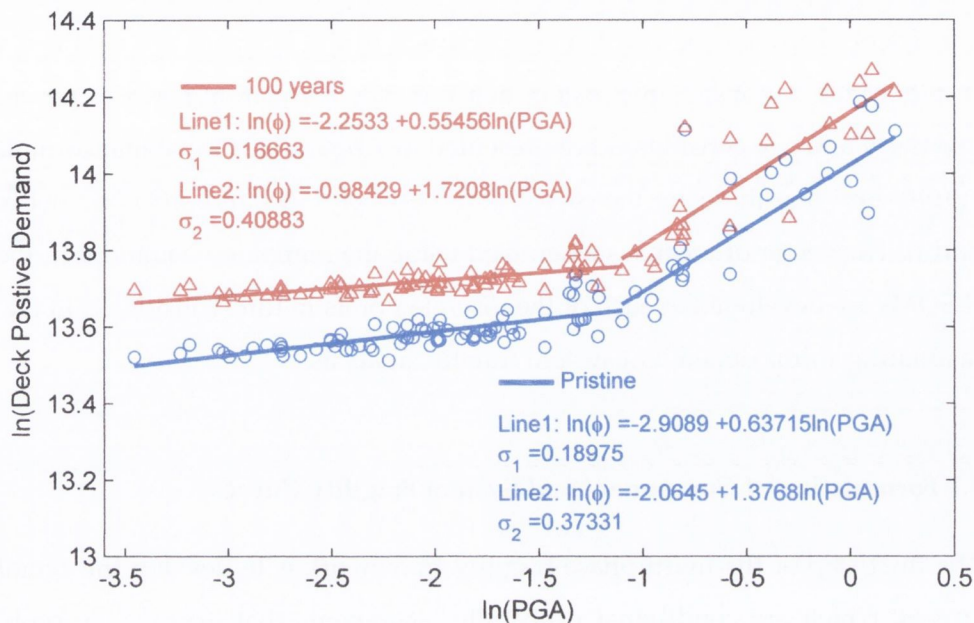


Figure 5.12 PSDM for the deck bending moment at centre span at 0 and 100 years

5.6.2 Joint Probabilistic Seismic Demand Models

Although a component fragility analysis is useful to highlight vulnerable components, a system wide fragility analysis is needed for loss estimation packages. For this study, the development of system fragility curves is facilitated by generating joint probabilistic seismic demand models (JPSDM), accounting for correlation between component demands, following Nielson (2005b). The JPSDM describes the demand placed on a number of components within the bridge system and takes the form of a joint probability distribution for a given earthquake intensity level. Given that the individual component PSDMs are assumed to be lognormally distributed, the JPSDM can be assembled in a straight forward manner. If $\underline{X} = (X_1, X_2, \dots, X_n)$, is a vector of demands placed on n bridge components whose marginals are lognormally distributed, then the vector, $\underline{Y} = \ln(\underline{X})$, represents the vector of normally distributed component demands in the transformed space. If a joint probability distribution is jointly normal then the only information needed to describe it is the vector of means, $\mu_{\underline{Y}}$, and the covariance matrix, $\sigma_{\underline{Y}}$. For this reason, the JPSDMs are developed in the

logspace for this study. In order to assemble the covariance matrix, correlation coefficients between the component demands in the logspace are obtained using the results of the nonlinear time history analysis. The correlation matrices for each bridge geometry and age considered are presented in Appendix A in tabular form. It must be emphasised that these are the correlations between $\ln(x_i)$ and $\ln(x_j)$ and not between x_i and x_j . The vector of means is assembled using the regression equations of the PSDMs. JPSDMs are developed at each of the discrete points in time considered in the analysis, facilitating a time dependent system fragility analysis.

5.7 Formulation of Component and System Fragility Curves

The final step of the multi phase fragility framework is to develop the actual fragility curves, which are conditional probability statements that express the probability of reaching or exceeding a defined limit state as a function of the ground motion intensity level. These curves provide a quantification of the probability of a given level of damage occurring, which is vital information for a seismic risk assessment. The following sections describe how a component and system fragility analysis is carried out.

5.7.1 Component Fragility Curves

In the previous sections, separate time dependent models for the seismic demand and capacity were developed for the deck and the abutments. These are assumed to be lognormally distributed and therefore the component fragilities can be derived using a closed form solution as follows:

$$P\left(\frac{Demand(t)}{Capacity(t)} > 1.0 | IM\right) = \Phi\left(\frac{\ln\left(\frac{S_D(t)}{S_C(t)}\right)}{\sqrt{\beta_{D|IM}^2(t) + \beta_c^2(t)}}\right) \quad \text{Eqn. 5.16}$$

where $\Phi(\cdot)$ is the standard normal cumulative distribution function; $S_D(t)$ and $\beta_{D|IM}^2(t)$ are the time dependent median and dispersion of the lognormally distributed seismic demand and $S_C(t)$ and $\beta_c^2(t)$ are the time dependent median and dispersion of

the lognormally distributed seismic capacity, which are defined based on the limit state under consideration. By substituting the formulation for the median seismic demand, $S_D(t)$, described in the PSDM formulation, Eqn. 5.16 becomes:

$$P\left(\frac{Demand(t)}{Capacity(t)} > 1.0 | IM\right) = \Phi\left(\frac{\ln(IM) - \frac{\ln(S_c) - \ln(a)}{b}}{\frac{\sqrt{\beta_{D|IM}^2(t) + \beta_c^2(t)}}{b}}\right) \quad \text{Eqn. 5.17}$$

This component fragility is lognormally distributed with a median, λ_{comp} , and dispersion, ζ_{comp} , leading to the following formulation:

$$P\left(\frac{Demand(t)}{Capacity(t)} > 1.0 | IM\right) = \Phi\left(\frac{\ln(IM) - \ln(\lambda_{comp})}{\zeta_{comp}}\right) \quad \text{Eqn. 5.18}$$

For the case of the columns where the demand to capacity ratio, $S_D(t)/S_c(t)$, is predicted by the PSDM, Eqn. 5.19 is found by substituting the formulation for the median seismic demand to capacity ratio into Eqn. 5.16:

$$P\left(\frac{Demand(t)}{Capacity(t)} > 1.0 | IM\right) = \Phi\left(\frac{\ln(IM) - \left(-\frac{\ln(a)}{b}\right)}{\frac{\sqrt{\beta_{D|IM}^2(t) + \beta_c^2(t)}}{b}}\right) \quad \text{Eqn. 5.19}$$

In this formulation of the lognormally distributed component fragility, $\lambda_{comp} = -\frac{\ln(a)}{b}$

and $\zeta_{comp} = \frac{\sqrt{\beta_{D|IM}^2(t) + \beta_c^2(t)}}{b}$. The component fragility curves are presented and discussed in the following chapter.

As mentioned previously, once the component fragilities are obtained, these need to be integrated to formulate a fragility curve for the entire bridge system. The system fragility curves are developed by comparing realizations of the component demands using the previously developed JPSDM with the component capacities via Monte Carlo Simulation. 10,000 samples are drawn from both the demand and capacity models and the probability of failure for a given earthquake intensity is evaluated. The process is

repeated at increasing increments of earthquake intensity to generate a fragility curve. It is noted that although correlation is not considered across the component capacities, 100% correlation is assumed across damage states for a given component to ensure the samples are in rank order ($C_{\text{slight}} < C_{\text{moderate}} < C_{\text{extensive}} < C_{\text{complete}}$). The intersection of a time dependent bi-variate joint probability density function with its failure domain is shown in Figure 5.13. An example of a system failure realization is shown. From the figure it can be seen that the demand model and failure domain change with time. As mentioned previously, a series system is assumed in this thesis, which means that failure of one component leads to failure of the system. This is possible since the component capacities have been combined in such a way that they have similar operational consequences. As mentioned in Section 2.5, the deck and columns contribute to extensive and complete system damage, whereas the abutments contribute only to slight and moderate system damage.

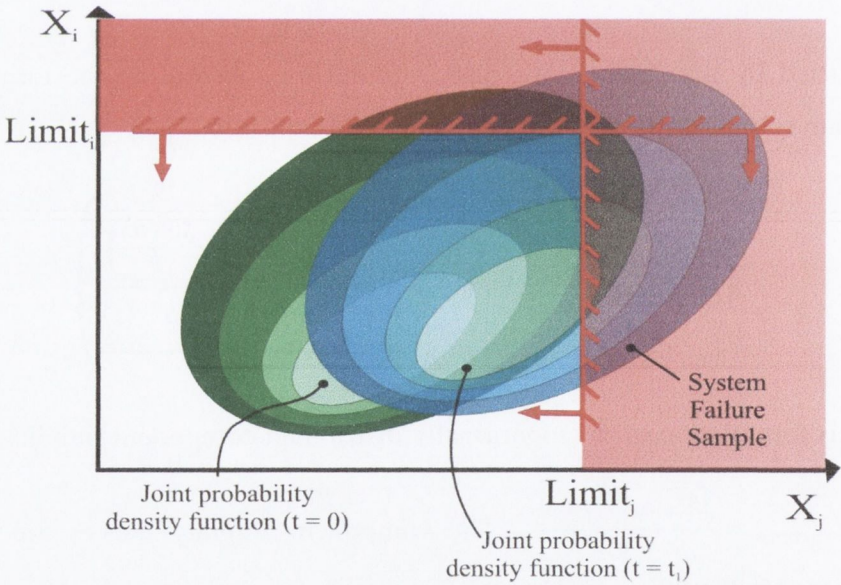


Figure 5.13 Intersection of a time dependent bivariate joint PDF and its failure domain

5.8 Conclusion

This chapter outlined the multi phase framework for the development of time dependent component and system fragility curves. The steps involved in the

development of the component and system fragility curves were described in detail including: the ground motion suite; the ageing stochastic finite element bridge models capturing uncertainty; the time dependent component capacities; the time dependent component and system probabilistic seismic demand models and finally the formulation of the time dependent component and system fragility curves. The seismic demand was predicted as a function of ground motion intensity by subjecting a group of 100 nominally identical yet statistically significant bridges to a suite of 100 ground motions representative of seismicity in the California region and recording the peak component demands. These demand models were compared with component capacity models at various points in time along the service life of the structure. The resulting component and system fragility curves can be seen in the following chapter.

CHAPTER 6 - TIME DEPENDENT COMPONENT AND SYSTEM

FRAGILITY CURVES

This chapter presents the component and system fragility curves for each geometric configuration of the three span integral bridge and the various levels of deterioration considered. The three main sections in this chapter examine the effect of corrosion, foundation settlement and variation in column height and bridge length on the component and system fragility curves. Previous studies that have developed fragility curves for highway bridges (Nielson, 2005, Padgett, 2007, Ramanathan, 2012) have typically fitted a lognormal distribution to the system fragility curves developed via Monte Carlo Simulation. Therefore, the fragility curves can be fully described by the values for the median, λ_{sys} , and dispersion, ζ_{sys} of the lognormal distribution. For this thesis however, the introduction of the deck as a component as well as bilinear demand models for the columns and the deck, results in a departure from the lognormal distribution for the system fragility curves. Although median values of the fragility curves cannot be compared for this thesis, point estimates of the failure probabilities (or damage state exceedance probabilities) are compared. In order to facilitate this, seismic hazard curves available from the United States Geological Survey (USGS) website were consulted. These hazard curves give the annual frequency of exceeding a level of peak ground acceleration (PGA), for a given geographic location and soil type. For this thesis, it is assumed that the bridge is located in the city of Los Angeles, California with a soil type C. Soil type C consists of sands, sandstones, mudstones and limestones and has a shear wave velocity, V_s , of between 350 m/s and 750 m/s. The associated hazard curve can be seen in Figure 6.1, which shows the annual frequency of exceedence for a given PGA. From the figure, the PGAs corresponding to a 10% and 2% probability of exceedence (PE) in 50 years are found to be 0.5g and 0.97g respectively (USGS, 2011).

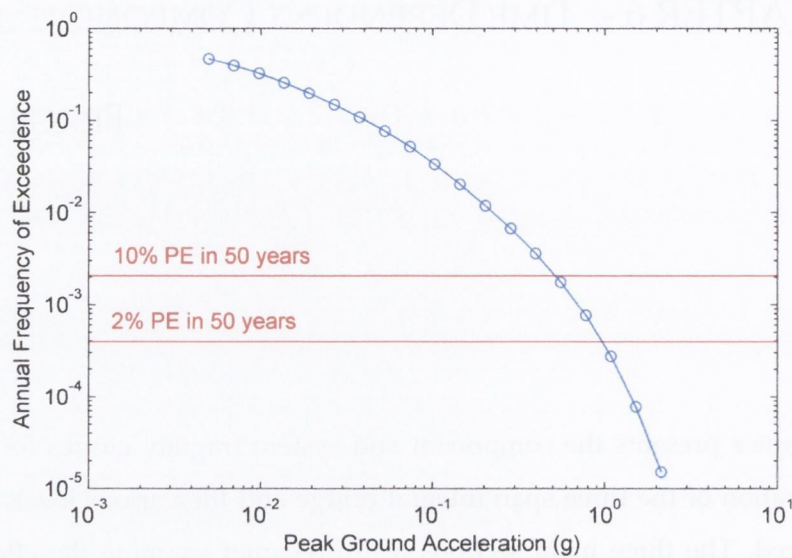


Figure 6.1 Seismic hazard curve for Los Angeles with soil type C (USGS, 2011)

6.1 Effect of Geometric Variation on Seismic Fragility Estimates

In order to investigate the effect of geometric variation on the seismic performance of this integral bridge type, two different bridge lengths and three column heights are considered, resulting in six geometric bridge samples. Chapter 3 outlined the approach adopted to generate realistic bridge samples of varying length and column height. The two bridge lengths considered are 52 m and 69 m and the three column heights are taken as 6.9 m, 9.5 m and 12 m. Figure 6.2 shows the column fragility curves for the integral bridge with varying column height at the slight and moderate damage states. The extensive and complete damage fragilities for these bridges are found to be negligible over the range of PGAs considered and therefore have not been included. From the figure it can be seen that the column fragility decreases with column height. For example, the probability of the columns exceeding the slight damage state for the 2% in 50 year earthquake decreases from 82% to 28%, with a 5.1 m increase in column height. This is the expected result since it is known that for a given level of displacement at the column top, the curvature experienced by a column decreases with column height. Although the maximum displacement at the column top under seismic loading increases with an increase in column height, this increase is not significant enough to result in an increase in curvature. Furthermore, this finding is consistent

with other studies on the seismic fragility of bridges with varying column height considering a fixed connection between the deck and the columns (Alipour et al., 2011, Reza, 2012).

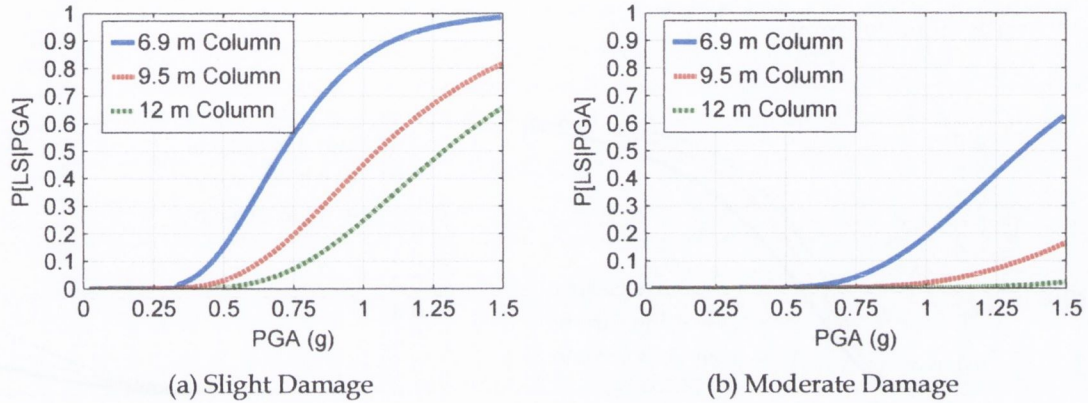


Figure 6.2 Column fragility curves with varying column height

Figure 6.3 shows the abutment fragility curves in the active direction for the three column heights at the slight and moderate damage states. From the figure it can be seen that the abutments become more vulnerable to seismic loading with an increase in column height, although the change in fragility is less significant than the change in fragility experienced by the columns. The probability of the abutments exceeding the slight damage state in the active direction for the 2% in 50 year earthquake increases from 81% to 91%, with a 5.1 m increase in column height. The abutment fragility curves in the passive and transverse directions have not been included but follow the same trend of increased vulnerability with an increase in column height. Figure 6.4 shows the system fragility curves at the slight and moderate damage states for the three column heights. From Figure 6.4 (a) it can be seen that at the slight damage state the system fragility experiences a slight increase with an increase in column height. The probability of exceeding slight damage for the 2% in 50 year earthquake increases from 40% for the 6.9 m column to 46% for the 12 m column. This result is due to the fact that the abutments contribute significantly to the system fragility at the slight damage state and as previously mentioned, the abutment fragility increases with an increase in column height. At the moderate damage state, shown in Figure 6.4 (b), it is noted that the system fragility decreases when the column height increases from 6.9 m

to 9.5 m. However, when the column height is further increased to 12 m, the system fragility increases, albeit a very slight increase of 1.5% for the 2% in 50 year earthquake. This trend occurs because the column fragility is lower for bridges with taller columns and therefore, the abutments tend to contribute more to the system fragility for bridges with taller columns.

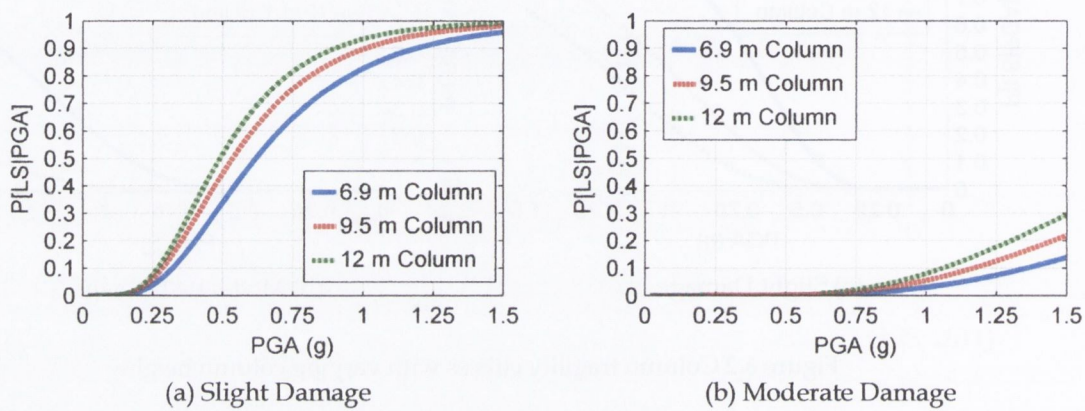


Figure 6.3 Abutment active fragility curves for varying column height

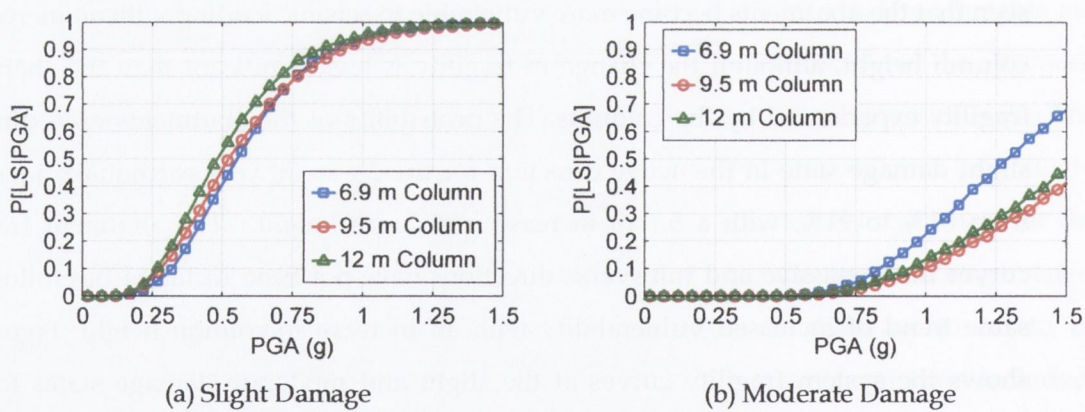


Figure 6.4 System fragility curves for varying column height

As discussed in Section 3.1, additional piles may be required at the abutments and foundations of the 69 m bridge to accommodate the increase in axial load due an increase in bridge length. In order to investigate how sensitive the fragility of the bridge is to an increase in the number of abutment and foundation piles, a further fragility analysis is performed including an increase on the order of 20% in the effective

pile stiffness at the abutments and foundations. Figure 6.5 shows the 69 m bridge system fragility curves for the original abutments and foundations as well as for the abutments and foundations with an increase of 20% in the effective pile stiffness.

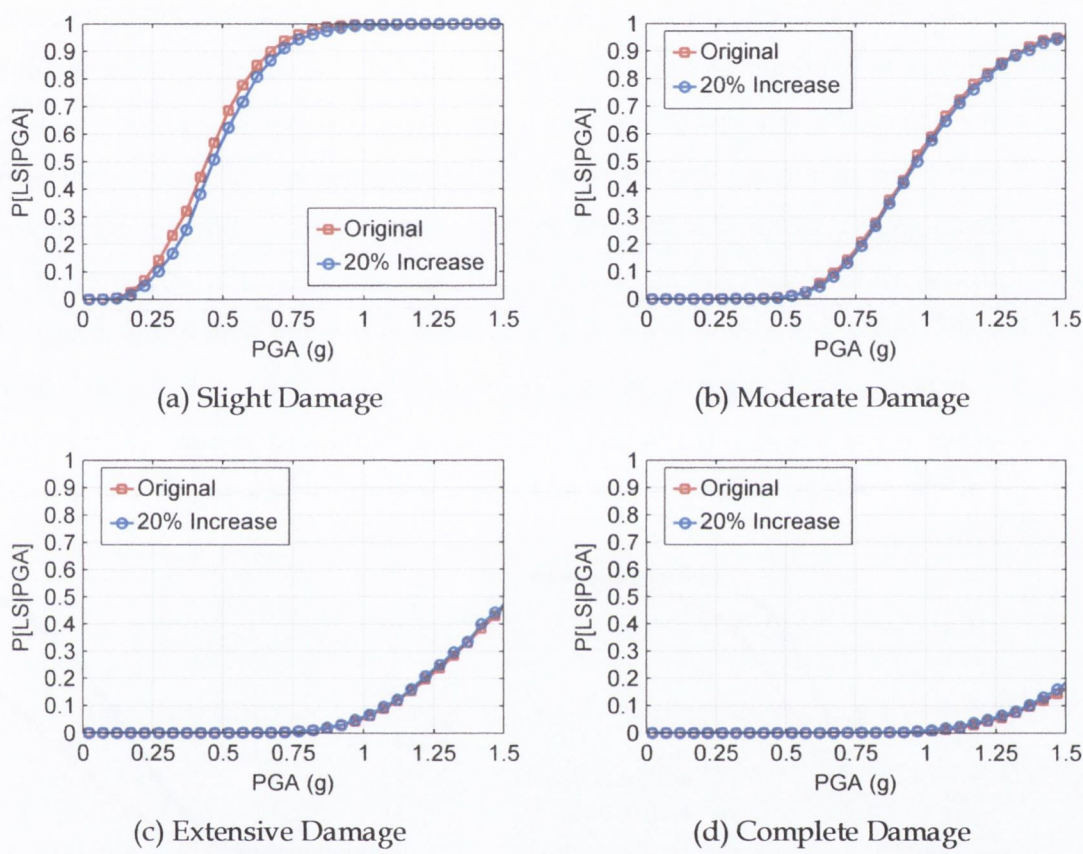


Figure 6.5 System fragility curves for the 69 m bridge with a 20% increase in the effective pile stiffness at the abutments and foundations

From the figure it can be seen that there is only a very slight difference between the fragility of the two bridge configurations. From Figure 6.5 (a), it can be seen that the probability of the bridge exceeding slight damage decreases by <5% for the 10% in 50 year earthquake with an increase of 20% in the effective pile stiffness. The difference in fragility between the two bridge configurations is even less significant at the other damage states. It is thought that the increase in the number of piles with bridge length would not be significantly greater than the order of 20%, given the increase in bridge length. Additionally, given the width and configuration of the foundations and abutments, they are unable to accommodate a further increase in pile numbers. From

these results, it is deemed acceptable to use the foundations and abutments of the 52 m bridge for an increase in bridge length.

Figure 6.6 shows the system fragility curves for the 52 m and 69 m bridge lengths, for all damage states. From the figure it can be seen that the system fragility increases with an increase in bridge length. For the 2% in 50 year earthquake, the probability of exceeding moderate damage increases from 19% to 44% with a 17 m increase in bridge length. This result is expected since the larger mass of the longer superstructure induces greater seismic demands on the bridge components. Although not shown here, both the columns and the abutments experience an increase in fragility with bridge length. The deck however experiences a decrease in fragility with bridge length, due to the increased deck capacity needed to accommodate the extra demand from live loading with length. The fragility estimates for all the bridge components and geometric configurations can be found in tabular form in Appendix B.

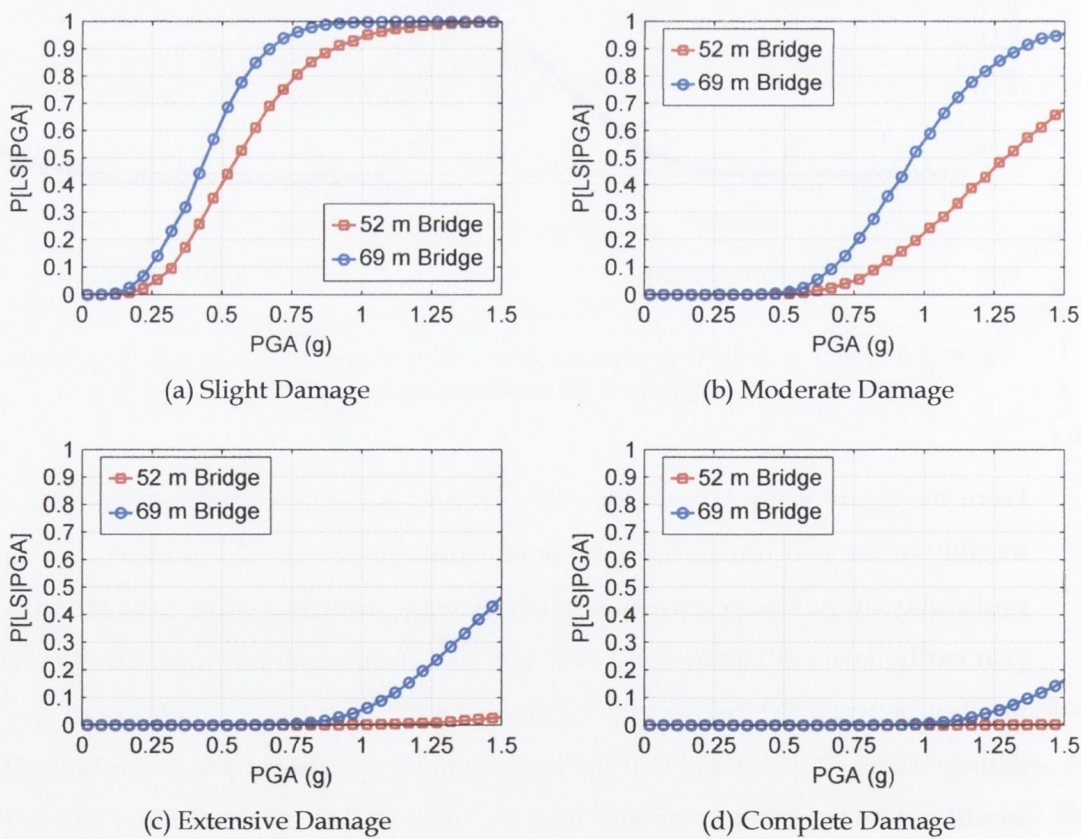


Figure 6.6 System fragility curves for varying bridge length

6.2 Impact of Corrosion on Seismic Fragility Estimates

Firstly the impact of corrosion on the seismic fragility of the various bridge components is examined. Figure 6.7 shows the column fragility curves for all damage states and all levels of corrosion. From the figure it can be seen that the fragility increases with bridge age and level of deterioration. For example, the probability of the columns exceeding slight damage, for the 10% in 50 year earthquake, increases from 15% to 36% after 100 years. At the moderate damage state however, the fragility actually decreases very slightly after the first 50 years but experiences an overall increase after 100 years. In order to understand this result, the moderate curvature capacity definition is examined. Recalling Chapter 5, it was seen that the moderate curvature limit, defined as the curvature at which concrete cover spalling occurs, actually increases with a reduction of steel cross sectional area. This trend is consistent with previous studies that have estimated column curvature limit states (Kowalsky, 2000). However, these results must be adopted with caution since cover spalling may have initiated already due to the buildup of corrosion products on the surface of the steel reinforcement. Additionally, this result suggests that a reduction in the concrete cover strength due to corrosion may need to be accounted for, which has been neglected in the current corrosion model. From Figure 6.7, it can be seen that corrosion has the greatest impact at the extensive and complete damage states. This is due to the fact that the complete curvature limit state, defined as the curvature at which fracture of the transverse confining steel initiates, decreases significantly with corrosion of the confining steel. Figure 6.7 (a) also shows that the column fragility curves at the slight damage state exhibit a lack of smoothness, reflecting the impact of the bilinear models for column demand.

Figure 6.8 -Figure 6.10 show the abutment fragility curves at the slight and moderate damage states in the active, passive and transverse directions. From the figures, it can be seen that corrosion has very little effect on the abutment fragility. The abutments actually experience an overall decrease in fragility with corrosion; however this decrease is very slight.

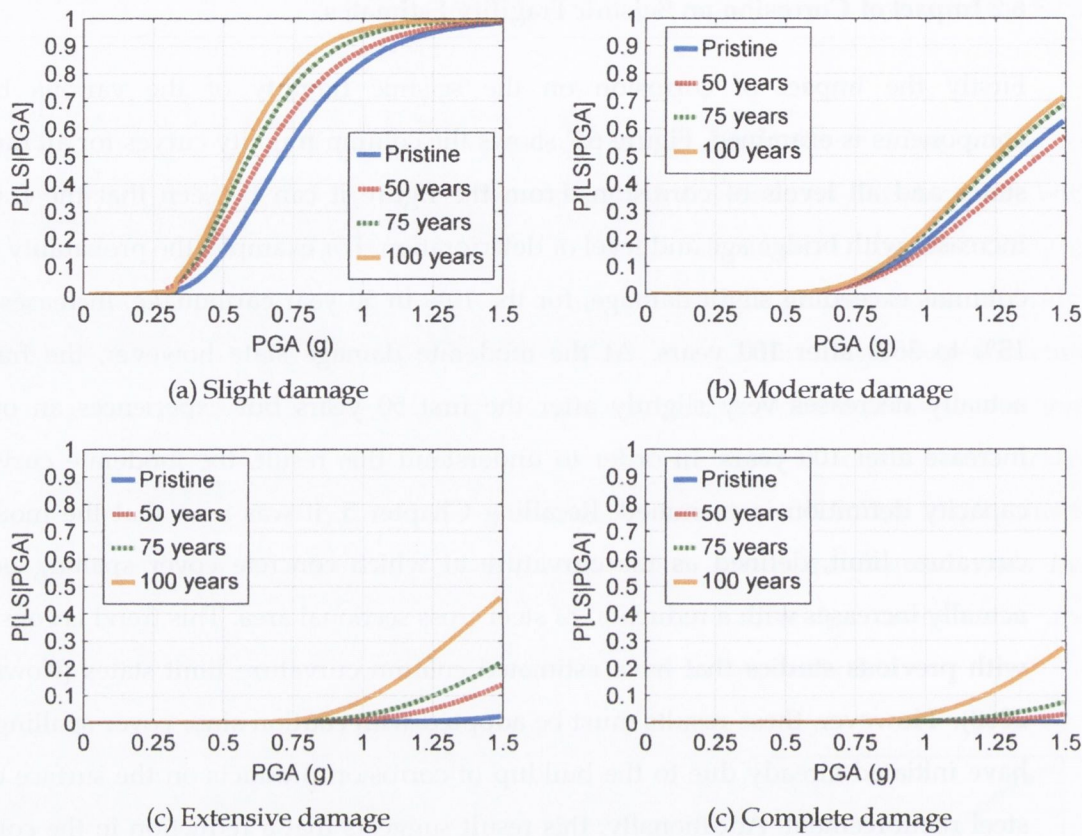


Figure 6.7 Column fragility curves at 0, 50, 75 and 100 years

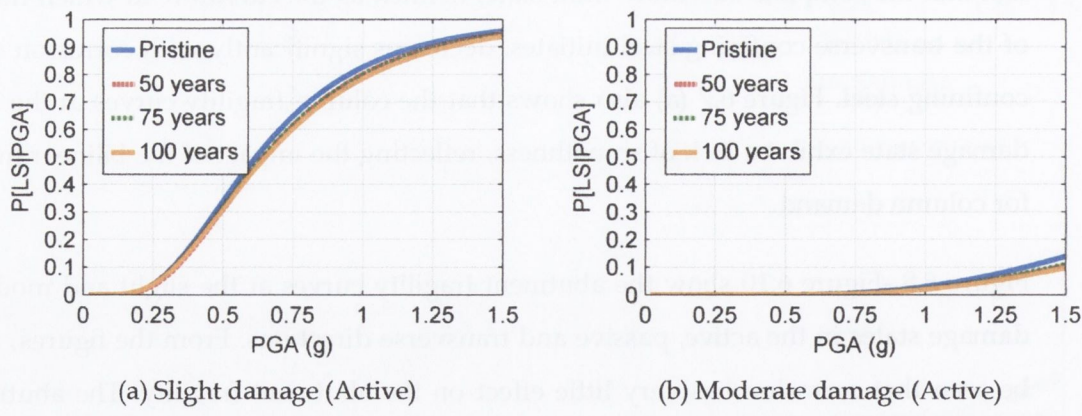


Figure 6.8 Active abutment fragility curves at 0, 50, 75 and 100 years

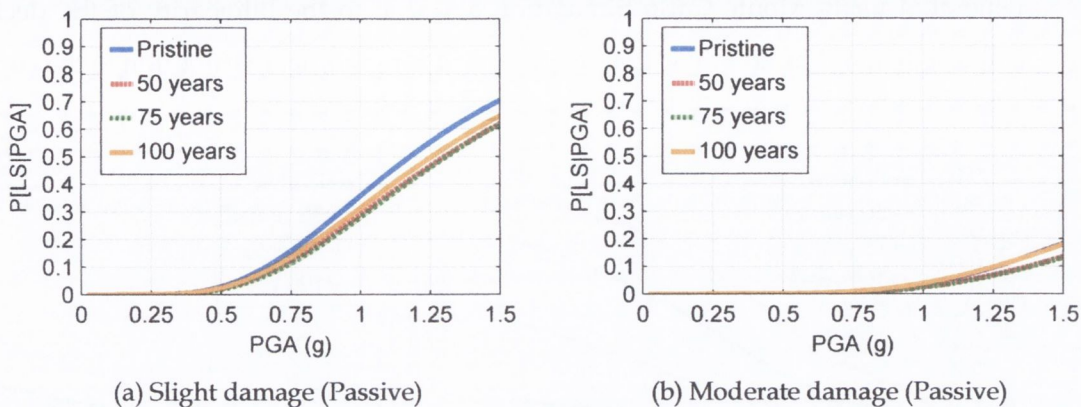


Figure 6.9 Passive abutment fragility curves at 0, 50, 75 and 100 years

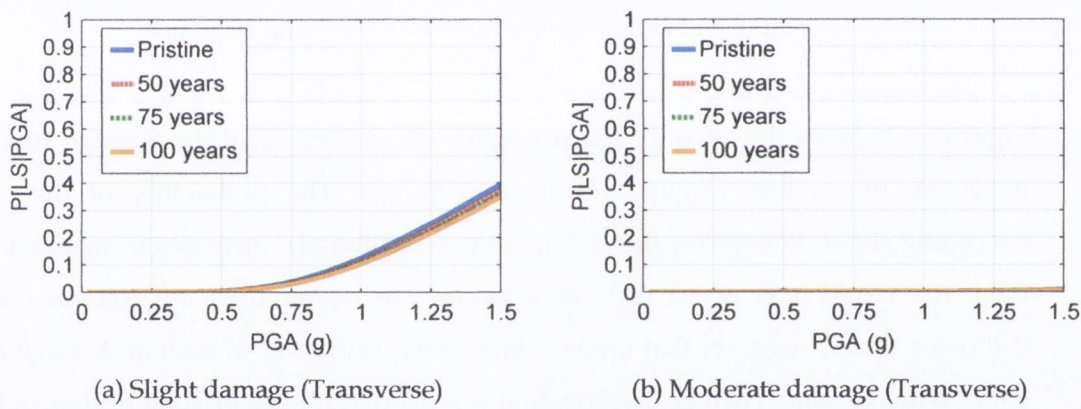


Figure 6.10 Transverse abutment fragility curves at 0, 50, 75 and 100 years

Figure 6.11 shows the slight and moderate damage fragility curves for the ageing deck. From the figure it can be seen that the deck suffers the greatest increase in fragility with corrosion of all the components. This is due to the significant reduction in the ultimate capacity of the deck with corrosion. From Figure 6.11 (a), it can be seen that the probability of the deck exceeding slight damage for the 10% in 50 year earthquake increases from < 1% to 29% after 100 years. It is also noted that the fragility curves in Figure 6.11 exhibit a lack of smoothness due to the bilinearity of the deck demand model. Although the deck experiences a large increase in vulnerability with age, it does not tend to contribute significantly to the system damage beyond the moderate damage state. As mentioned previously, the deck and system fragility curves are

generated using Monte Carlo Simulation and due to the bilinearity of the deck and column demand models, it is not possible to fit a lognormal distribution to the curves.

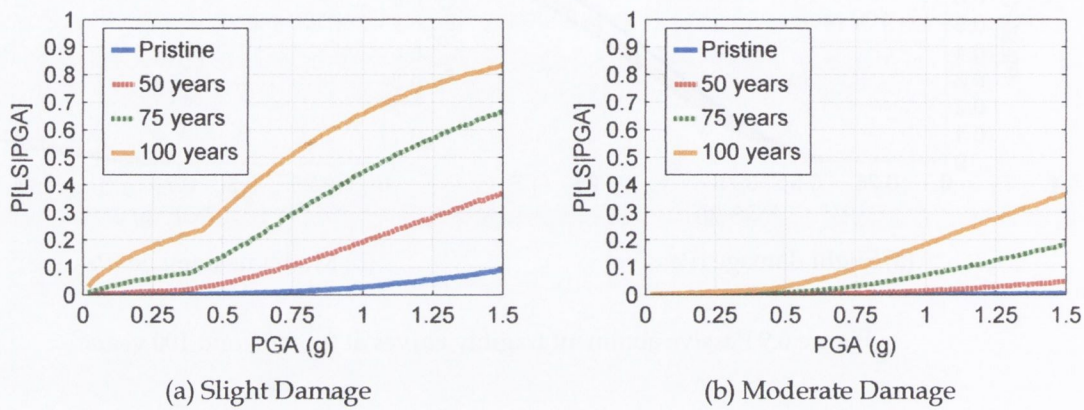


Figure 6.11 Deck fragility curves at 0, 50, 75 and 100 years

Figure 6.12 shows the ageing system fragility curves across all the damage states. As expected, the system fragility increases with age. The probability of the bridge exceeding slight damage for the 10% in 50 year earthquake increases from 40% to 62% after 100 years. It is noted that these values are higher than the column fragility estimates which suggests that other components contribute to system damage at the slight damage state. Further investigation reveals that for the pristine bridge, only the abutments and columns contribute to slight system damage, whereas after 75 years, the deck begins to contribute to slight damage of the system. As with the columns, the moderate system fragility experiences a very slight decrease after 50 years but experiences an overall increase after 100 years. Comparing the system fragility estimates to the column fragility estimates at the moderate damage state for the pristine bridge, it can be seen that the columns dominate the system fragility. After 100 years however, the probability of reaching moderate system damage is 35%, compared to a 21% probability of reaching moderate column damage for the 2% in 50 year earthquake. This difference is due to the contribution of the corroded deck to the system fragility. At the extensive and complete damage states however, the deck has a negligible effect on the system fragility for the range of PGA values considered, even after 100 years of corrosion. The next section will investigate whether this holds for the case of coupled corrosion and foundation settlement of one of the intermediate piers.

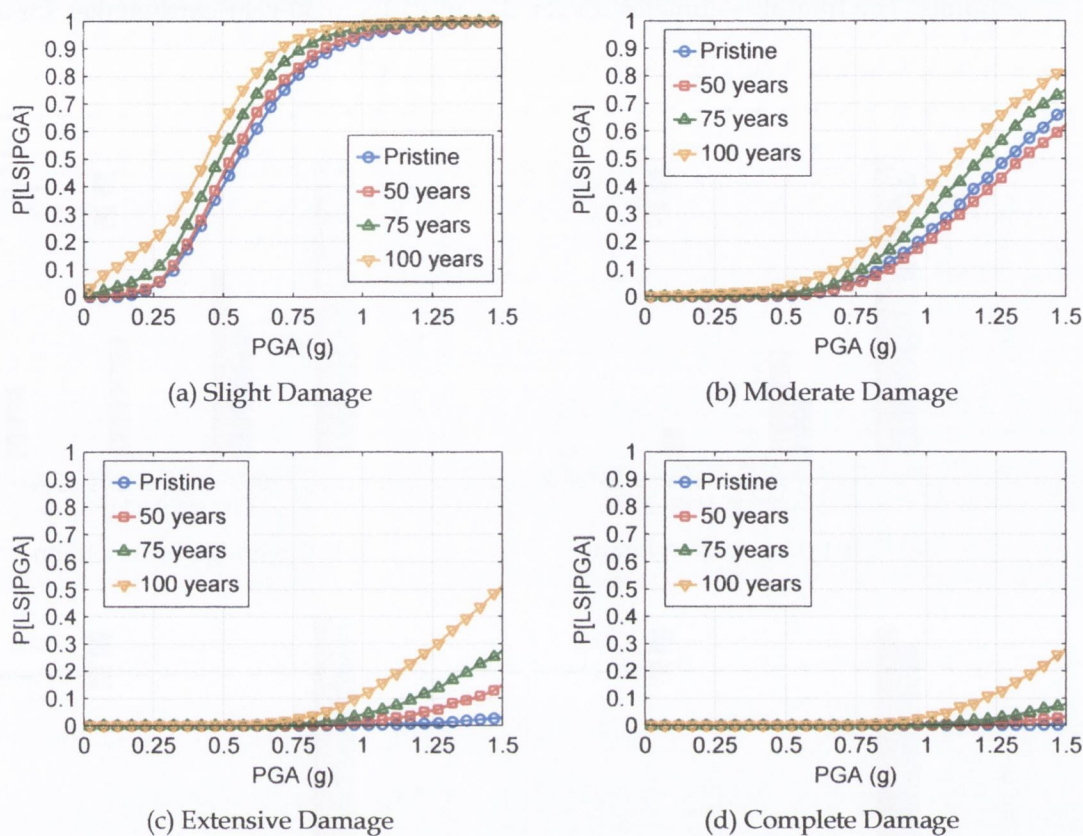


Figure 6.12 System fragility curves at 0, 50, 75 and 100 years

Figure 6.13 shows the probabilities of the bridge system reaching the four limit states for the 2% in 50 year earthquake, for each of the six bridge samples, at 0 and 100 years. From Figure 6.13, it can be seen that the system experiences a smaller increase in fragility with age with an increase in column height. Section 6.1 revealed that the columns become less vulnerable to seismic damage with an increase in height; this means that the fragility of a bridge system with taller columns is less dominated by the column fragility. Considering the abutments tend to experience a slight reduction in fragility with age, the bridge system experiences a smaller increase in fragility with age when they begin to dominate the system fragility. From Figure 6.13 (f), it can be seen that the bridge system actually experiences a slight reduction in fragility with age at the moderate damage state. This is due to the fact that the abutments completely dominate the system fragility at this damage state for the 69 m bridge with 12 m

columns. The fragility estimates for the 2% and 10% in 50 year earthquakes for all the bridge components are shown in tabular form in Appendix B.

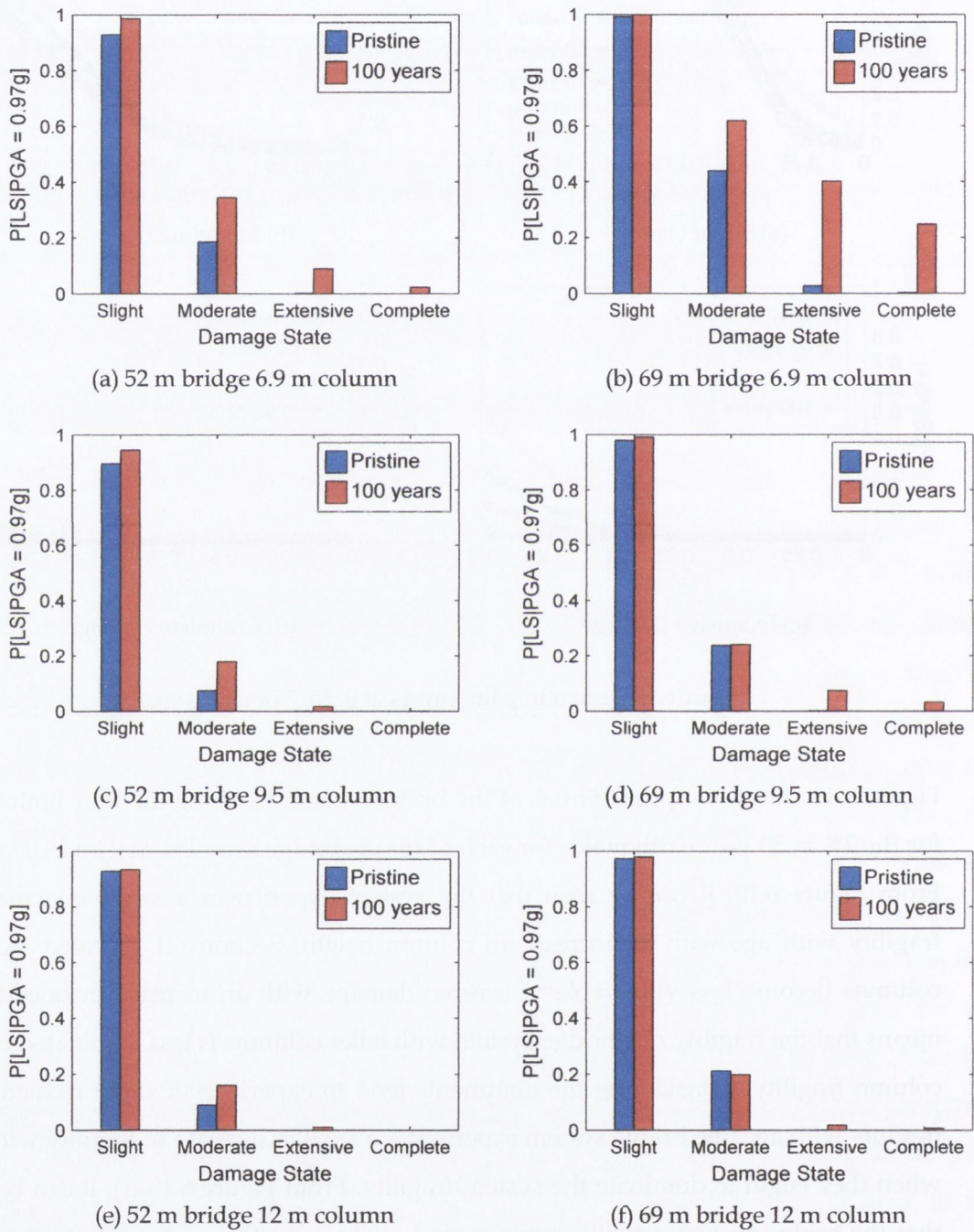


Figure 6.13 System fragility estimates for the six bridge samples for the 2% in 50 year earthquake

6.3 Impact of Settlement on Seismic Fragility Estimates

A fragility analysis is conducted for the 100 year old bridge with settlement levels of 2.5 cm and 5 cm. Recalling Chapter 4, these levels of settlement were chosen based on a survey of the foundation movement of 321 bridges in the US and Canada (FHWA, 1985). It was found that of the bridge components, settlement of the intermediate pier foundation has the greatest impact on the deck fragility. Figure 6.14 shows the deck fragility curves for all the damage states. From Figure 6.14 (a) it can be seen that the probability of slight damage occurring to the deck for the 10% in 50 year earthquake, increases from 29% to 81% with 5 cm of settlement. It is also noted however that the probability of slight damage occurring under gravity loads is 20% and 57% for 2.5 cm and 5 cm of settlement respectively. This suggests, for the higher level of settlement especially, that slight damage is likely to occur due to the occurrence of settlement alone and therefore may not be of concern in a seismic risk assessment. However, the risk of extensive damage may still be of interest and from Figure 6.14 (c), it can be seen that the probability of extensive damage occurring increases from 3% to 21% for the 2% in 50 year earthquake. From Figure 6.14 (d), it can be seen that the complete damage fragility is negligible over the range of PGAs considered in this study.

Figure 6.15 shows the system fragility curves for each of the settlement levels. It can be seen that for the slight, moderate and extensive damage states the fragility increases with settlement. However, at the complete damage state the seismic fragility decreases slightly with settlement. In order to understand why this trend occurs we refer to the probabilistic seismic demand to capacity ratio model for the columns, since the columns dominate the system fragility at the complete damage state. Figure 6.16 shows the probabilistic seismic demand to complete capacity ratio models for the case of no settlement, and settlements of 2.5 cm and 5 cm. This figure shows that at the lower levels of PGA, settlement causes an increase in the column seismic demand; however, at higher levels of PGA, settlement has little effect on the seismic demand. From the figure it can be seen that the regression models accurately predict the demand for the range of earthquake PGAs included in the analysis (i.e. 0.03g - 1.09g), however, extrapolating beyond a PGA of 1.09g leads to inaccurate conclusions. This highlights the danger of predicting seismic fragility beyond the range of PGAs considered.

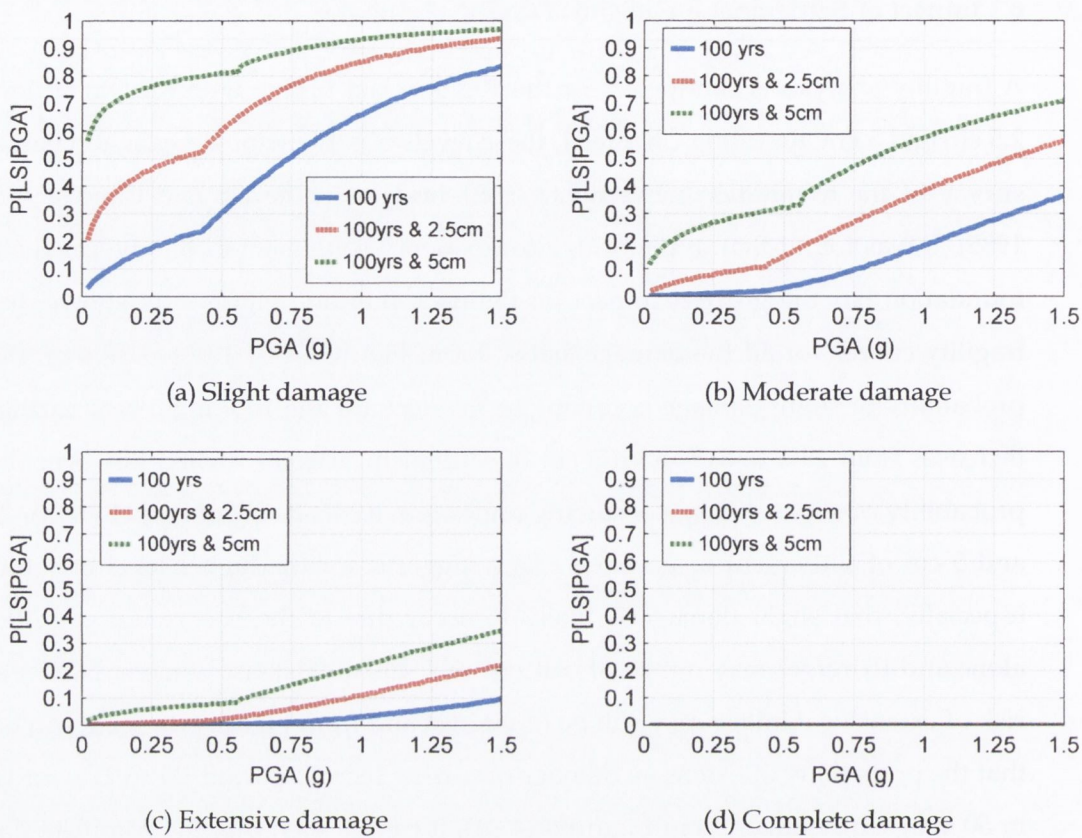


Figure 6.14 Deck fragility curves for 100 year old bridge and effect of settlement

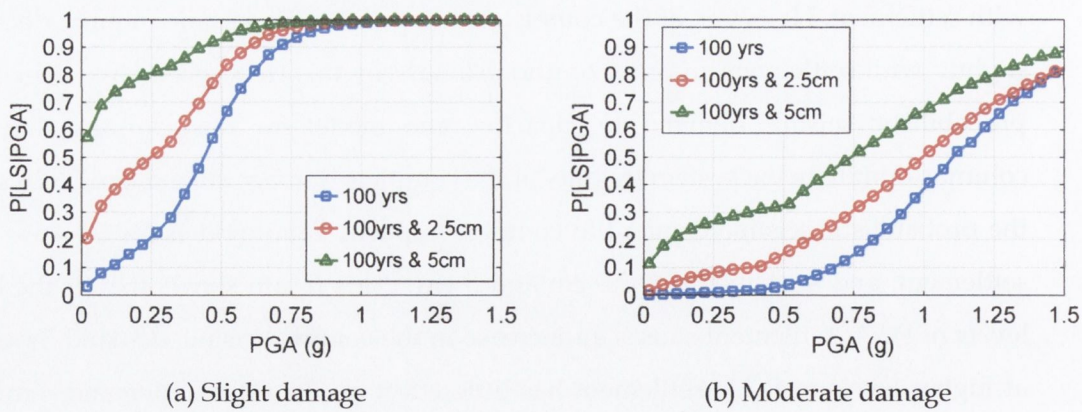


Figure 6.15 (a) & (b) System fragility curves for 100 year old bridge and effect of settlement

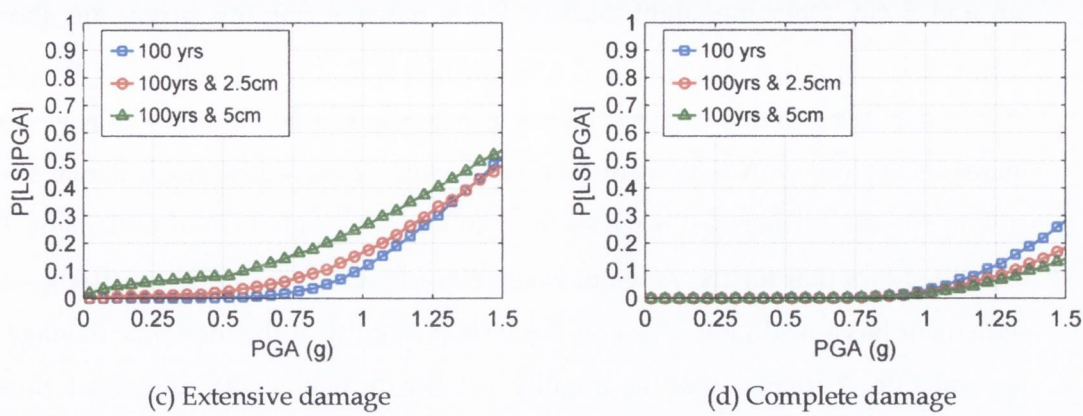


Figure 6.15 (c) & (d) System fragility curves for 100 year old bridge and effect of settlement

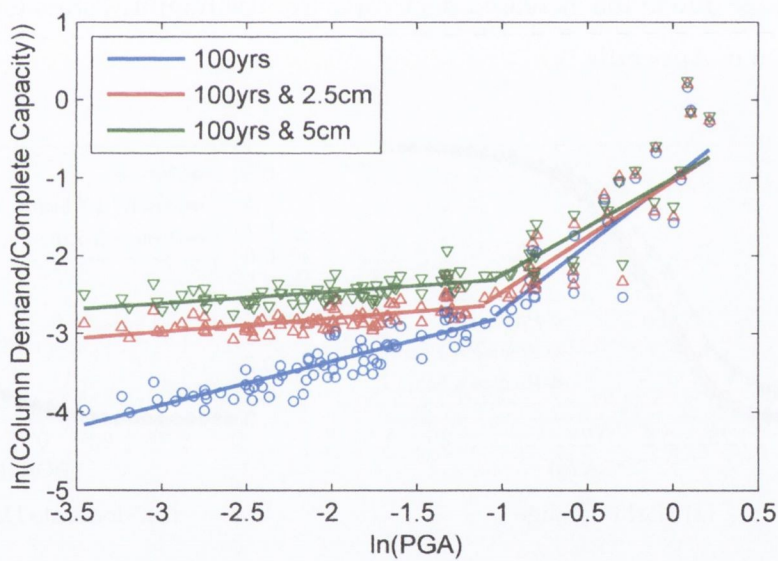


Figure 6.16 Probabilistic seismic demand to capacity ratio model showing the effect of settlement

The previous analysis assumed a high level of corrosion coupled with foundation settlement of an intermediate pier. A further analysis is carried out that assumes settlement levels of 2.5 cm and 5 cm with no corrosion. This analysis is conducted to establish the effect of settlement on the seismic fragility of this integral bridge type in regions where chloride induced corrosion is not an issue. Figure 6.17 shows the system fragility curves for the pristine bridge with no settlement and settlements levels of 2.5

cm and 5 cm. Only the slight and moderate damage fragility curves are shown in Figure 6.17 since the extensive and complete fragility is negligible over the range of PGAs considered in this analysis. Figure 6.17 (a) shows that the slight damage fragility increases slightly with settlement. The probability of exceeding slight damage for the 10% in 50 year earthquake increases from 40% to 58% with 5 cm of settlement. Figure 6.17 (b) shows that for the range of PGAs considered in this study (i.e. 0.03g - 1.09g), settlement has a negligible effect on the system fragility at the moderate damage state. Beyond 1.0g, it appears that the fragility actually decreases with settlement, however, as mentioned previously, extrapolating beyond the PGA levels included in the analysis may lead to an inaccurate prediction of fragility and therefore these results are discounted. A fragility analysis is also carried out on the 69 m bridge accounting for settlement but it was found that settlement has less of an impact on the fragility of the 69 m bridge due to the increased deck capacity. The fragility estimates for this analysis are shown in Appendix B.

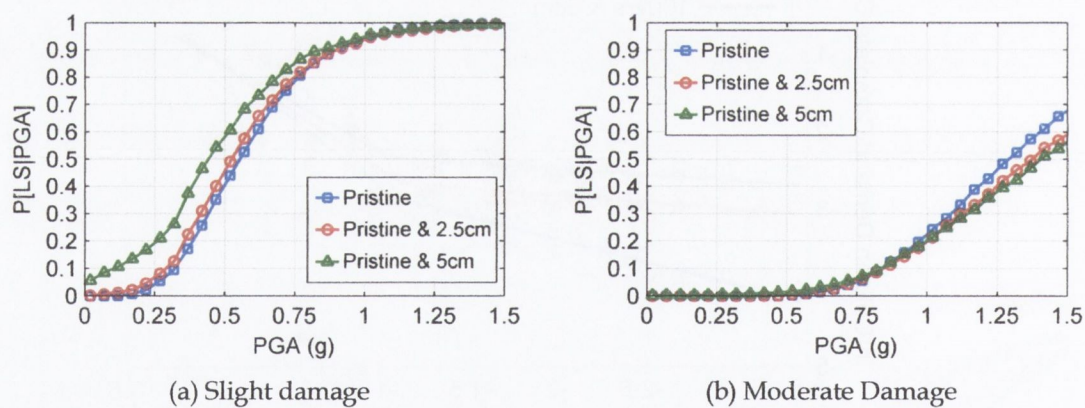


Figure 6.17 System fragility curves for pristine bridge and effect of settlement

6.4 Conclusion

This chapter presents the results of a seismic fragility analysis for a group of three span fully integral ageing bridges. The results are broken into three sections to examine the impact of geometric variation, chloride induced corrosion of the RC deck and columns and foundation settlement of an intermediate pier on the seismic fragility of this

integral bridge type. From the results of this analysis it is seen that the following trends were observed:

- The columns become less vulnerable to seismic loading with an increase in height, however, the abutments become more vulnerable to seismic loading with an increase in column height. The system fragility curves therefore experience an increase in fragility when the abutments dominate the system fragility (i.e. at the slight damage state) and a decrease in fragility when the columns dominate the system fragility.
- Both the columns and the abutments experience an increase in fragility with an increase in bridge length due to the larger superstructure mass inducing greater demands on the bridge components. The system therefore experiences an overall increase in fragility with bridge length. It is important to note that this is the case when the column details are adjusted with bridge length such that the axial load ratio, longitudinal reinforcement ratio and transverse reinforcement ratio are kept constant.
- The system fragility of the integral bridge increases significantly with corrosion. For example, the probability of exceeding extensive damage for the 2% in 50 year earthquake in the Los Angeles region increases by 10% after 100 years. For the uncorroded case, the current fragility method predicts that there is less than 1% probability of exceeding extensive damage, however, this increases to 9% after 100 years. This result underscores the importance of accounting for ageing when assessing the seismic fragility of bridges, since a bridge that may otherwise be deemed safe can become vulnerable to seismic damage when exposed to corrosion deterioration.
- The deck, which has not been included in fragility analyses to date, experiences the greatest increase in fragility with corrosion. For example, the probability of exceeding slight damage for the 10% in 50 year earthquake in the Los Angeles region increases by 29% after 100 years. That being said, the deck only begins to contribute to the bridge system fragility at the slight and moderate damage

states after the equivalent of 75 years of corrosion. Beyond the moderate damage state, the columns dominate the system fragility at all levels of deterioration.

- The columns also experience an overall increase in fragility with corrosion. An exception to this trend occurs at the moderate damage state, when the column fragility decreases slightly after 50 years of corrosion. This is due to the fact that the column capacity at the moderate damage state, defined as the curvature at which concrete spalling occurs, actually increases slightly with corrosion. Therefore, up to 50 years of chloride induced corrosion, the increase in capacity outweighs the increase in demand resulting in an overall decrease in the fragility. It was also seen that corrosion leads to a very slight decrease in abutment fragility.
- Settlement has the greatest impact on the deck fragility when coupled with chloride induced corrosion. Settlement results in an overall increase in the system fragility at the slight and moderate damage states due to moment redistribution in the deck. The system fragility at the extensive damage state is only slightly affected by settlement, whereas the system fragility at the complete damage state is unaffected by settlement. It was also found that settlement only affects the system fragility at the slight damage state when corrosion is neglected.

CHAPTER 7 - COMPARISON BETWEEN THE SEISMIC PERFORMANCE OF INTEGRAL AND JOINTED CONCRETE BRIDGES

7.1 Introduction

As mentioned in Chapter 2, over the past few decades, integral or jointless bridges have become a popular alternative to conventional bridges designed with bearings and expansion joints. In the United Kingdom (UK) for example, designers are now required to consider the integral form for bridges up to 60 m (The Highways Agency, 2001). This trend is also reflected in the United States (US) where at least 40 states are now building some form of jointless bridges and although superstructures with deck joints still predominate, the trend appears to be moving towards the integral form (Mistry, 2005).

Chapter 2 outlined the many advantages associated with integral bridge construction. The main advantage associated with the integral form is the elimination of expansion joints which leads to improved durability. In addition to improved durability, another claim that is under investigation in this thesis is that integral bridges perform better during seismic events. A primary benefit of integral bridges is the elimination of the unseating potential which is a common problem for traditional bridge design. Another advantage is that the moment resisting connection between the column and pier cap provides increased redundancy in the structure and enhances the energy dissipation potential by increasing the number of plastic hinges that are needed to form a collapse mechanism. Mistry (2005) claims that integral bridges have consistently performed well in actual seismic events and significantly reduced or avoided problems such as back wall and bearing damage, associated with seat type abutments. Although there

has been a limited amount of research on the seismic performance of integral bridges that concludes that the integral designs under consideration perform satisfactorily under seismic loads (Sritharan et al., 2005, Frosch et al., 2009), to the author's knowledge, there have only been a few studies carried out comparing the seismic fragility of bridges with integral and seat type abutments (Ramanathan, 2012, Zakeri et al.). Furthermore, these studies only examined bridges that were integral over the column bents. This comparison is important in order to draw conclusions on the change in seismic performance when integral construction is used. It has also been suggested that without a system of joints and bearings to allow for superstructure movement in jointed bridges, greater demands may be placed on the abutments and substructure (Frosch et al., 2009). This claim will be investigated in this chapter by directly comparing abutment and column fragilities of the two bridge forms under investigation. In the coming sections, finite element models are developed for a group of three-span prestressed concrete beam and slab bridges, articulated with elastomeric bearings at the abutments and column bents. As with the previous group of integral bridges, six geometric configurations are considered with varying bridge length and column height. Fragility curves for the group of jointed bridges are developed following the method described in Chapter 4. The impact of geometric variation on the seismic performance of the jointed bridges is then presented. Finally, the fragility curves of the jointed bridges are compared with the integral bridge fragility curves, in order to draw conclusions on the relative performance of the integral and jointed bridge types.

7.2 Analytical Bridge Component Modelling Strategies for the Jointed Bridges

For this section of the thesis, the six geometric configurations of the integral bridge examined in the previous chapters will be adapted to include bearings at the abutments and over the column bents, as shown in Figure 7.1. This approach is adopted in order to make a direct comparison between the seismic performance of the integral and jointed bridge forms. It is acknowledged that in reality, certain aspects of the design would differ between integral and jointed bridges. These differences need to be addressed in order to provide a valid comparison between the two bridge types.

Firstly, the abutments typically used for jointed bridges are seat type abutments, as opposed to diaphragm abutments, which are used for integral bridges. Due to the lack of available design drawings for a jointed bridge with similar geometric properties to the case study integral bridge, it is decided to adopt the integral abutment force deformation model for the jointed bridge abutment. This approach is acceptable because although the seismic response of the integral bridge is strongly affected by the abutment behaviour due its continuity with the superstructure, for the case of jointed bridges, the superstructure is isolated from the abutments and therefore the abutments do not strongly influence the bridge response. Furthermore, results of the fragility analysis presented in Section 7.6 show that the abutments are less vulnerable to seismic loading than the other bridge components and therefore do not dominate the system fragility at any damage state.

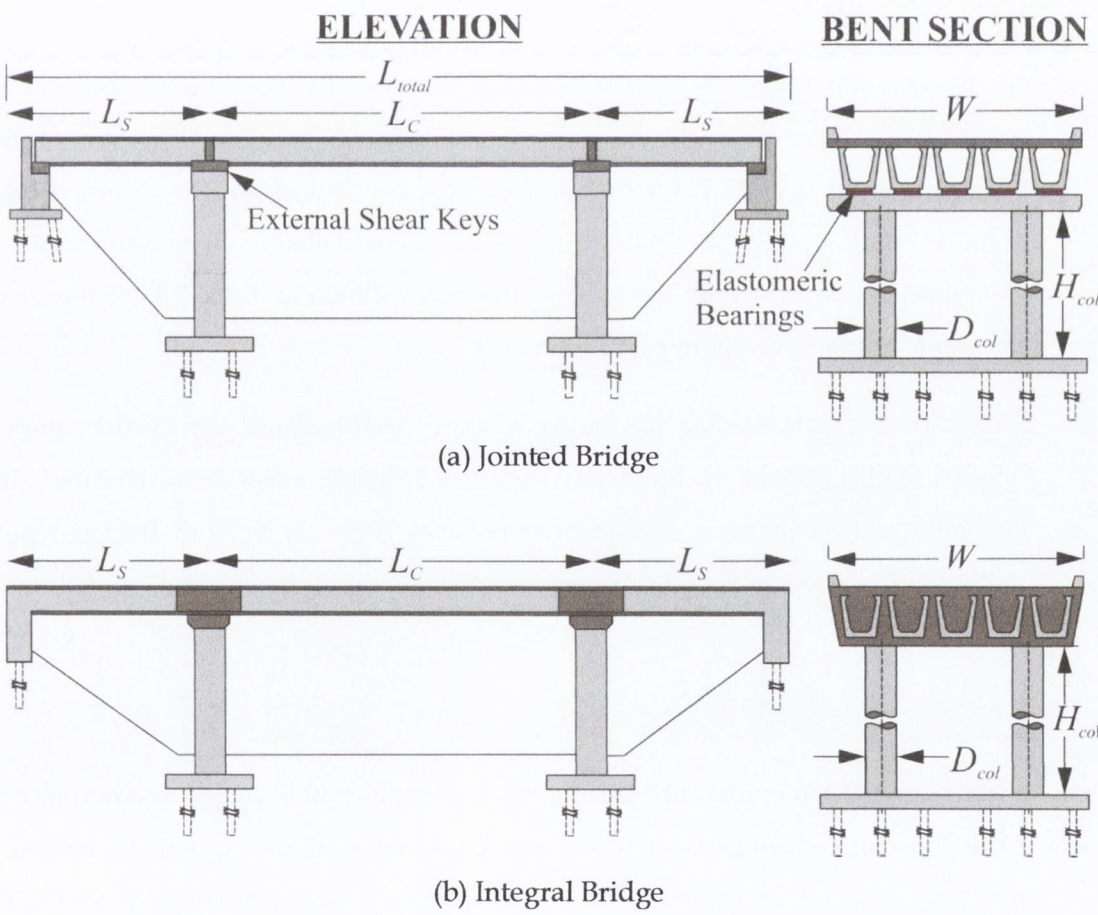


Figure 7.1 General layout of integral and jointed bridges

Another detail of the design expected to differ between the integral and jointed bridges is the RC column size and reinforcing details. Section 3.2 presented the details of the RC columns for the integral bridge according to design drawings. From these drawings, it was seen that the base bridge columns have a diameter of 1.0 m and are reinforced longitudinally with 18 No. 40 mm $\varnothing$ bars, resulting in a longitudinal reinforcement ratio of 2.88%. It must be noted that in practice, the RC columns of the jointed bridge would be designed for smaller moments under gravity loading due to the fact that there is no moment transfer between the deck and the columns. This must be taken into consideration when assessing the results of the fragility analysis. In order to investigate the effect this design change might have on the seismic fragility of the jointed bridge, a further fragility analysis is carried out for the jointed bridge with a reduction of 10% in column diameter while maintaining the same longitudinal reinforcing ratio.

In practice, the reinforcement in the deck slab and the prestressed concrete beams would also differ between the jointed and integral bridges. However, as further described in Section 7.2.1, for the purpose of this comparison it is assumed the deck remains elastic during the seismic analysis and is modelled using elastic beam column elements. In this case, only the section properties shown in Table 7.1 are required and therefore, the reinforcement is not considered.

The following sections describe the analytical modelling strategies for the components unique to the jointed bridge type. These include the elastomeric bearings, impact elements and shear keys. For the components common to both bridge types, the methods described in Section 3.2 are adopted for the finite element modelling of the jointed bridge.

7.2.1 Deck Elements

Until now, the superstructures of the integral bridges under consideration have been modelled using displacement based beam column elements and fiber defined cross sections. This was a departure from the usual approach presented in the literature, which adopts elastic beam column elements assuming the superstructure generally remains elastic during seismic events (Aviram et al., 2008). The new approach was adopted in order to investigate whether corrosion leads to the superstructure reaching

reduced limit state capacities during seismic events. However, from the results of the fragility analysis presented in Chapter 6, it was seen that the deck only begins to contribute to the slight and moderate damage states of the bridge system after 75 years and even after 100 years, it does not contribute to the system fragility beyond the moderate damage state. For the combined case of 100 years of corrosion and foundation settlement of the intermediate pier, the deck contributes to the system fragility and the slight, moderate and extensive damage states. However, the cases mentioned include very extensive levels of deterioration and for the purpose of this chapter, which is to compare the seismic performance of the pristine integral and jointed bridges, the deck is modelled using elastic beam column elements following the recommendations of Aviram et al. (2008). A further motivation for using elastic beam column elements to model the superstructure is the fact that such models are much less computationally expensive. The elastic models have run times on the order of 100 times less than the models using displacement based beam column elements to model the superstructure.

Figure 7.2 shows the system fragility curves for the 52 m integral bridge with a superstructure modelled using both fiber defined and elastic cross sections. From the figure it can be seen that there is a maximum difference of 10% between the probabilities of exceeding slight damage for the fiber and elastic models. The difference between the models at the other damage states is negligible. The difference in fragility is attributed to the fact that the fiber model accounts for the nonlinear behaviour of the reinforcing steel and concrete and OpenSEES estimates the stiffness of the fiber cross section numerically. The elastic section properties used in the current analysis are the gross section properties since Caltrans (2004) recommends that no reduction in stiffness is carried out for prestressed box girder sections. This is due to the fact that the location of the pre-stressing steel's centroid and the direction of bending have a significant impact on how cracking affects the stiffness of pre-stressed members. Multi-modal analyses are incapable of capturing the variations in stiffness caused by moment reversal (Aviram et al., 2008). The elastic section properties could be adjusted slightly to adopt the fiber section stiffness; however, given that the difference in fragility is slight and that the deck will not be accounted for the current fragility analysis, this is deemed unnecessary. Furthermore, considering the aim of this chapter is to compare

the seismic performance of integral and jointed bridges, modelling the superstructure with elastic beam column elements is considered suitable, once the approach adopted is consistent for both bridge types. The section properties of each bridge length are calculated for the composite deck cross section. The section properties are calculated for each bridge using the axes presented in Figure 7.3 and can be seen in Table 7.1.

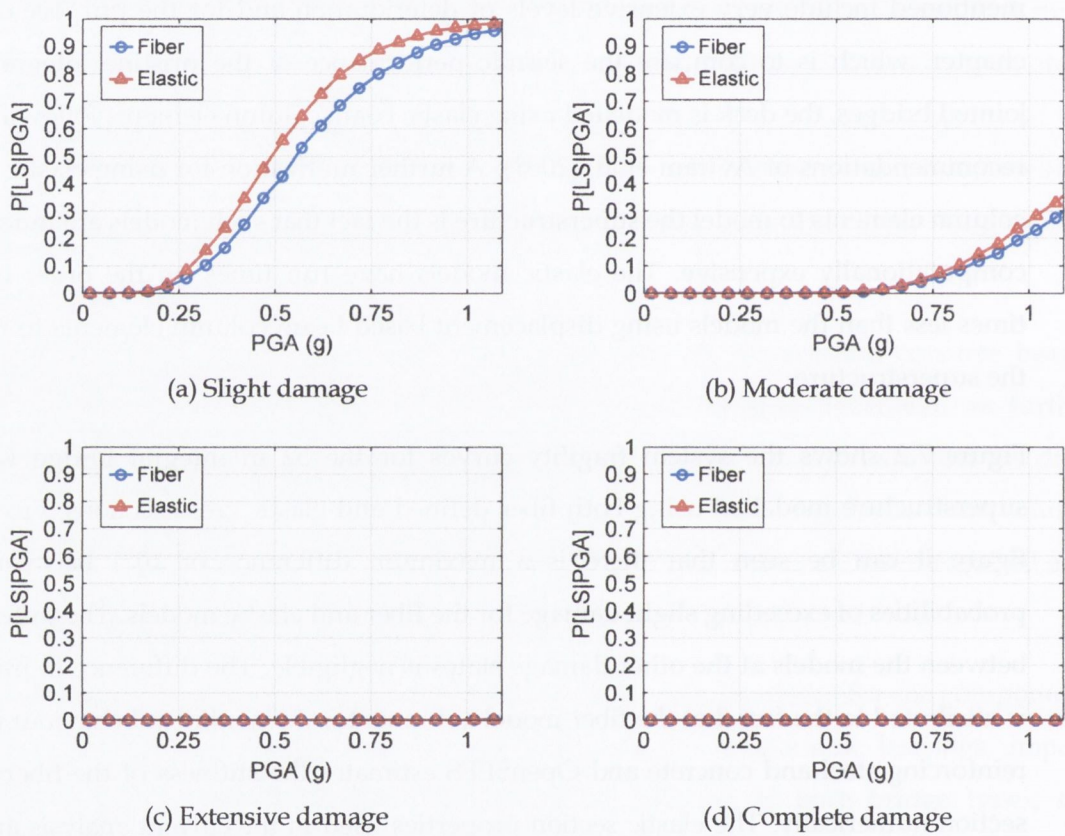


Figure 7.2 System fragility curves for the 52 m integral bridge with a superstructure modelled using fiber defined cross and elastic cross sections

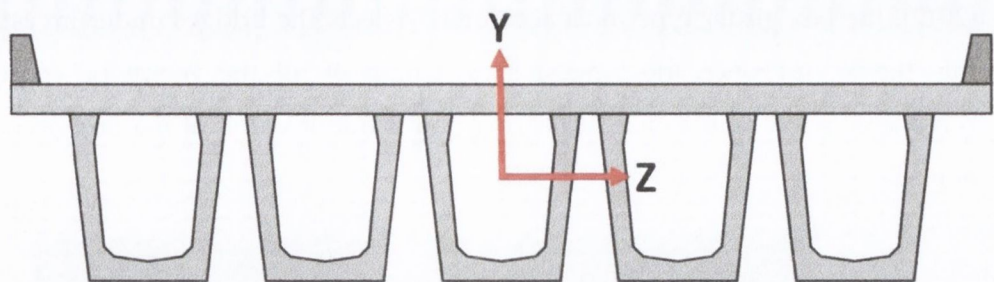


Figure 7.3 Schematic of bridge superstructure

Table 7.1 Deck section properties for the 52 m and 69 m bridges

	Area (m2)	Centroid (m)	I _{zz} (m4)	I _{yy} (m4)	J(m4)
Bridge 1	5.4482	1.00441	1.84834	25.624789	1.99623
Bridge 2	5.6153	1.061339	2.159929	39.088075	2.22847

7.2.2 Elastomeric Bearings

For the jointed bridge, elastomeric bearings consisting of an elastomeric or rubber pad are used, since they have been cited as a very common bearing for concrete girder type bridges (Nielson, 2005, Ramanathan, 2012). An example of an elastomeric bearing pad and concrete girder configuration can be seen in Figure 7.4. Elastomeric bearing pads resist horizontal forces by developing a frictional force between the pad and the girder. Their behaviour is characterized by sliding with an initial stiffness that remains constant until the coefficient of friction is exceeded. Once the coefficient of friction is exceeded, the stiffness of the rubber pad drops to zero and it can resist no further horizontal forces.

Given that the author did not have access to design drawings for the jointed bridge, Ramanathan (2012) was consulted to identify suitable elastomeric bearing sizes given the span lengths and superstructure geometry of the case study bridges considered herein. Ramanathan (2012) consulted the National Bridge Inventory (NBI) to capture and understand the design and detailing of various bridge components of Californian highway bridges. This work found that multi-span continuous concrete box girder bridges, with spans of less than 40 m, built after 1990, tend to use elastomeric bearing pads with dimensions of 35.6 × 35.6 × 6.35 cm. Of the bridges surveyed by Ramanathan

(2012), the box girder type most accurately reflects the bridges under investigation in this thesis and since the longest span length of all the geometric configurations considered is 38 m, this size of bearing pad is adopted for all the bridges considered herein.



Figure 7.4 Elastomeric bearing pad and concrete girder configuration (ODOT, 2013)

As mentioned previously, the elastomeric bearing pads resist horizontal forces with an initial stiffness that drops to zero once the yield force is exceeded. The elastomeric pads can therefore be modelled using an elastic perfectly plastic material. The pads are modelled using the *Steel 01* material provided by OpenSEES, with a strain hardening ratio of zero. This material is then applied to zero length elements connecting the abutments and bent caps to the superstructure. Figure 7.5 shows the force deformation response of an elastomeric bearing pad. It is noted that the force deformation response is the same in both the longitudinal and transverse directions. From the figure, it can be seen that the two parameters needed to model the response of the bearing are the initial shear stiffness, k_0 , and the yield force, F_y . Choi (2002) proposed that the initial shear stiffness, k_0 , of the bearing pad can be estimated as follows:

$$k_0 = \frac{GA}{h_r} \quad \text{Eqn. 7.1}$$

where A is the area of the elastomeric bearing, G is the shear modulus of the elastomer and h_r is the thickness of the elastomeric pad.

Following Ramanathan (2012), the yield force is estimated by multiplying the normal force, N , acting on the bearing by the coefficient of friction, μ , of the pad. The coefficient of friction takes into account the interface between the elastomeric rubber and the concrete surface. Experimental tests carried out by Schrage (1981) have shown that the coefficient of friction for an elastomeric bearing pad is a function of the normal stress on the bearing, σ_m , and is found as follows:

$$\mu = 0.05 + \frac{0.4}{\sigma_m}$$

Eqn. 7.2

where μ is the coefficient of friction and σ_m is the normal stress on the bearing given in MPa. For the fragility analysis, the shear modulus and coefficient of friction are considered random variables and their distributions are described in Section 7.3.

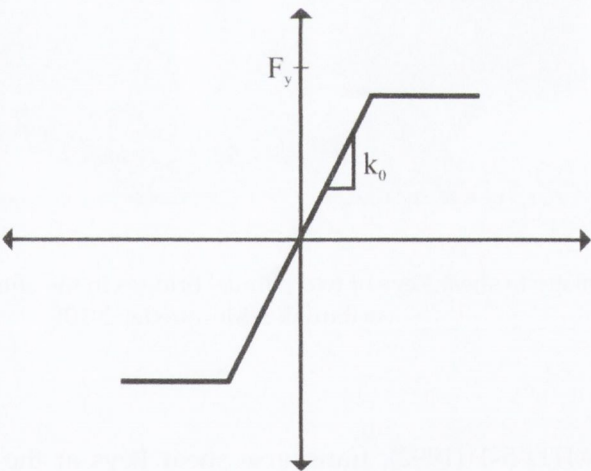


Figure 7.5 Force deformation response of an elastomeric beading pad

7.2.3 Shear Keys

For this bridge, external shear keys are located at the abutments and column bents. According to Ramanathan (2012), internal shear keys are more typical of older bridges and in California for example, most have been removed and replaced with external

shear keys during past retrofit programmes. Shear keys play a crucial role in restraining transverse movement. They are necessary to transfer the horizontal forces from the superstructure to the substructure (i.e. the abutments and column bents) when bearing supported details are adopted. Figure 7.6 shows two parallel bridges in the aftermath of the magnitude 8.8 earthquake that occurred in Chile in 2010. From the figure it can be seen that the shear keys at the column bent of the left bridge have sheared off, leaving the girder partially overhanging at the bent. The shear keys at the column bent of the right bridge have some shear cracks but remain intact.



Figure 7.6 Damage to shear keys of two parallel bridges in the aftermath of the 2010 Chilean earthquake (Mosqueda, 2010)

According to MTD 5-1 (1992), transverse shear keys at the column bents should be designed to resist 75% of the adjacent bent shear capacity. According to personal communication between Ramanathan (2012) and Caltrans, the shear keys at the abutments should be designed to resist 120% of the bent shear capacity. This design approach is in line with the capacity design process, since the intention is that the formation of plastic hinges in the underlying columns occurs before the failure of the shear keys (Ramanathan, 2012).

The model adopted for the force deformation relationship of the transverse shear keys is based on a series of experiments on external shear keys in bridge abutments carried

out by Megally et al. (2002). Figure 7.7 shows the force deformation relationship for the transverse shear keys. This force deformation relationship is modelled using the *Hysteretic* material provided by OpenSEES, which is applied to zero length elements connecting the abutments and bent caps to the superstructure. From the figure it can be seen that the necessary parameters to model the shear key behaviour are the shear key capacity, P_{cap} , the gap between the shear key and the girder, δ_{gap} , and the maximum displacement before the capacity reduces to zero, δ_{max} . As mentioned previously, the capacity of the shear keys should be estimated as:

$$P_{cap} = factor \times V_{bent} \quad \text{Eqn. 7.3}$$

where *factor* is 0.75 at the column bents and 1.2 at the abutments and V_{bent} is the shear capacity of the bent. Following the current American Concrete Institute code (ACI, 2005), the shear strength of a concrete column is found as follows:

$$V_n = V_c + V_s \quad \text{Eqn. 7.4}$$

where V_c is the portion of the design shear force to be carried by the concrete shear resisting mechanism and V_s is the remainder, carried by the truss mechanism involving transverse reinforcement. The ACI code presents the following equation to estimate V_c for members subjected to combined shear, moment and axial compression:

$$V_c = 0.166 \left(1 + \frac{P}{13.8A_g} \right) \sqrt{f'_c} D d_{eff} \quad \text{Eqn. 7.5}$$

where P is the axial load of the column; A_g is the gross cross-sectional area of the column; f'_c is the concrete compressive strength; D is the diameter of the column and d_{eff} is the effective depth of the column taken as $0.8D$. The transverse reinforcement contribution is estimated as:

$$V_s = \frac{A_v f_y d}{s} \quad \text{Eqn. 7.6}$$

where A_v is the area of transverse reinforcement; s is the vertical spacing between the transverse reinforcement and f_y is the yield stress of the transverse reinforcement. Once the shear capacity of a column is known the total shear capacity of the bent can be found as:

$$V_{bent} = n \times V_n$$

Eqn. 7.7

where n is the number of columns per bent. According to the research carried out by Megally et al. (2002), it was found that shear keys undergo a maximum displacement of 8.9 cm before their capacity reduces to zero. Therefore, this value is adopted for δ_{max} . δ_{gap} is considered a random variable whose distribution is discussed in Section 7.3.

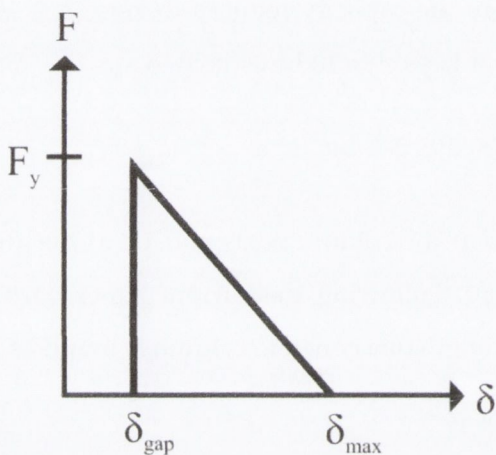


Figure 7.7 Force deformation relationship for the transverse shear keys

7.2.4 Impact Elements

For the case of this bridge, whose superstructure is supported on elastomeric bearings at the column bents and the abutments, pounding between the deck and the abutment backwall may be an issue when the bridge is seismically loaded. According to Nielson (2005), it is recognised that pounding between the deck and the abutment backwall can affect the way a bridge responds to seismic loading and it is therefore important to include the effects of pounding when generating analytical bridge models. A study carried out by Muthukumar (2003) recommends a procedure for developing an appropriate model for pounding behaviour. This study found that impact models that do not allow for energy dissipation, such as linear models, tend to overestimate the system response due to impact. Therefore, a bilinear model is recommended. The model recommended by Muthukumar (2003) is shown in Figure 7.8.

From Figure 7.8, δ_{gap} represents the gap between the deck and the abutment backwall, which needs to be closed before the model is engaged. The other parameters of the model, K_{t1} , K_{t2} , δ_y and δ_m are derived by comparing the area under the hysteresis to the energy dissipated upon impact. Using a stereomechanical approach, which is further described in Muthukumar (2003), the energy dissipated during impact, ΔE can be derived as follows:

$$\Delta E = \frac{k_h \delta_m^{n+1} (1 - e^2)}{n + 1} \quad \text{Eqn. 7.8}$$

where k_h is an impact stiffness parameter with a typical value in imperial units of 2,608 kip-in^{3/2} per 1.9 m width of the abutment; n is the Hertz coefficient, typically taken as 3/2; e is the coefficient of restitution taken as 0.8 and δ_m is the maximum penetration of the two decks, taken as 25.4 mm for this thesis following Nielson (2005).

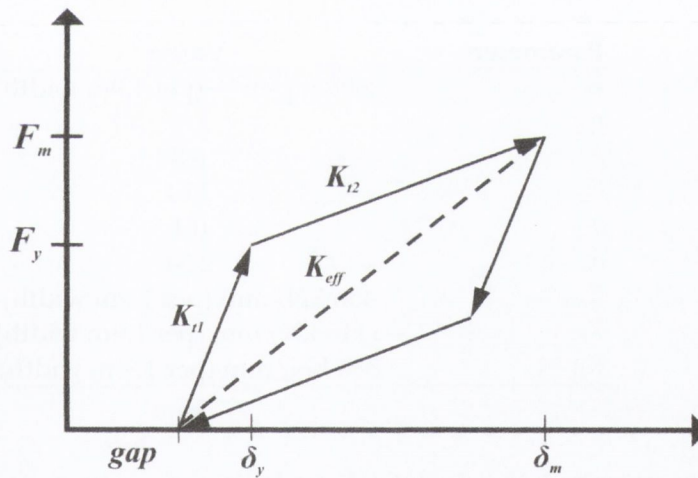


Figure 7.8 Force deformation model for pounding between the deck and the abutment backwall

Once the dissipated energy is estimated, the parameters of the bilinear model are found which yield the same energy dissipation. The effective stiffness, K_{eff} , is obtained as follows:

$$K_{eff} = k_h \sqrt{\delta_m} \quad \text{Eqn. 7.9}$$

After some manipulation, the remaining impact model parameters are obtained using the following equations:

$$\delta_y = a\delta_m \quad \text{Eqn. 7.10}$$

$$K_{t1} = K_{eff} + \frac{\Delta E}{a\delta_m^2} \quad \text{Eqn. 7.11}$$

$$K_{t2} = K_{eff} - \frac{\Delta E}{(1-a)\delta_m^2} \quad \text{Eqn. 7.12}$$

where K_{t1} is the initial stiffness; K_{t2} is the post-yield stiffness; a is a yield parameter that is taken as 0.1 following Nielson (2005) and δ_y is the yield penetration. Table 7.2 shows the values for the parameters found using the procedure proposed by Muthukumar (2003) and values recommended by Nielson (2005).

Table 7.2 Impact Element Modelling Parameters

Parameter	Value
k_h	2608 kip-in ^{-3/2} (per 1.9m width)
n	3/2
e	0.8
δ_m	25.4
a	0.1
δ_m	2.54
K_{eff}	456 kN/mm (per 1.9m width)
K_{t1}	1116 kN/mm (per 1.9m width)
K_{t2}	384 kN/mm (per 1.9 m width)

This section described the analytical modelling procedures for those components unique to the jointed bridge type. For components common to the integral and jointed bridges, the procedures described in Section 3.2 are used for the finite element modelling of the jointed bridges considered herein.

7.3 Uncertainty in Modelling Parameters

As described in Chapter 5, in order to carry out a fragility analysis, uncertainty in certain bridge modelling parameters must be accounted for. For those parameters common to the integral and jointed bridge types, the distributions described in Section 5.4 are adopted for the jointed bridges. However, there are certain bridge modelling

parameters that are unique to jointed bridges and therefore their distributions are described here.

7.3.1 Elastomeric Bearing Pad Shear Modulus

As previously described, the stiffness of an elastomeric bearing pad, k_o , is a function of the rubber's shear modulus, G . According to Nielson (2005), there is very little information available on the distribution of the shear modulus for the elastomeric bearings. For this reason, a uniform distribution is assumed following (Nielson, 2005). According to the AASHTO LRFD Design Specifications (AASHTO, 2012), the shear modulus of an elastomeric bearing material should vary between 0.66 MPa and 2.07 MPa. Therefore, these values are used for the upper and lower limits of the uniform distribution following Nielson (2005).

7.3.2 Coefficient of Friction for Elastomeric Bearings

The coefficient of friction between the bearings and the concrete bent is the other main parameter needed to define the force deformation behaviour of an elastomeric bearing pad. The empirical expression shown in Eqn. 7.2 is used to find the coefficient of friction for the bearings. In order to account for uncertainty in the coefficient of friction, a lognormally distributed multiplication factor is used with a median value of zero and a logarithmic standard deviation of 0.10, following the recommendations of Mander et al. (1996) and Dutta (1999).

7.3.3 Gaps

For the jointed bridge under consideration, a gap exists between the deck and the abutment backwall, as well as between the girders and the external shear keys. Following Ramanathan (2012), who carried out work on multi-span concrete jointed bridges in California of similar length to the bridges under consideration, it is assumed that both gaps are uniformly distributed between 0.0 mm and 38.1 mm.

7.4 Component Limit States

Chapter 5 described the procedure adopted in this thesis to carry out a time dependent component and system seismic fragility analysis of a highway bridge. Recalling Eqn.

5.1, in order to carry out a component fragility analysis, knowledge is required of the seismic demand for a given ground motion intensity, as well as the seismic capacity for a given limit state. It is noted that although Chapter 5 proposed a method to carry out a time dependent seismic fragility analysis accounting for corrosion of the bridge components, for the purpose of the current comparison between integral and jointed bridges, fragility analyses are carried out for only the pristine bridges. In Chapter 5, models for the deck, column and abutment capacities were presented. For the case of the jointed bridge, additional components must be included in the fragility analysis including the elastomeric bearings and shear keys. Furthermore, as mentioned previously, the deck is not considered as a component in the current fragility analyses.

As described in Chapter 5, components of the bridge are divided into primary and secondary components depending on their contribution to system level damage. Following recommendations by Ramanathan (2012), the elastomeric bearings and shear keys are considered secondary components, since it is thought they will not contribute to extensive or complete damage of the bridge system. Table 7.3 summarises the primary and secondary components for the integral and jointed bridges. The following sections present the capacity models for the elastomeric bearing pads and shear keys.

Table 7.3 List of primary and secondary components of the integral and jointed bridges

Integral	Jointed
Primary components	
Columns	Columns Deck unseating
Secondary components	
Abutment active displacement	Abutment active displacement
Abutment passive displacement	Abutment passive displacement
Abutment transverse displacement	Abutment transverse displacement
	Elastomeric bearing pads
	Shear keys

7.4.1 Elastomeric Bearing Pads

The Engineering Demand Parameter (EDP) adopted for the elastomeric bearings is displacement. Capacity models for the elastomeric bearings are adopted from

Ramanathan (2012). According to Ramanathan (2012), elastomeric bearing pads are manufactured to undergo large displacements without any significant strength degradation based on Caltrans specifications. The bearings remain elastic until about 100% shear strain, which is taken as the slight damage state. The bearings experience tearing when 300% to 350% shear strain is exceeded, which is taken as the moderate damage state. The displacement values corresponding to these levels of shear strain were calculated by Ramanathan (2012) and are listed in Table 7.4. As for the component capacities described in Chapter 5, the elastomeric bearing pad capacities are lognormally distributed with a lognormal standard deviation or dispersion, β_C , of 0.35 across all damage states following Ramanathan (2012).

7.4.2 Shear Keys

As mentioned previously, the jointed bridge under consideration has external shear keys at the abutments and bents to limit transverse displacement of the superstructure. These components are typically designed to shear off when they fail. The values for the displacements corresponding to the slight and moderate damage states are found based on testing of external shear keys in the University of California, San Diego (Megally et al., 2002). At the displacement corresponding slight damage, minor cracks in the shear keys are expected. At the displacement corresponding to moderate damage, the shear keys are expected to have broken off. The displacement values corresponding to the slight and moderate damage states are show in Table 7.4. The values for dispersion, β_C , are taken as 0.35 across all damage states following Ramanathan (2012).

Table 7.4 Limit states for the elastomeric bearings and shear keys

	Median Values, S_C (mm)				β_C
	Slight	Moderate	Extensive	Complete	
Elastomeric Bearings	25.4	101.6	N/A	N/A	0.35
Shear Keys	38.1	127	N/A	N/A	0.35

7.4.3 Deck Unseating

Deck Unseating is defined only at the complete damage and depends on the abutment seat with. Following Ramanathan (2012), the unseating limit state is taken as 600 mm

based on bridge plans. This reflects typical abutment seat widths in Californian modern bridges. Following Ramanathan (2012), the dispersion, β_C , is taken as 0.35.

7.5 Modal and Deterministic Time History Analysis of the Jointed Base Bridge

As mentioned in Chapter 3, the first step in carrying out a dynamic analysis is to perform a modal or eigenvalue analysis to determine the natural periods and mode shapes of a structure. Eigenvalue analyses of the different bridge geometries of the jointed bridge are performed using OpenSEES and the first three periods are shown in Table 7.5. From the table it can be seen that the period increases with both column height and bridge length. As with the integral bridge, the first three mode shapes for the different geometric configurations are similar, therefore, only the mode shapes for the 52 m bridge with 6.9 m columns are shown in Figure 7.9. From the figure it can be seen that the first two mode shapes are in the longitudinal and transverse directions, while the third mode invokes a torsional response of the bridge.

Table 7.5 First three periods of the jointed bridge configurations considered

Column Height	6.9 (m)		9.5 (m)		12.0 (m)	
Bridge Length	52 (m)	70 (m)	52 (m)	70 (m)	52 (m)	70 (m)
1st mode	0.93	1.06	1.0	1.14	1.06	1.2
2nd mode	0.78	0.90	0.82	0.95	0.87	1.0
3rd mode	0.55	0.64	0.56	0.65	0.58	0.67

Figure 7.10 shows the comparison between the fundamental periods for the different geometric configurations of the integral and jointed bridges. From the figure it can be seen that the fundamental periods of the jointed bridges are more than twice as long as the fundamental periods of the integral bridges. Furthermore, it can be seen that the jointed bridges experience a greater increase in period with bridge length than the integral bridges.

A time history analysis is then performed on the 52 m jointed bridge with a column height of 6.9 m. The bridge is first subjected to the X and Y components of a ground motion record in the longitudinal and transverse directions respectively, both separately and simultaneously. The ground motion components applied in the

longitudinal and transverse directions have peak ground accelerations (PGA) of 0.99g and 1.1g respectively. A description of these records, including plots of the acceleration time histories and response spectra, can be found in Section 3.3. As with the deterministic analysis in Chapter 3, 4.5% Raleigh damping is used in the current time history analyses.

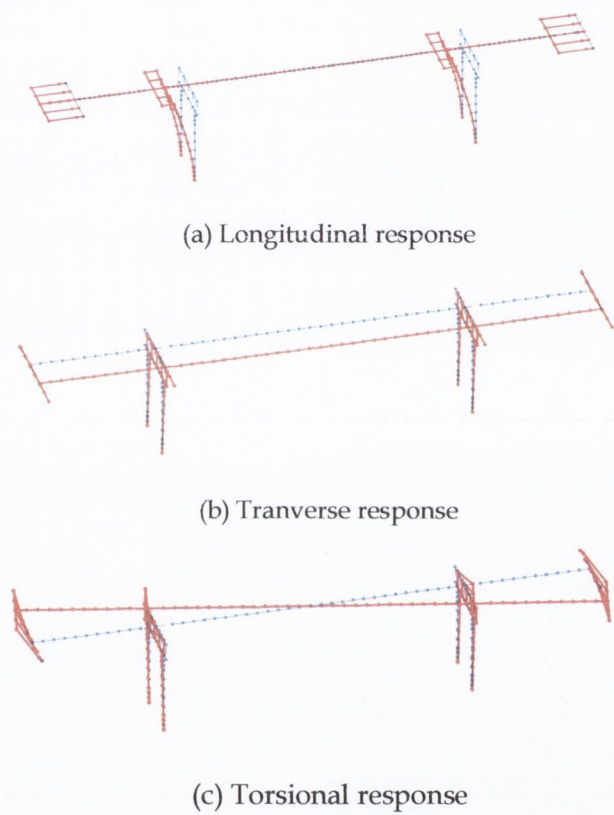


Figure 7.9 First three mode shapes for base bridge

From studying the response time histories of the various bridge components when the ground motions are applied both separately and simultaneously in the longitudinal and transverse directions, it is seen that applying the ground motions simultaneously in the longitudinal and transverse directions does not have a significant effect on the component responses. In contrast to this result, applying the ground motion records simultaneously had a significant effect on the integral bridge component responses. For example, recalling Figure 3.22 (a) and Figure 3.23 (a), it was seen that applying the ground motion records simultaneously resulted in the maximum curvature increasing

from 0.0083 /m to 0.0142 /m. This represents a 71% increase. From Figure 7.11 (a) and Figure 7.12 (a), it can be seen that applying the ground motion records simultaneously for the jointed bridge results in the maximum curvature increasing from 0.0072 /m to 0.0076 /m. This represents a much smaller increase of 5%. The rest of the results presented in this section are the component responses when the ground motions are applied in the longitudinal and transverse directions simultaneously.

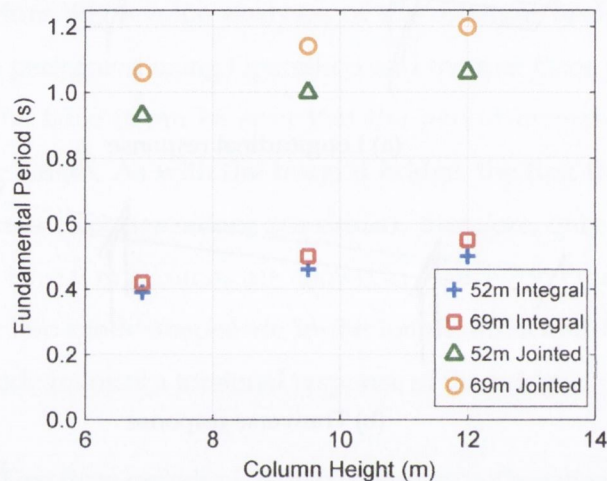


Figure 7.10 Fundamental periods for jointed and integral bridges

Figure 7.13 shows a plot of the maximum curvature response over the height of the column in the longitudinal and transverse directions. The yield curvature of 0.007 /m is marked on both figures. From Figure 7.13 (a), it can be seen that the yield curvature in the longitudinal direction is exceeded at the column base and that the curvature is decreasing along the height of the column. From Figure 7.13 (b), it can be seen that the curvature in the transverse direction does not exceed the yield curvature under the imposed seismic load. From the figure it is also noted that a plastic hinge could occur at either the column base or the column top in the transverse direction. This is due to the fact that the bent cap is stiffer in the transverse direction than the longitudinal direction and therefore prevents rotation at the column top, resulting in larger curvatures at the column top in the transverse direction. It must also be noted here that the curvature demands seen in the columns of the jointed bridge are smaller than those seen in the integral bridge columns for the same ground motion records. This is due to

the fact that the bearings allow movement of the superstructure and therefore less force is transferred to the substructure. This finding is in line with suggestions in the literature that without a system of joints and bearings, greater demands may be placed on the substructures of integral bridges (Frosch et al., 2009). Additionally, the jointed bridge has a longer fundamental period and from examining the ground motion response spectra shown in Figure 3.17 (a) & (b), a smaller response is expected for bridges with larger periods.

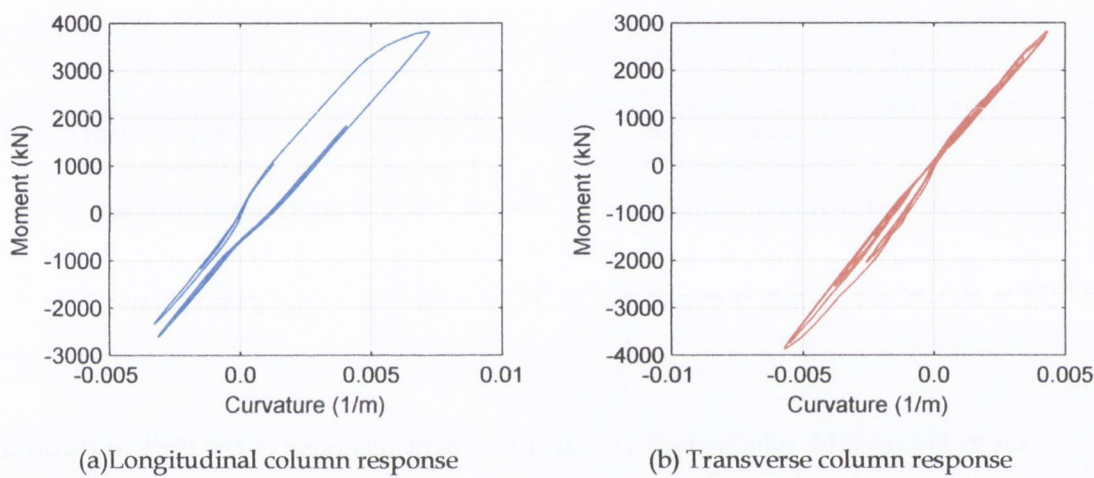


Figure 7.11 Longitudinal and transverse column moment curvature response when the ground motions are applied separately in the longitudinal and transverse directions

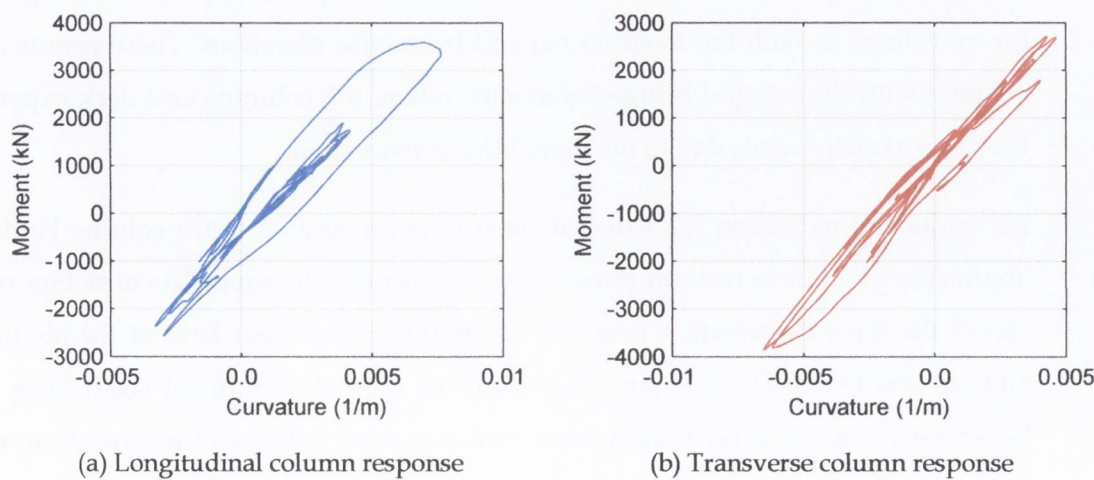


Figure 7.12 Longitudinal and transverse column moment curvature response when the ground motions are applied simultaneously in the longitudinal and transverse directions

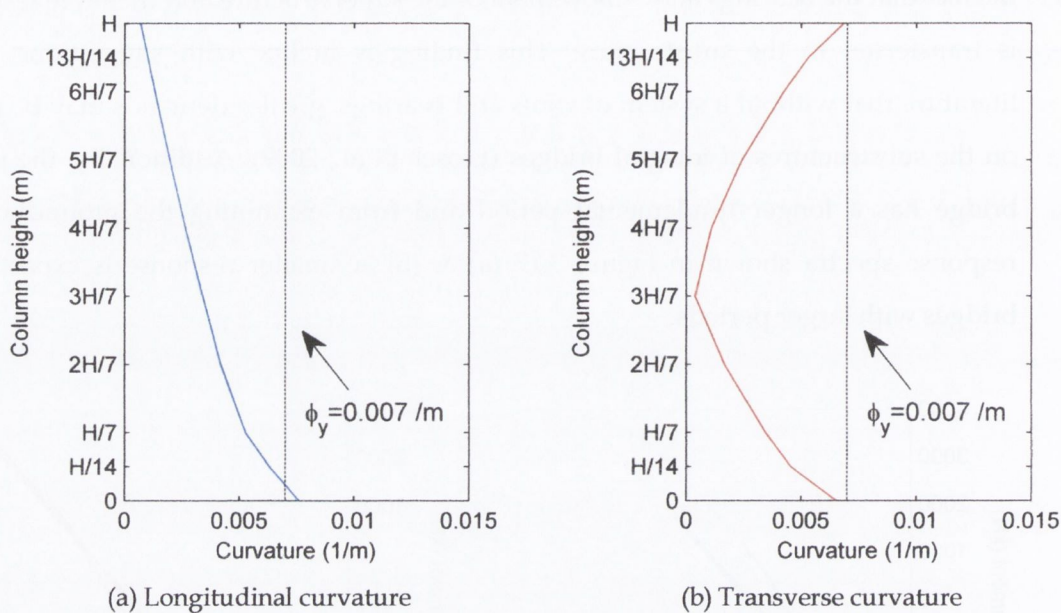


Figure 7.13 Variation of curvature over the height of the column

From Figure 7.14, which shows the displacement response of the deck and columns in the longitudinal direction, it can be seen that the deck experiences greater displacements than the column tops. This is due to the presence of elastomeric bearings, which allow for extra movement of the superstructure. The displacement response of the deck and column top in the transverse direction has not been included; however, the same trend is seen in the transverse direction since the elastomeric bearings allow for movement in both the longitudinal and transverse directions. These results are in contrast with the integral bridge behaviour, where the columns and deck experience the same displacements due to the monolithic construction.

As mentioned in Section 7.2, external shear keys are located at the column bents and abutments in order to restrain transverse movement of the superstructure. Figure 7.15 shows the force deformation response of the transverse shear keys at the abutments and column bents. From Figure 7.15, it can be seen the abutment shear keys resist larger forces than the bent shear keys, which is expected since they are designed to have greater capacities, recalling Section 7.2.3. It is also noted from the figures that in the negative direction, the shear keys completely fail as the force is reduced to zero in

both cases. In the positive direction however, it can be seen that the shear keys at the bent experience strength degradation but do not fail. The shear keys at the abutment do not experience any strength degradation in the positive direction.

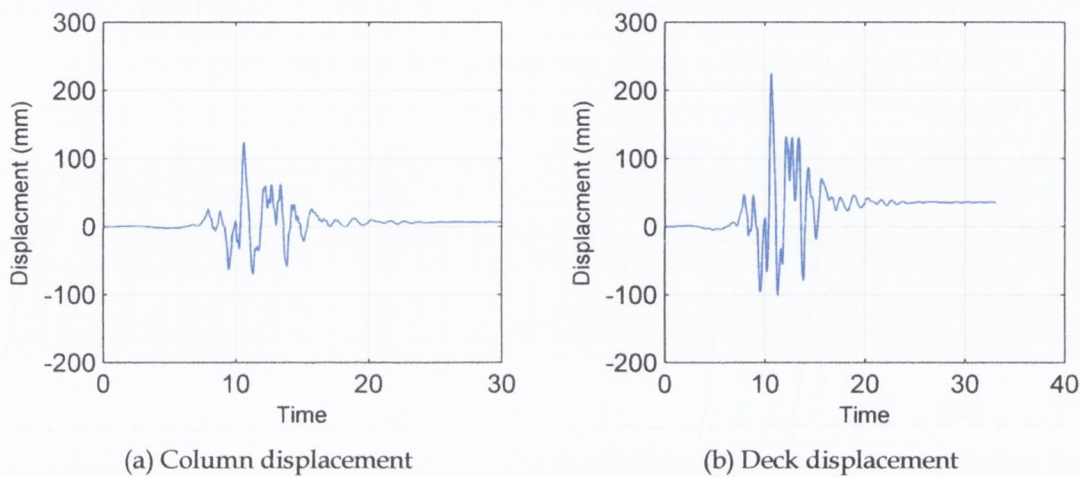


Figure 7.14 Column and deck displacement response in the longitudinal direction

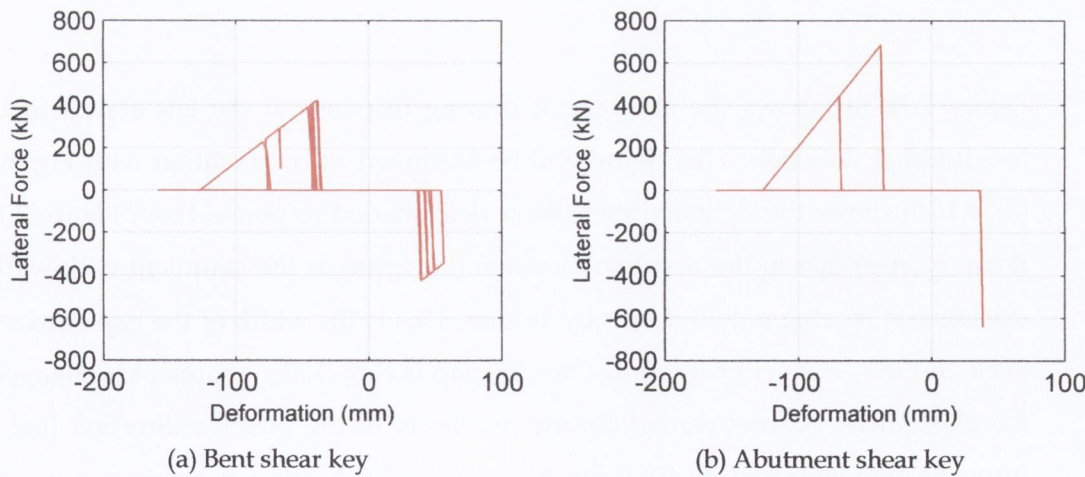


Figure 7.15 Shear key force deformation response at the abutment and column bent

Figure 7.16 shows the longitudinal and transverse response of one of the elastomeric bearings at the left abutment. First Figure 7.16 (b) is examined, which shows the

transverse bearing response, in combination with the transverse shear key response at the abutment shown in Figure 7.15 (b). From the figures it can be seen that the bearings and shear keys experience the same deformation in both the positive and negative directions as expected. For transverse movement, the elastomeric bearings control the response until the gap between the shear key and the girder is closed. After this point, the shear keys and elastomeric bearings work in parallel to resist transverse movement until the elastomeric bearings become plastic and no longer resist force.

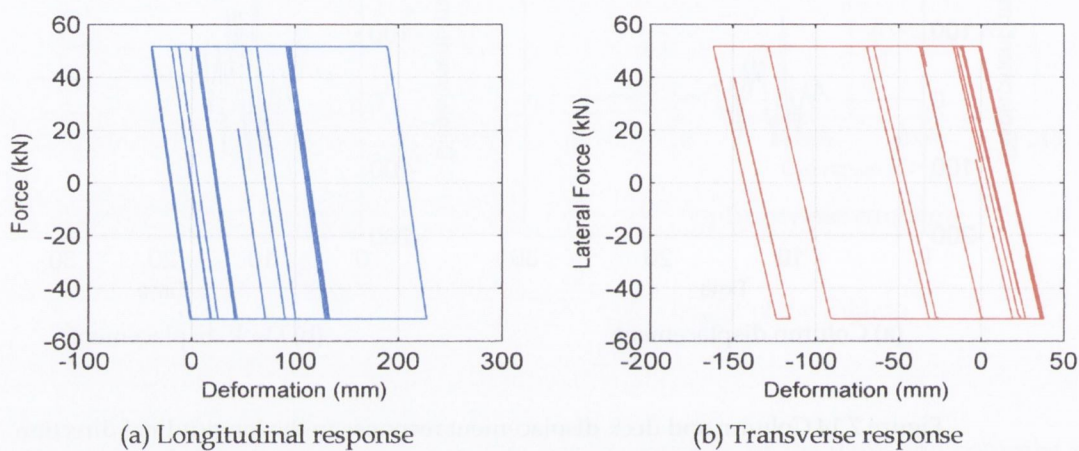


Figure 7.16 Elastomeric bearing response in the longitudinal and transverse directions

Figure 7.16 (a) shows the elastomeric bearing response at the left abutment in the longitudinal direction. This figure will be examined in combination with Figure 7.17 (a), which shows the right abutment force deformation response. From Figure 7.16 (a), it can be seen that in the negative direction (i.e. towards the abutment back wall), the elastomeric bearing only deforms by 40 mm. This is the width of the gap between the deck and the abutment backwall. Once the gap is closed, the abutment is engaged and the elastomeric bearing cannot deform anymore. In the positive direction (i.e. away from the abutment back wall), it can be seen that the bearing experiences a maximum displacement of 228 mm. This deformation is a result of the longitudinal movement of the opposite abutment which can be seen in Figure 7.17 (a). From this figure it can be seen that the abutment experiences a maximum deformation of 188 mm in the passive direction (i.e. towards the soil backfill). It can be seen that the maximum deformation

of the bearing is 40 mm greater than the displacement of the abutment since the gap between the abutment and deck must be closed before the abutment is engaged.

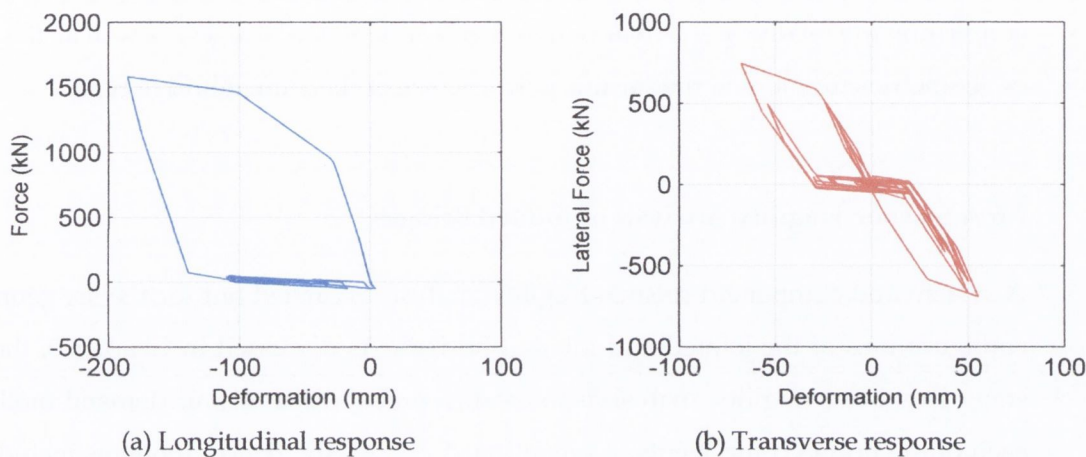


Figure 7.17 Abutment force deformation response in the longitudinal and transverse directions

As mentioned previously, the jointed bridge abutments follow the same force deformation behaviour as the integral bridge abutments. From Figure 7.17 (a), it can be seen that in contrast to the integral bridge abutment response, the jointed bridge abutments experience almost no deformation in the active direction. This is because the bearings allow for deck movement away from the abutment backwall. In the passive direction, the jointed abutments experience greater maximum displacements than the integral abutments. This is due to the effect of pounding between the deck and the abutment backwall. The jointed bridge abutment experiences a maximum deformation of 186 mm, whereas the integral abutment experiences a maximum deformation of 91 mm, which was seen in Figure 3.24 (a). In the transverse direction, the jointed abutments experience similar maximum displacements to the integral abutments, which can be seen by comparing Figure 7.17 (b) and 3.25 (b). For the jointed bridge, the abutment shear keys transfer the transverse forces from the deck to the abutments.

This section presented the component responses for a deterministic time history analysis. The purpose of this section was to gain insight into the overall seismic response of the bridge and to ensure that the bridge component responses are as expected, given the analytical models described in Sections 3.2 & 7.2. From the results presented in this section, it is thought that the component responses appear reasonable

and therefore a full probabilistic analysis of the jointed bridge is carried out in the next section. As mentioned previously, due to the variability of the problem, it is concluded that a probabilistic approach is required to facilitate appropriate conclusions concerning the relative performance of integral and jointed bridges, as well as the effect of geometric variation on the seismic performance of the jointed bridge type.

7.6 A Seismic Fragility Analysis of Jointed Bridges

A system and component seismic fragility analysis is carried out for the six geometric configurations of the jointed and integral bridges. As discussed in Chapter 5, the first step of a seismic fragility analysis is to create a probabilistic seismic demand model for each of the bridge components. As mentioned previously, the components included in the fragility analysis of the integral bridge are the columns and the abutments, whereas for the jointed bridge, the columns, abutments, bearings, shear keys and deck unseating are included in the fragility analysis. The parameters of the PSDMs for the integral and jointed bridge components are shown in Appendix C. For the jointed bridge columns, a linear regression model is used, as a bilinear regression model does not lead to an improvement in the R^2 value. A bilinear regression is used for the integral bridge columns as discussed in Section 2.6.

The key aims of this chapter are to investigate the effect of geometric variation on the seismic fragility of the jointed bridge as well as to compare the seismic performance of integral and jointed bridges. The latter aim is important since it is claimed in the literature that integral bridges perform better than alternative designs in earthquakes (Mistry, 2005, Paraschos and Amde, 2011), therefore, a direct comparison between the seismic performance of an integral and jointed bridge will provide insight into the change in seismic performance when integral construction is used. The following sections present system fragility curves comparing different geometric configurations of the jointed bridge, as well as component and system fragility curves for the integral and jointed bridges.

7.6.1 Effect of Geometric Variation

The six geometric configurations considered for the jointed bridge class include three different column heights and two bridge lengths. The two bridge lengths considered are 52 m and 69 m and the three column heights are taken as 6.9 m, 9.5 m and 12 m. Figure 7.18 shows the system fragility curves for the jointed bridge with varying column height at all four damage states. From the figure it can be seen that the effect of column height only has a very slight impact on the seismic performance of the jointed bridge.

From Figure 7.18 (a), it is seen that column height has a negligible impact on the slight damage fragility curves. Figure 7.18 (b) - (d) show a very slight decrease in fragility with column height. For example, for the 2% in 50 year earthquake, which has a PGA of 0.97g, the probability of exceeding moderate damage decreases by only 2% with a 5.1 m increase in column height. From the results presented in Section 6.1 regarding the effect of column height on the seismic performance of integral bridges, it was seen that column height has a greater impact on the seismic fragility of the integral bridge. The component fragility curves have not been included in this chapter as it was found that the column height does not significantly affect any of the individual component fragilities. The fragility estimates for the 2% and 10% in 50 year earthquakes for all the bridge components and geometric configurations can be found in Appendix C.

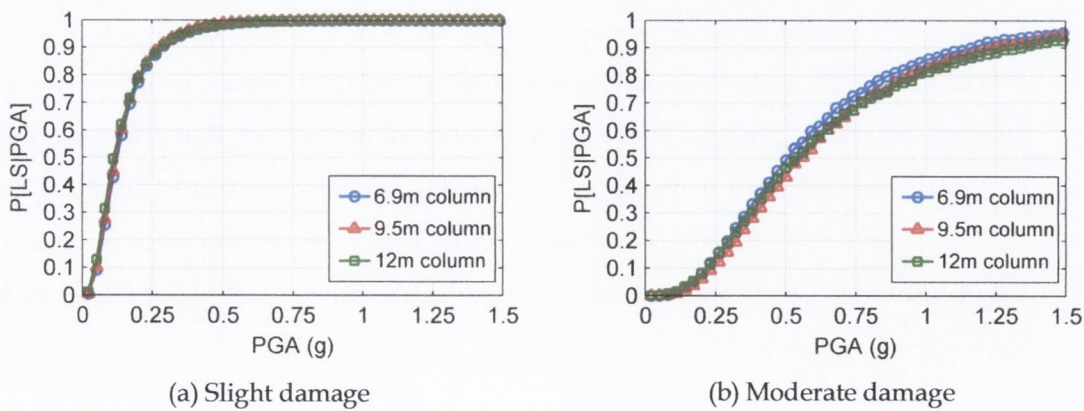


Figure 7.18 c & d System fragility curves for the jointed bridge with varying column height

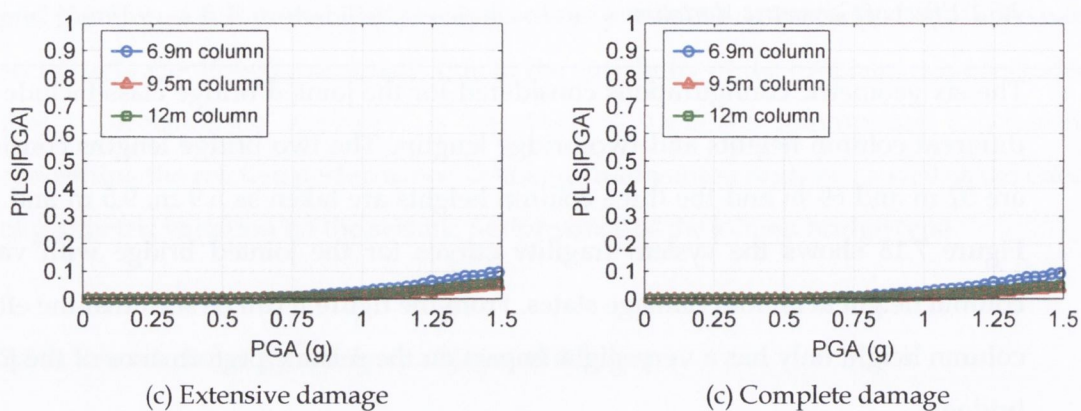


Figure 7.18 c & d System fragility curves for the jointed bridge with varying column height

Figure 7.19 shows the system fragility curves for the jointed bridge with varying length at all four damage states. From the figure it can be seen that increasing the bridge length from 52 m to 69 m results in a very slight decrease in the system fragility. For example, for the 2% in 50 year earthquake, the probability of exceeding moderate damage decreases by only 4% with a 17 m increase in total length. These results are in contrast with the results presented in Section 6.1 showing the effect of bridge length on the seismic fragility of the integral bridge, which showed that an increase in bridge length results in a significant increase in bridge fragility. By examining the component fragility curves it can be seen that the shear keys are the only components to be significantly affected by bridge length. The shear key fragility curves can be seen in Figure 7.20. For the 10% in 50 year earthquake, which has a PGA value of 0.5 g, the probability of the shear keys exceeding slight damage decreases by 16% with a 17 m increase in bridge length. This can be attributed to the fact that the longer bridge contains stronger shear keys, thus limiting transverse movement. Recalling Section 7.2.3, the shear key capacity is a function of the bent capacity and since the columns of the 69 m bridge are stronger than the 52 m bridge, the shear keys of the 69 m bridge have a greater capacity.

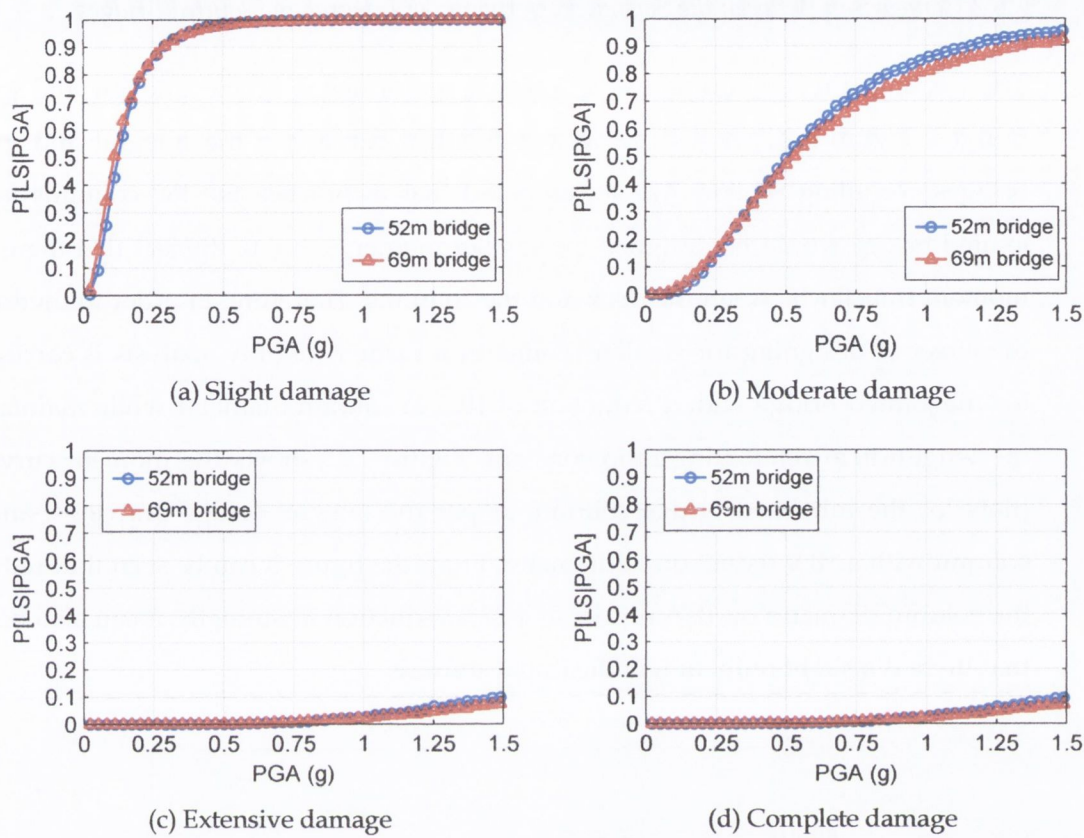


Figure 7.19 System fragility curves for the jointed bridge with varying length

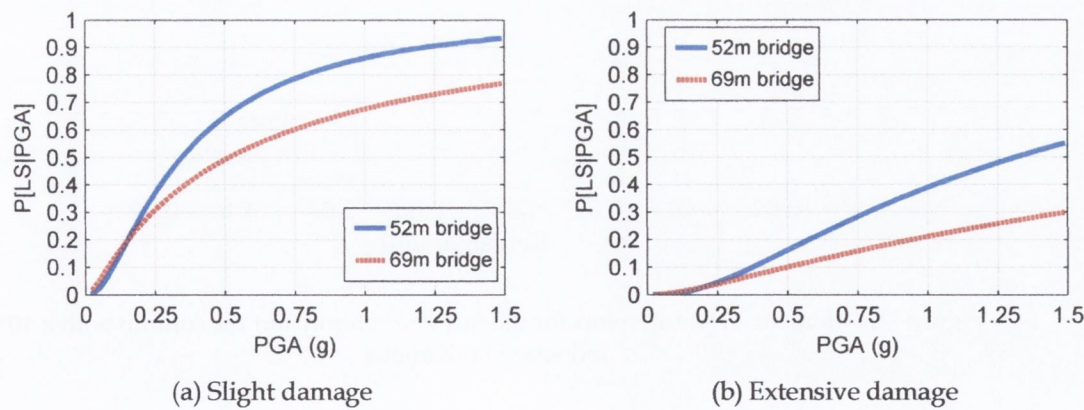


Figure 7.20 Shear key fragility curves

In summary, the results of this section show that neither column height, nor bridge length have a significant impact on the fragility of the jointed bridge.

7.6.2 Comparison Between the Seismic Performance of Integral and Jointed Bridges

This section directly compares the seismic performance of the integral and jointed bridges. Figure 7.22 shows the system fragility curves for the integral and jointed bridges. Recalling Section 7.2, it was noted that in practice the RC columns for the jointed bridge would be designed for smaller moments due to the fact that there is no moment transfer between the deck and the columns. Therefore, in order to investigate the effect of designing for smaller moments, a further fragility analysis is carried out for the jointed bridge with a reduction of 10% in column diameter while maintaining the longitudinal reinforcing ratio constant. Figure 7.21 shows the moment curvature plots for the full sized bridge column, as per the integral bridge drawings, and the column with a 10% reduction in diameter. From the figure it can be seen that reducing the column diameter by 10% results in a 27% reduction in strength. It can also be seen that there is a slight reduction in the initial stiffness.

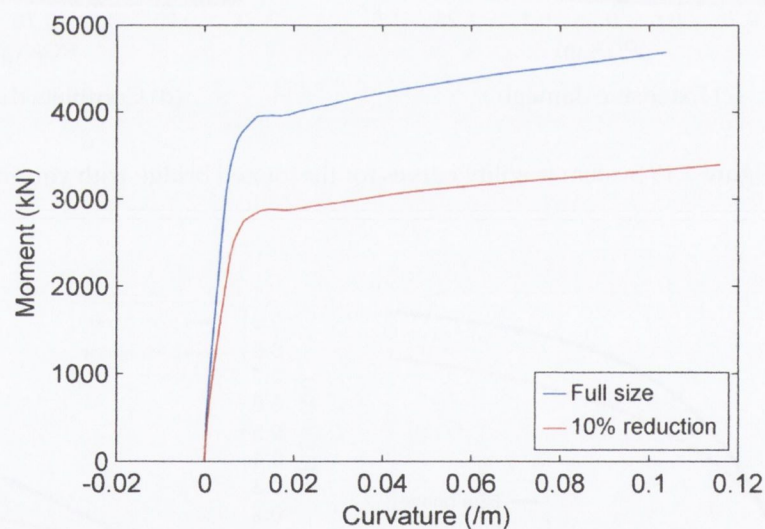


Figure 7.21 Moment curvature plots for the full size column and the column with a 10% reduction in diameter

Figure 7.22 shows the system fragility curves for the integral and jointed bridges as well as for the jointed bridge with a 10% reduction in column diameter. From the figures, it can be seen that the jointed bridge is significantly more vulnerable to seismic damage than the integral bridge. For example, for the 10% in 50 year earthquake, the probability of the jointed bridge exceeding slight damage is 48% greater than the

probability of the integral bridge exceeding slight damage. Furthermore, for the 2% in 50 year earthquake, the probability of exceeding moderate damage increases by 80% when a jointed design is used. From the figure it is also noted that there is very little difference between the fragility of the jointed bridge with the original columns and the jointed bridge with a 10% reduction in column diameter. The jointed bridge with a 10% reduction in column diameter is very slightly less vulnerable than the jointed bridge. This is due to the fact that although the column fragility increases with a decrease in diameter, the system fragility of the jointed bridge is not dominated by the column fragility. The system fragility of the jointed bridge is dominated by the bearings at the slight and moderate damage state and by deck unseating at the extensive and complete damage states. This would suggest that the relative movement between the deck and the columns is greater for the bridge with stiffer columns.

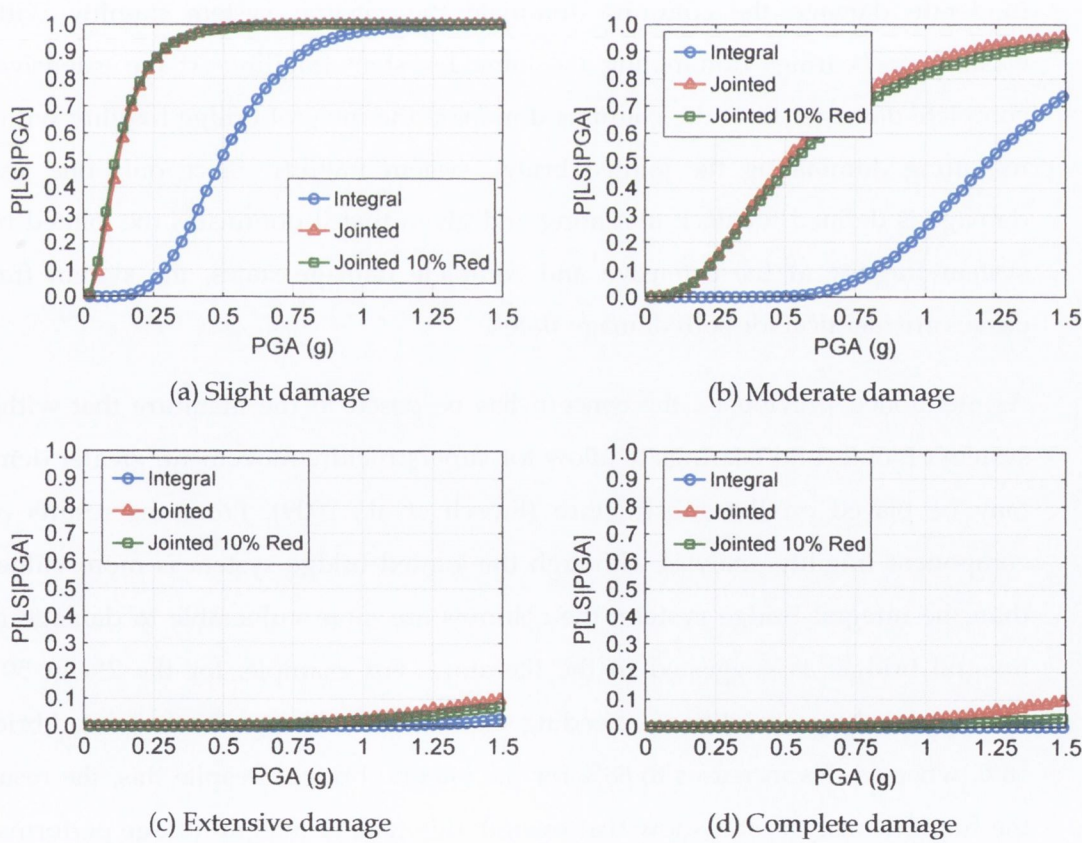


Figure 7.22 System fragility curves for the integral and jointed bridges

In order to gain more insight into the relative fragility of the various components of the integral and jointed bridges, the component fragility curves for both bridge types are shown in Figure 7.23. The components considered include the abutments in the active direction (Abt-A); abutments in the passive direction (Abt-P); abutments in the transverse direction (Abt-T); columns (Col); bearings (brg); shear keys (S-key) and deck unseating (Useat). The system (Sys) fragility curves are also shown. Since the reduction of the column diameter has little impact on the seismic fragility of the jointed bridge, it is not included in Figure 7.23.

At the slight damage state, it can be seen from Figure 7.23(a) that the columns and abutments in the active direction dominate the system fragility of the integral bridge. For the jointed bridge, the elastomeric bearings completely dominate the bridge fragility at the slight damage state which can be seen in Figure 7.23 (e). At the moderate damage the columns dominate the integral system fragility with the elastomeric bearings dominating the jointed system fragility. At the extensive and complete damage states the columns dominate the integral bridge fragility with deck unseating dominating the jointed bridge system fragility. Since only one level of damage is defined for deck unseating and given that it dominates the jointed bridge system fragility at the extensive and complete damage states, the system fragility curves are identical for both damage states.

As mentioned previously, the concern has been posed in the literature that without a system of joints and bearings to allow for superstructure movement, greater demands may be placed on the substructure (Frosch et al., 2009). From the results of the component fragility analysis, although the jointed bridge system is more vulnerable than the integral bridge system, the columns are more vulnerable to damage in the integral bridge, as suggested in the literature. For example, for the 2% in 50 year earthquake, the probability of exceeding slight column damage for the jointed bridge is 58%, whereas this increases to 88% for the integral bridge. Despite this, the results of the fragility analysis still show that overall, this type of integral bridge performs well under seismic loads, which is consistent with the general trend of adopting jointless bridges in high seismic zones. It is seen that there is a less than 1% probability of extensive damage occurring to the integral bridge for the 0.97g earthquake.

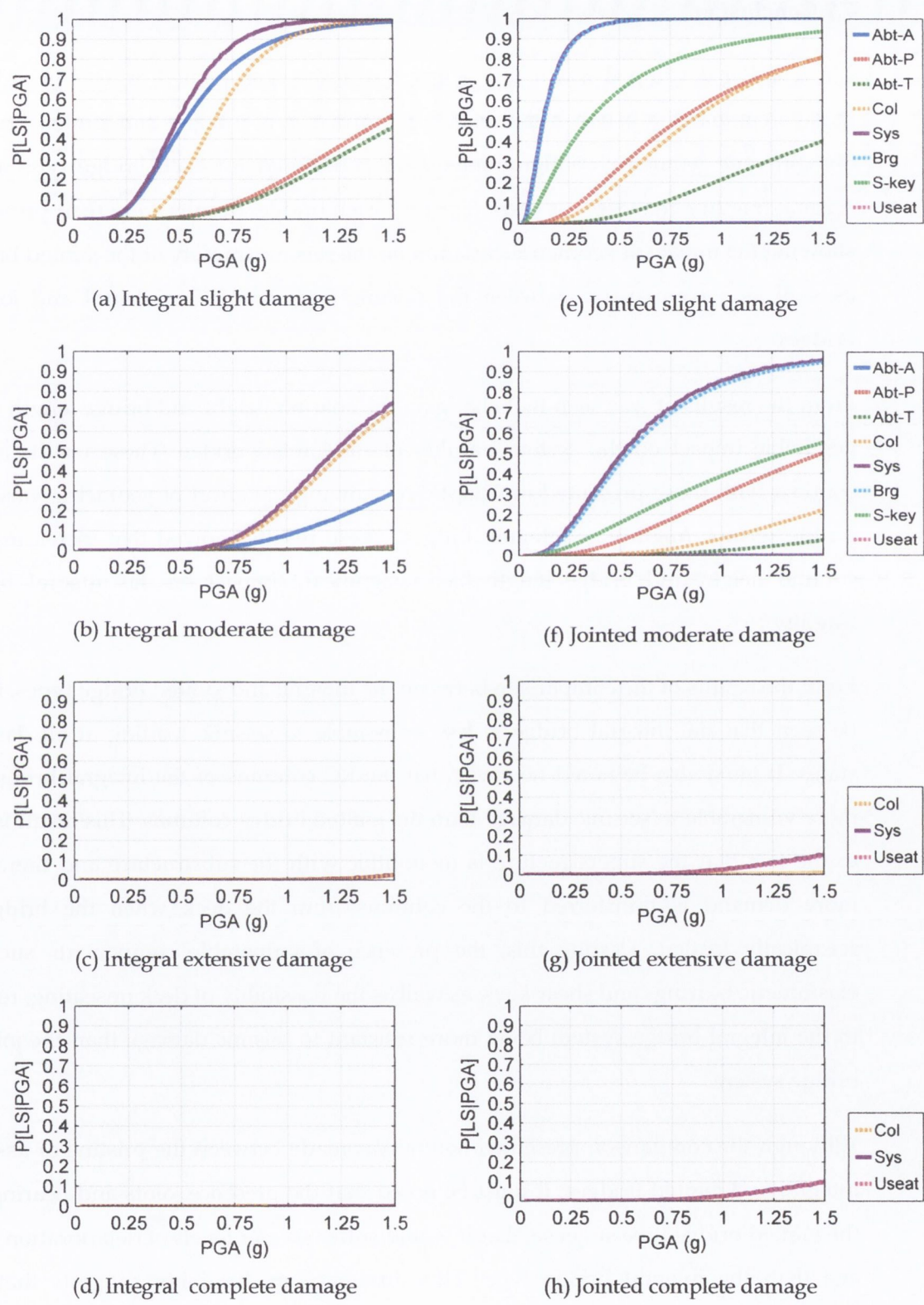


Figure 7.23 Component fragility curves for the integral and jointed bridges

7.7 Conclusion

This chapter developed models for a group of three span jointed bridges in order to make a comparison between the seismic performance of integral and jointed bridges. Six different geometric configurations were considered for each bridge type and a seismic fragility analysis was carried out for each bridge. Results were then presented showing the impact of geometric variation on the seismic fragility of the jointed bridge, as well as a comparison between the seismic fragility of the integral and jointed bridges.

From the results, it was seen that changing the column height and bridge length has a negligible impact on the system fragility of the jointed bridge. These results are in contrast with those presented in Chapter 6, showing the effect of geometric variation on the seismic fragility of integral bridges. These results showed that increasing the column height and bridge length has a significant impact on the integral bridge fragility.

From the results of the comparison between the integral and jointed bridge types, it can be seen that the integral bridge is less vulnerable to seismic loading at all damage states. It must also be noted however that the RC columns of the integral bridge are more vulnerable to seismic damage than the jointed bridge columns. This result is due to the fact that the superstructure is monolithic with the substructure and therefore, more demand is transferred to the columns from the deck when the bridge is seismically loaded. Despite this, the presence of vulnerable components such as elastomeric bearings and shear keys, as well as the possibility of deck unseating, results in the integral bridge system being more resistant to seismic damage than the jointed bridge system.

Although the comparison presented herein was made between the pristine or as-built integral and jointed bridges, it must be noted that the presence joints and bearings in the jointed bridge type suggests that it would suffer greater levels of deterioration with age than the integral bridge type. This difference in durability suggests that the inclusion of deterioration in the fragility analysis would result in the integral bridge performing better relative to the jointed bridge.

CHAPTER 8 - CONCLUSIONS

8.1 Summary and Conclusions

Recent earthquakes that have caused devastation to urban areas have created a need for earthquake damage mitigation in the form of retrofit projects and improved building codes. However, due to the finite nature of resources at government level, funds must be directed where there are needed most. During the recovery period of an earthquake, the uninterrupted operation of local infrastructure and more specifically bridges, is vital for rescue operations as well as the rehabilitation of an area. For this reason, over the past two decades, the probabilistic risk assessment of highway bridges has developed rapidly in order to prioritise bridges for retrofit and rehabilitation based on their seismic risk.

One approach to facilitate decision making for seismic bridge retrofit and rehabilitation is the use of seismic fragility curves. Seismic fragility curves are conditional probability statements that give the probability of a bridge exceeding defined damage states for a given ground motion intensity level. These tools are very useful and have been adopted in seismic risk assessment packages such as HAZUS (FEMA, 2005), REDARS (Werner et al., 2006) and MAEVIZ (MAE and NCSA, 2006). However, the current seismic risk assessment packages assume the bridges being assessed remain in their as-built or pristine condition. Although this approach may be acceptable for bridges that are relatively new, certain bridges that are approaching the end of their design service life and are exposed to chloride ion ingress may have experienced significant levels of deterioration. Since deterioration affects the main force resisting components of a bridge, it is reasonable to assume it increases the vulnerability of bridges subject to seismic hazard. Therefore, the use of fragility functions based on bridges in their pristine condition may lead to unreliable loss estimates following the current risk assessment methods.

The main aim of this thesis was to carry out a time dependent seismic risk assessment of integral bridges accounting for ageing of the bridge components. To this end, a group of three span integral bridges were chosen due to the popularity of this arrangement for highway overpasses. The integral bridge form was chosen as no work to date has been carried out on its ageing fragility despite its prevalence in bridge inventories worldwide. Integral bridges have become popular in recent years because they eliminate the need for maintenance associated with joints and bearings. Despite this growth in popularity, the design process still relies on a limited amount of technical references. Furthermore, only a very limited number of studies have compared the seismic fragility of bridges with integral and seat type abutments. This provided the second aim of this thesis: to directly compare the seismic performance of integral and jointed bridges in order to draw conclusions on the change in seismic performance when integral construction is used.

To address the first aim of this thesis, an approach for time dependent seismic fragility analysis was developed, such that the consideration of ageing on bridge capacity and seismic demands were jointly incorporated when assessing the probabilistic seismic performance of bridge components and systems. The approach was applied for a group of three span concrete integral bridges with varying column height and bridge length. This approach facilitated the comparison between various geometric configurations of the integral bridge under consideration. The components considered in the fragility analysis were the columns, abutments and deck, noting that although the deck is traditionally neglected in seismic fragility analyses, its relative vulnerability in the face of deterioration and heightened stresses from settlement was yet to be explored. Chloride induced corrosion of the reinforcing steel in the columns as well as the deck slab is considered. Additionally, foundation settlement of an intermediate pier was accounted for by choosing appropriate and discrete levels of vertical foundation movement and applying them to the corroded and uncorroded structures. The main conclusions from this part of the thesis are summarised below:

- For the integral bridge in its pristine condition, the abutments and columns dominate the bridge system fragility at the slight damage state. Beyond the slight damage state, the columns completely dominate the system fragility for

the pristine bridge. The fact that the seismic response is dominated by the columns highlights the importance of good column design in terms of both seismic detailing and durability for this integral bridge type.

- The columns become less vulnerable to seismic loading with an increase in height, however, the abutments become more vulnerable to seismic loading with an increase in column height. The system fragility curves therefore experience an increase in fragility when the abutments dominate the system fragility (i.e. at the slight damage state) and a decrease in fragility when the columns dominate the system fragility.
- Both the columns and the abutments experience an increase in fragility with an increase in bridge length due to the larger superstructure mass inducing greater demands on the bridge components. The system therefore experiences an overall increase in fragility with bridge length.
- The deck only begins to contribute to the bridge system fragility at the slight and moderate damage states after the equivalent of 75 years of corrosion. Beyond the moderate damage state, the columns dominate the system fragility at all levels of deterioration. An aim of this thesis is to determine whether accounting for the effects of ageing on the bridge structure would increase the deck vulnerability significantly enough to justify nonlinear modelling and its inclusion in a fragility analysis for this bridge type. It was concluded that although corrosion did increase the vulnerability of the deck, it was found that even after 100 years of corrosion, the deck had a negligible effect on fragility beyond the moderate damage state. It is therefore recommended that if bridge owners are concerned with bridge functionality, then the inclusion of the deck as a component in the fragility analysis is important. If life safety limits are of concern however, it is recommended that the inclusion of the deck in the fragility analysis may be unnecessary. Furthermore, the deck has a negligible effect on the system fragility at any damage state until after 75 years, therefore, its inclusion may be unnecessary unless corrosion levels are significant. It is

important to note here that vertical ground motions were not considered in this thesis and their inclusion may lead to greater deck demands providing an opportunity for future research.

- The columns and the deck experience the greatest increase in fragility with corrosion. An exception to this trend occurs at the moderate damage state, when the column fragility decreases slightly after 50 years of corrosion. This is due to the fact that the column capacity at the moderate damage state, defined as the curvature at which concrete spalling occurs, actually increases slightly with corrosion. Therefore, up to 50 years of chloride induced corrosion, the increase in capacity outweighs the increase in demand resulting in an overall decrease in the fragility. It was also seen that corrosion leads to a very slight decrease in abutment fragility.
- Overall, it is concluded that the system fragility increases with the inclusion of corrosion. The probability of the bridge exceeding slight damage for the 10% in 50 year earthquake in the Los Angeles region increases from 40% to 63% after 100 years. The probability of the bridge exceeding moderate damage for the 2% in 50 year earthquake in the Los Angeles region increases from 19% to 34% after 100 years. These results underscore the importance of accounting for ageing when assessing the seismic fragility of bridges, since a bridge that may otherwise be deemed safe can become vulnerable to seismic damage when exposed to corrosion deterioration.
- In terms of the effect of foundation settlement, it was concluded that it has the greatest impact on the deck fragility when coupled with chloride induced corrosion. Settlements on the order of the discrete levels adopted for this thesis increase the system fragility of the 100 year old bridge at the slight and moderate damage states. The system fragility at the extensive damage state is only slightly affected by foundation settlement whereas the system fragility at the complete damage state is unaffected by settlement. It was also found that settlement only affects the system fragility at the slight damage state when

corrosion is neglected. From these results it can be seen that when settlement is found to be an issue for integral bridges, it may only need to be included in the fragility analysis if corrosion of the deck slab is also found to be an issue.

The second aim of this thesis was to directly compare the seismic performance of integral and jointed bridges. In order facilitate this comparison, a group of three span jointed bridges were modelled with similar geometric properties to the integral bridges previously discussed. A seismic fragility analysis was carried out on the group of jointed bridges and the resulting fragility curves were compared directly with the integral bridge fragility curves. It is noted that for this comparison, the deck was modelled as an elastic beam and it was not included as a component in the fragility analysis. This approach was adopted since the previous conclusions suggest that the deck does not contribute to the system fragility of the pristine bridge and since a comparison is being made between the pristine integral and jointed bridges, the deck was deemed an unnecessary component in the analysis. The main conclusions from this part of the thesis are summarised below:

- It was found that changing the column height and bridge length has a negligible impact on the system fragility of the jointed bridge. These results are in contrast with those showing the effect of geometric variation on the seismic fragility of integral bridges. These results showed that increasing the column height and bridge length has a significant impact on the integral bridge fragility.
- In terms of the comparison between integral and jointed bridges, it was found that the integral bridge is less vulnerable to seismic loading at all damage states. It must also be noted however that the RC columns of the integral bridge are more vulnerable to seismic damage than the jointed bridge columns. This result is due to the fact that the superstructure is monolithic with the substructure and therefore more demand is transferred into the columns from the deck. Despite this, the presence of vulnerable components such as elastomeric bearings and shear keys, as well as the possibility of deck

unseating, results in the integral bridge system being more resistant to seismic damage than the jointed bridge system.

8.2 Recommendations for Future Work

Although the previous section has presented valuable insights into the ageing seismic performance of integral bridges, as well as into the relative seismic performance of integral and jointed bridges, there is potential for more work in the area. The main possibilities for future work are outlined below:

- Future studies could focus on the application of the proposed methodology for time dependent fragility analysis to different bridge forms. Furthermore, the influence of geometry on the seismic fragility of the integral bridge type under investigation may suggest that parameterized fragilities are warranted using metamodeling. A limited amount of research has been carried out to date using statistical learning techniques to develop metamodels, which efficiently approximate the complex and implicit relationship between predictor variables, such as bridge design and ground motion intensity parameters, and the predicted bridge component seismic responses (e.g., column and abutment deformations) (Ghosh et al., 2013).
- Additional deterioration mechanisms could be included in the time dependent seismic fragility analysis including pitting corrosion, a loss of bond strength between the concrete and the reinforcing steel, a loss of steel ductility and a loss of concrete cover strength.
- Including the above deterioration mechanisms in the corrosion model adds to the complexity of the model and may be unnecessary in terms of the impact on the aged seismic fragility of the bridge. For this reason, a sensitivity study could be carried out to reveal the influence of pitting corrosion, a reduction of steel ductility, a reduction of concrete cover strength and a loss of bond strength on the aged seismic fragility of the bridge. The results of this study would facilitate the development of an efficient approach for the time dependent seismic

fragility analysis of reinforced concrete bridges, subject to chloride induced corrosion, while maintaining a reasonable degree of complexity to give reliable results.

- Development of a Markov model of joint criterion of deterioration and seismic risk. This would involve the development of transition probabilities for different states of seismic vulnerability taking deterioration into account. Such a study would be extremely useful if incorporated into a bridge management system (BMS).

REFERENCES

- AASHTO. 1996. Standard Specification for Highway Bridges. American Association of State Highway and Transportation Officials. Washington, D.C.
- AASHTO. 2012. AASHTO LRFD Bridge Design Specifications. American Association of State Highway and Transportation Officials. Washington, D.C.
- Aboutaha, R. S. 2012. *Syracuse University Magazine: Research Snapshot* [Online]. Available:
<http://sumagazine.syr.edu/2013spring/orangematters/researchsnapshot.html>
[Accessed 6th July 2013].
- ACI. 2005. Building Code Requirements for Structural Concrete. ACI Committee 318. Farmington Hills, Mich.
- Akgul, F. & Frangopol, D. M. 2004. Lifetime performance analysis of existing prestressed concrete bridge superstructures. *Journal of Structural Engineering*, 130, 1889-1903.
- Akiyama, M., Frangopol, D. M. & Matsuzaki, H. 2011. Life-cycle reliability of RC bridge piers under seismic and airborne chloride hazards. *Earthquake Engineering and Structural Dynamics*, 40, 1671-1687.
- Akkari, M. & Duan, L. 2003. Nonlinear Analysis of Bridge Structures. *Bridge Engineering*. CRC Press.
- Alipour, A., Shafei, B. & Shinozuka, M. 2011. Performance evaluation of deteriorating highway bridges located in high seismic areas. *Journal of Bridge Engineering*, 16, 597-611.
- Almusallam, A. A. 2001. Effect of degree of corrosion on the properties of reinforcing steel bars. *Construction and Building Materials*, 15, 361-368.
- Andrade, C., Alonso, C., Garcia, D. & Rodriguez, J. 1991. Remaining lifetime of reinforced concrete structures: effect of corrosion in the mechanical properties of the steel. In: *Proceedings of the International Conference on Life Prediction of Corrodible Structures*, Cambridge, U.K.
- Angst, U., Elsener, B., Larsen, C. K. & Vennesland, Ø. 2009. Critical chloride content in reinforced concrete — A review. *Cement and Concrete Research*, 39, 1122-1138.
- Aquino, W. & Hawkins, N. M. 2007. Seismic retrofitting of corroded reinforced concrete columns using carbon composites. *ACI Structural Journal*, 104, 348-356.

- ASCE. 2013. *Report card for America's infrastructure* [Online]. Available: <http://www.infrastructurereportcard.org/a/#p/bridges/overview> [Accessed May 27th 2013].
- ATC. 1985. Earthquake damage evaluation data for California. *Report No: ATC-13*. Applied Technology Council, Redwood City, CA.
- ATC. 1991. Seismic vulnerability and impact of disruption of lifelines in the conterminous United States. *Report No: ATC-25*. Applied Technology Council, Redwood City, CA.
- ATC. 1996. Seismic evaluation and retrofit of concrete buildings. *Report No: ATC-40*. Applied Technology Council, Redwood City, CA.
- Aviram, A., Mackie, K. R. & Bozidar, S. 2008. Guidelines for nonlinear analysis of bridge structures in California. *Report No: PEER-2008/03*. Pacific Earthquake Engineering Research Center (PEER), University of California, Berkeley, CA.
- Baker, J. W. & Cornell, A. C. 2006. Vector-valued ground motion intensity measures for probabilistic seismic demand analysis. *Report No: PEER 2006/08*. Pacific Earthquake Engineering Research Center (PEER), University of California, Berkeley, CA.
- Banerjee, S. & Shinozuka, M. 2007. Nonlinear static procedure for seismic vulnerability assessment of bridges. *Computer-Aided Civil and Infrastructure Engineering*, 22, 293-305.
- Basoz, N. & Kiremidjian, A. S. 1997. Evaluation of bridge damage data from the Loma Prieta and Northridge CA earthquakes. *Report No: MCEER-98-0004*. MCEER, University at Buffalo, The State University of New York, Buffalo, New York.
- Bavirisetty, R., Vinayagamoorthy, M. & Duan, L. 2003. Dynamic Analysis. In: Chen, W.-F. & Duan, L. (eds.) *Bridge Engineering Handbook*. CRC Press.
- Bazant, Z. P. 1979. Physical model for steel corrosion in concrete sea structures - theory. *Journal of the Structural Division*, 105, 1137-1153.
- Berke, N. S. & Hicks, M. C. 1992. The life cycle of reinforced concrete decks and marine piles using laboratory diffusion and corrosion data. In: Chaker, V. (ed.) *Corrosion Forms and Control for Infrastructures (ASTM STP 1137)*. Philadelphia, PA: American Society for Testing and Materials.
- Berry, M. & Eberhard, M. O. 2003. Performance Models for Flexural Damage in Reinforced Concrete Columns. *Report No: Peer 2003/18*. Pacific Earthquake Engineering Research Center, University of California, Berkeley, CA.

- Bertolini, L., Elsener, B., Pedferri, P. & Polder, R. P. 2004. *Corrosion of steel in concrete*, Weinheim, Wiley-VCH.
- Broomfield, J. P. 1996. *Corrosion of Steel in Concrete: Understanding Investigation and Repair*
Taylor & Francis Inc.
- Cady, P. D. & Weyers, R. E. 1983. Chloride Penetration and the Deterioration of Concrete Bridge Decks. *Cement, Concrete and Aggregates*, 5, 81-87.
- Caltrans. 1990. Caltrans Structures Seismic Design References. California Department of Transportation. Sacramento, CA.
- Caltrans. 2004. Caltrans Seismic Design Criteria. California Department of Transportation. Sacramento, CA.
- Caltrans. 2006. Seismic Design Criteria. *Capacities of Structure Components* California Department of Transportation. Sacramento, CA.
- Celik, O. C. & Ellingwood, B. R. 2010. Seismic fragilities for non-ductile reinforced concrete frames – Role of aleatoric and epistemic uncertainties. *Structural Safety*, 32, 1-12.
- Choe, D.-E., Gardoni, P. & Rosowsky, D. 2010. Fragility increment functions for deteriorating reinforced concrete bridge columns. *Journal of Engineering Mechanics*, 136, 969-978.
- Choe, D.-E., Gardoni, P., Rosowsky, D. & Haukaas, T. 2008. Probabilistic capacity models and seismic fragility estimates for RC columns subject to corrosion. *Reliability Engineering and System Safety*, 93, 383-393.
- Choe, D.-E., Gardoni, P., Rosowsky, D. & Haukaas, T. 2009. Seismic fragility estimates for reinforced concrete bridges subject to corrosion. *Structural Safety*, 31, 275-283.
- Choi, E. 2002. *Seismic analysis and retrofit of Mid-America bridges*. PhD thesis, Georgia Intitute of Technology.
- Chopra, A. K. 2007. *Dynamics of Structures: Theory and Applications to Earthquake Engineering*, Prentice Hall, NY.
- Concast. 2012. *U Beams* [Online]. Available:
[http://www.concast.ie/sites/default/files/pdfs/Bridge%20Beams/u_beams.p](http://www.concast.ie/sites/default/files/pdfs/Bridge%20Beams/u_beams.pdf)
df [Accessed 17th April 2013].

- Concrete Construction. 2011. *Troubled bridge over water* [Online]. Available: <http://www.concreteconstruction.net/bridges/troubled-bridge-over-water.aspx> [Accessed 6th July 2013].
- Cornell, C. A., Jalayer, F., Hamburger, R. O. & Foutch, D. A. 2002. Probabilistic basis for 2000 SAC federal emergency management agency steel moment frame guidelines. *Journal of Structural Engineering*, 128, 526-533.
- Coronelli, D. & Gambarova, P. 2004. Structural assessment of corroded reinforced concrete beams: modeling guidelines. *Journal of Structural Engineering*, 130, 1214-1224.
- Cruz Noguez, C. & Saiidi, M. 2012. Shake-table studies of a four-span bridge model with advanced materials. *Journal of Structural Engineering*, 138, 183-192.
- Denton, S. & Tsionis, G. 2012. The evolution of Eurocodes for bridge design. Luxembourg: European Commission, Joint Research Centre, Institute for the Protection and Security of the Citizen.
- Der Kiureghian, A. 1981. A response spectrum method for random vibration analysis of mdf systems. *Earthquake Engineering & Structural Dynamics*, 9, 419-435.
- Der Kiureghian, A. 2002. Bayesian methods for seismic fragility assessment of lifeline components. *Technical Council on Lifeline Earthquake Engineering Monograph*, 61-77.
- Du, Y. G., Clark, L. A. & Chan, A. H. C. 2005a. Effect of corrosion on ductility of reinforcing bars. *Magazine of Concrete Research*, 57, 407-419.
- Du, Y. G., Clark, L. A. & Chan, A. H. C. 2005b. Residual capacity of corroded reinforcing bars. *Magazine of Concrete Research*, 57, 135-147.
- Dutta, A. 1999. *On energy based seismic analysis and design of highway bridges*. PhD thesis, University at Buffalo, The State University of New York.
- Ellingwood, B. & Hwang, H. 1985. Probabilistic descriptions of resistance of safety-related structures in nuclear plants. *Nuclear Engineering and Design*, 88, 169-78.
- Ellingwood, B. R. & Wen, Y.-K. 2005. Risk-benefit-based design decisions for low-probability/high consequence earthquake events in Mid-America. *Progress in Structural Engineering and Materials*, 7, 56-70.
- Elnashai, A. S., Borzi, B. & Vlachos, S. 2004. Deformation-based vulnerability functions for RC bridges. *Structural Engineering and Mechanics*, 17, 215-244.
- Elwood, K. J. & Eberhard, M. O. 2009. Effective stiffness of reinforced concrete columns. *ACI Structural Journal*, 106, 476-84.

- Enright, M. P. & Frangopol, D. M. 1998. Probabilistic analysis of resistance degradation of reinforced concrete bridge beams under corrosion. *Engineering Structures*, 20, 960-971.
- Eurocode 1. 2006. BS EN 1991-1-7: Actions on structures - Part 1-7: General actions - Accidental actions.
- Eurocode 8. 2004. BS EN 1998-1 Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings.
- Fajfar, P. 1999. Capacity spectrum method based on inelastic demand spectra. *Earthquake Engineering and Structural Dynamics*, 28, 979-993.
- Fang, C., Lundgren, K., Chen, L. & Zhu, C. 2004. Corrosion influence on bond in reinforced concrete. *Cement and Concrete Research*, 34, 2159-2167.
- Fang, J. Q., Li, Q. S., Jeary, A. P. & Liu, D. K. 1999. Damping of tall buildings: its evaluation and probabilistic characteristics. *The Structural Design of Tall Buildings*, 8, 145-153.
- FEMA. 1997. NEHRP Guidelines for the seismic rehabilitation of buildings. *Report No: FEMA-273*. Federal Emergency Management Agency, Washington, DC.
- FEMA. 2005. HAZUS-MH Software. Washington D.C.
- FHWA. 1985. Tolerable movement criteria for highway bridges. *Report No: FHWA/RD-85/107*. Federal Highway Administration, Washington, DC.
- FHWA. 1995. Seismic retrofitting manual for highway bridges *Report No: FHWA-RD-94-052*. Federal Highway Administration, Washington, DC.
- FHWA. 2006. Development of a comprehensive plan for a long-term bridge performance program. *Report No: DTFH61-05-RA-00105*. Federal Highway Administration, Washington, DC.
- Filippou, F. C., Popov, E. P. & Bertero, V. V. 1983. Effects of bond deterioration on hysteretic behavior of reinforced concrete joints. *Report No: EERC 83-19*. Pacific Earthquake Engineering Research Centre, University of California, Berkeley, CA.
- Freddi, F. 2012. *Local engineering demand parameters for seismic risk evaluation of low ductility reinforced concrete buildings*. PhD thesis, Università Politecnica delle Marche.
- Freeman, S. A. 2004. Review of the development of the capacity spectrum method. *ISET Journal of Earthquake Technology*, 14, 1-13.

- Freeman, S. A., Nicoletti, J. P. & Tyrrell, J. B. 1975. Evaluation of existing building for seismic risk - a case study of puget sound naval shipyard, Bremerton, Washington. In: *Proceedings of the 1st U.S. National Conference on Earthquake Engineering*, Berkeley, CA. 113-122.
- Freyssinet. 2013. *Structural Enhancement Specialists* [Online]. Available: <http://freyssinet.co.uk/tag/scotland/> [Accessed 6th July 2013].
- Frosch, R. J., Kreger, M. E. & Talbott, A. M. 2009. Earthquake Resistance of Integral Abutment Bridges. *Report No: FHWA/IN/JTRP-2008/11*. INDOT & Purdue University, Indiana, US.
- Gardoni, P., Der Kiureghian, A. & Mosalam, K. 2002. Probabilistic capacity models and fragility estimates for reinforced concrete columns based on experimental observations. *Journal of Engineering Mechanics*, 128, 1024-1038.
- Gardoni, P., Mosalam, K. M. & Kiureghian, A. D. 2003. Probabilistic seismic demand models and fragility estimates for RC bridges. *Journal of Earthquake Engineering*, 7, 79-106.
- Ghosh, J. & Padgett, J. E. 2010. Ageing considerations in the development of time-dependent seismic fragility curves. *Journal of Structural Engineering*, 136, 1497-1511.
- Ghosh, J. & Padgett, J. E. 2011. Probabilistic seismic loss assessment of ageing bridges using a component-level cost estimation approach. *Earthquake Engineering and Structural Dynamics*, 40, 1743-1761.
- Ghosh, J. & Padgett, J. E. 2012. Impact of multiple component deterioration and exposure conditions on seismic vulnerability of concrete bridges. *Earthquake and Structures*, 3, 649-673.
- Ghosh, J., Padgett, J. E. & Dueñas-Osorio, L. 2013. Surrogate modeling and failure surface visualization for efficient seismic vulnerability assessment of highway bridges. *Probabilistic Engineering Mechanics*, 34, 189-199.
- Gupta, A. & Krawinkler, H. 2000. Behavior of ductile SMRFs at various seismic hazard levels. *Journal of structural engineering*, 126, 98-107.
- Hassiotis, S., Khodair, Y., Roman, E. & Dehne, Y. 2006. Evaluation of integral abutments. *Report No: FHWA-NJ-2005-025*. New Jersey Department of Transportations & Federal Highway Administration.
- HETEK. 1997. A system for estimation of chloride ingress into concrete: theoretical background. *Report No: 83*. The Danish Road Directory.

- Hoffman, P. C. & Weyers, R. E. 1996. Probabilistic durability analysis of reinforced concrete bridge decks. In: *Proceedings of the 7th Speciality Conference on Probabilistic Mechanics and Structural Reliability, August 7, 1996 - August 9, 1996, Worcester, MA, US. American Society of Civil Engineers*, 290-293.
- Hoffman, P. C. & Weyers, R. W. 1993. Predicting critical chloride levels in concrete bridge decks. In: *Proceedings of the 6th International Conference on Structural Safety and Reliability, ICOSSAR 1993, August 9, 1993 - August 13, 1993, Innsbruck, Austria. A.A. Balkema*, 957-957.
- Howard Holland, F. 2012. *Snow and Ice Control Operations Manual: Bridge Maintenance Needs* [Online]. Available: http://onlinemanuals.txdot.gov/txdotmanuals/sic/bridge_maintenance_needs.htm [Accessed 6th July 2013].
- Hwang, H., Jernigan, J. B. & Lin, Y.-W. 2000. Evaluation of seismic damage to Memphis bridges and highway systems. *Journal of Bridge Engineering*, 5, 322-330.
- INDOT. 2010. *Indiana Bridge Inspection Manual*. Indiana Department of Transportation.
- Irfanoglu, A. 2012. *Response of Structures to Earthquake Ground motions* [Online]. Available: https://engineering.purdue.edu/~ce573/Documents/Response%20Spectrum_PPT.pdf [Accessed 28th December 2013].
- Kashani, M. M., Crewe, A. J. & Alexander, N. A. 2013. Nonlinear stress-strain behaviour of corrosion-damaged reinforcing bars including inelastic buckling. *Engineering Structures*, 48, 417-429.
- Kim, S.-H. & Shinozuka, M. 2004. Development of fragility curves of bridges retrofitted by column jacketing. *Probabilistic Engineering Mechanics*, 19, 105-112.
- Kowalsky, M. 2000. Deformation limit states for circular reinforced concrete bridge columns. *Journal of Structural Engineering*, 126, 869-878.
- Kowalsky, M. J. & Priestley, M. J. N. 2000. Improved analytical model for shear strength of circular reinforced concrete columns in seismic regions. *ACI Structural Journal*, 97, 388-396.
- Krawinkler, H., Medina, R. & Alavi, B. 2003. Seismic drift and ductility demands and their dependence on ground motions. *Engineering Structures*, 25, 637-653.
- Kumar, R., Gardoni, P. & Sanchez-Silva, M. 2009. Effect of cumulative seismic damage and corrosion on the life-cycle cost of reinforced concrete bridges. *Earthquake Engineering and Structural Dynamics*, 38, 887-905.

- Lee, H., Tomosawa, F. & Noguchi, T. 1996. Effect of rebar corrosion on the structural performance of single reinforced beams. In: Sjostrom, C. (ed.) *Durability of Building Materials and Components*, Vol. 1. London: E & FN Spon.
- Lemnitzer, A., Ahlberg, E. R., Nigbor, R. L., Shamsabadi, A., Wallace, J. W. & Stewart, J. P. 2009. Lateral performance of full-scale bridge abutment wall with granular backfill. *Journal of Geotechnical and Geoenvironmental Engineering*, 135, 506-514.
- Liu, T. & Weyers, R. W. 1998a. Modeling the dynamic corrosion process in chloride contaminated concrete structures. *Cement and Concrete Research*, 28, 365-379.
- Liu, Y. & Weyers, R. E. 1998b. Modeling the time-to-corrosion cracking in chloride contaminated reinforced concrete structures. *ACI Materials Journal*, 95, 675-681.
- Lomnitz, C. & Rosenblueth, E. 1976. *Seismic Risk and Engineering Decisions*, Amsterdam, NL, Elsevier Scientific Publishing Company.
- Lu, Y., Gu, X. & Guan, J. 2005. Probabilistic drift limits and performance evaluation of reinforced concrete columns. *Journal of Structural Engineering*, 131, 966-978.
- Ma, Y. & Deng, N. 2000. Deep Foundations. In: Chen, W. & Duan, L. (eds.) *Bridge Engineering Handbook*. CRC Press.
- Mackie, K. & Stojadinovic, B. 2001. Probabilistic seismic demand model for California highway bridges. *Journal of Bridge Engineering*, 6, 468-481.
- Mackie, K. & Stojadinovic, B. 2005a. Fragility basis for california highway overpass bridge seismic decision making. *Report No: 2005/02*. Pacific Earthquake Engineering Research Center, University of California, Berkeley, CA.
- Mackie, K. R., Cronin, K. J. & Nielson, B. G. 2011. Response sensitivity of highway bridges to randomly oriented multi-component earthquake excitation. *Journal of Earthquake Engineering*, 15, 850-876.
- Mackie, K. R. & Stojadinovic, B. 2003. Seismic demands for performance-based design of bridges. *Report No: 2003/16*. Pacific Earthquake Engineering Research Center, University of California, Berkeley, CA.
- Mackie, K. R. & Stojadinovic, B. 2005b. Comparison of incremental dynamic, cloud, and stripe methods for computing probabilistic seismic demand models. In: *2005 Structures Congress and the 2005 Forensic Engineering Symposium - Metropolis and Beyond, April 20, 2005 - April 24, 2005*, New York, NY, United states. American Society of Civil Engineers, 1835-1845.
- MAE & NCSA. 2006. Mid-America Earthquake Center Seismic Loss Assessment System.

- Mander, J. B. & Basoz, N. 1999. Seismic fragility curve theory for highway bridges. *Technical Council on Lifeline Earthquake Engineering Monograph*, 31-40.
- Mander, J. B., Kim, D. K., Chen, S. S. & Premus, G. J. 1996. Response of steel bridge bearings to the reversed cyclic loading. *Report No: NCEER 96-0014*. National Center for Earthquake Engineering Research.
- Mander, J. B., Priestley, M. J. N. & Park, R. 1988. Theoretical stress-strain model for confined concrete. *Journal of structural engineering*, 114, 1804-1826.
- Maroney, B. H. & Chai, Y. H. 1994. Seismic design and retrofitting of reinforced concrete bridges. In: *Proceedings of the 2nd International Workshop, Earthquake Commission of New Zealand, Queenstown, New Zealand*.
- McGee, R. 2000. Modelling of durability performance of Tasmanian bridges. In: Melchers, R. E. & Stewart, M. G. (eds.) *Applications of statistics and probability in civil engineering*. Rotterdam: Balkema.
- McKay, M. D., Beckman, R. J. & Conover, W. J. 1979. A Comparison of three methods for selecting values of input variables in the analysis of output from a computer code. *Technometrics*, 21, 239-245.
- McKenna, F. & Fenves, G. L. 2010. Open System for Earthquake Engineering Simulation (OpenSees), Version 2.0.0. Pacific Earthquake Engineering Research Centre (PEER) Berkeley, CA.
- Mechta, P. K. 1991. *Concrete in the marine environment*, Barking, U.K, Elsevier Applied Science.
- Megally, S. H., Silva, P. F. & Seible. 2002. Seismic response of sacrificial shear keys in bridge abutments. *Report No: SSRP-2001/23*. Department of Structural Engineering, University of California, San Diego, CA.
- Melchers, R. E. 1999. *Structural reliability analysis and prediction*, New York, Wiley.
- Menegotto, M. & Pinto, P. E. 1973. Method of analysis for cyclically loaded reinforced concrete plane frames including changes in geometry and non-elastic behavior of elements under combined normal force and bending. In: *Proceedings of the IABSE Symposium on Resistance and Ultimate Deformability of Structures Acted on by Well-Defined Repeated Loads*, Lisbon, Portugal.
- Mistry, V. C. 2005. Integral abutment and jointless bridges. In: *Proceedings of the FHWA Conference on Integral Abutment and Jointless Bridges*, Baltimore, Maryland, US.
- Mitoulis, S. A. & Tegos, I. A. 2011. Two new earthquake resistant integral abutments for medium to long span bridges. *Structural Engineering International: Journal of*

- the International Association for Bridge and Structural Engineering (IABSE)*, 21, 157-161.
- Montjo, L. A. 2007. Cumbia Octave: Section and Member Response of RC Members of Circular Section. North Carolina State University, NC. .
- Morinaga, S. 1996. Remaining life of reinforced concrete structures after corrosion cracking. In: Sjostrom, C. (ed.) *Durability of Building Materials and Components*, Vol. 1. London: E & FN Spon.
- Moschonas, I., Kappos, A., Panetsos, P., Papadopoulos, V., Makarios, T. & Thanopoulos, P. 2009. Seismic fragility curves for greek bridges: methodology and case studies. *Bulletin of Earthquake Engineering*, 7, 439-68.
- Mosqueda, G. 2010. *Preliminary Damage Reports from the Chile Earthquake: February 27, 2010* [Online]. Available: <http://mceer.buffalo.edu/research/reconnaissance/Chile2-27-10/damage-reports.asp> [Accessed 18th September 2013].
- MTD 5-1. 1992. Memo to Designers (5-1). California Department of Transportation, Sacramento, CA.
- MTO. 2013. *Pulling the teeth on old bridge structures* [Online]. Available: <http://www.mto.gov.on.ca/english/transtek/roadtalk/rt17-1/> [Accessed 6th July 2013].
- Muthukumar, S. 2003. *A contact element approach with hysteresis damping for the analysis and design of pounding in bridges*. PhD thesis, Georgia Institute of Technology.
- Nielson, B. G. 2005. *Analytical fragility curves for highway bridges in moderate seismic zones*. PhD thesis, Georgia Institute of Technology.
- Nielson, B. G. & Mackie, K. R. 2009. Tracking uncertainties from component level to system level fragility analyses through simulation. In: *Proceedings of 10th International Conference on Structural Safety and Reliability*, Osaka, Japan.
- Nowak, A. S., Yamani, A. S. & Tabsh, S. W. 1994. Probabilistic models for resistance of concrete bridge girders. *ACI Structural Journal*, 91, 269-276.
- O'Brien, E. J. & Keogh, D. L. 1999. *Bridge Deck Analysis*, London, Taylor & Francis.
- ODOT. 2013. *ODOT On-line Bridge Maintenance Manual Preventive Maintenance/Repair Guidelines for Bridges and Culverts* [Online]. Available: <http://www.dot.state.oh.us/divisions/engineering/structures/bridge%20operations%20and%20maintenance/preventivemaintenancemanual/Pages/default.aspx> [Accessed 17th September 2013].

- Padgett, J. E. 2007. *Seismic vulnerability assessment of retrofitted bridges using probabilistic methods*. PhD thesis, Georgia Institute of Technology.
- Padgett, J. E. & DesRoches, R. 2007. Bridge functionality relationships for improved seismic risk assessment of transportation networks. *Earthquake Spectra*, 23, 115-30.
- Padgett, J. E., Nielson, B. G. & DesRoches, R. 2008. Selection of optimal intensity measures in probabilistic seismic demand models of highway bridge portfolios. *Earthquake Engineering and Structural Dynamics*, 37, 711-725.
- Palsson, R. & Mirza, M. S. 2002. Mechanical response of corroded steel reinforcement of abandoned concrete bridge. *ACI Structural Journal*, 99, 157-162.
- Paraschos, A. & Amde, A. M. 2011. A survey on the status of use, problems and costs associated with integral abutment bridges. Available: <http://www.betterroads.com/integral-abutment-bridges>.
- PEER. Strong Motion Database. [Online] Available: <http://peer.berkeley.edu/smcat>
- Ramamoorthy, S. K., Gardoni, P. & Bracci, J. M. 2006. Probabilistic demand models and fragility curves for reinforced concrete frames. *Journal of Structural Engineering*, 132, 1563-1572.
- Ramanathan, K., DesRoches, R. & Padgett, J. E. 2010. Analytical fragility curves for multispan continuous steel girder bridges in moderate seismic zones. *Transportation Research Record*, 173-182.
- Ramanathan, K. N. 2012. *Next generation seismic fragility curves for California bridges incorporating the evolution in seismic design philosophy*. PhD thesis, Georgia Institute of Technology.
- Reza, S. M. 2012. *Seismic performance of multi-span RC bridge with irregular column heights*. M.A.Sc thesis, University of British Columbia.
- Romstad, K., Kutter, B., Maroney, B. H., Vanderbilt, E., Griggs, M. & Chai, Y. H. 1995. Experimental measurements of bridge abutment behavior. Report No: UCD-STR-95-1. Dept. of Civil and Environmental Engineering, University of California, Davis, CA.
- Rosenblueth, E. 1951. *A Basis for Aseismic Design*. PhD thesis, University of Illinois.
- Saassouh, B. & Lounis, Z. 2012. Probabilistic modeling of chloride-induced corrosion in concrete structures using first- and second-order reliability methods. *Cement and Concrete Composites*, 34, 1082-1093.

SAC. Los Angeles Project Database. [Online]
Available:http://nisee.berkeley.edu/data/strong_motion/

Saifullah, M. 1994. *Effect of Reinforcement Corrosion on Bond Strength in Reinforced Concrete*. PhD thesis, The University of Birmingham.

Saraswathy, V. & Song, H.-W. 2007. Evaluation of corrosion resistance of Portland pozzolana cement and fly ash blended cements in pre-cracked reinforced concrete slabs under accelerated testing conditions. *Materials Chemistry and Physics*, 104, 356-361.

Schrage, I. 1981. Anchoring of Bearings by Friction. In: *Joint Sealing and Bearing Systems for Concrete Structures: World Congress on Joints and Bearings, Vol 1*. Niagara Falls, NY: American Concrete Institute.

Scott, B. D., Park, R. & Priestley, M. J. N. 1982. Stress-strain behaviour of concrete confined by overlapping hoops at low and high strain rates. *Journal of the American Concrete Institute*, 79, 13-27.

Seraj, S. M., Kotsovos, M. D. & Pavlovic, M. N. 1992. Non-linear finite-element analysis of prestressed concrete members. *Proceedings of the Institution of Civil Engineers: Structures and Buildings*, 94, 403-418.

Sergi, G. 1986. *Corrosion of steel in concrete: cement matrix variables* PhD thesis, University of Aston.

Shamsabadi, A., Khalili-Tehrani, P., Stewart, J. P. & Taciroglu, E. 2010. Validated simulation models for lateral response of bridge abutments with typical backfills. *Journal of Bridge Engineering*, 15, 302-311.

Shamsabadi, A., Law, L. & Martin, G. 2006. Comparison of direct method versus substructure method for seismic analyses of a skewed bridge. In: *5th National Seismic Conference for Bridges and Highways*, San Francisco, CA, US.

Shamsabadi, A., Rollins, K. M. & Kapuskar, M. 2007. Nonlinear soil-abutment-bridge structure interaction for seismic performance-based design. *Journal of Geotechnical and Geoenvironmental Engineering*, 133, 707-720.

Shinozuka, M., Feng, M., Lee, J. & Naganuma, T. 2000a. Statistical analysis of fragility curves. *Journal of Engineering Mechanics*, 126, 1224-1231.

Shinozuka, M., Feng, M. Q., Kim, H.-K. & Kim, S.-H. 2000b. Nonlinear static procedure for fragility curve development. *Journal of Engineering Mechanics*, 126, 1287-1296.

- Shinozuka, M., Feng, M. Q., Kim, H.-K., Uzawa, T. & Ueda, T. 2003. Statistical analysis of fragility curves. *Report No: MCEER-03-0002*. MCEER, University at Buffalo, The State University of New York, Buffalo, NY.
- Simon, J., Bracci, J. M. & Gardoni, P. 2010. Seismic response and fragility of deteriorated reinforced concrete bridges. *Journal of Structural Engineering*, 136, 1273-1281.
- Sritharan, S., Werff, J. V., Abendroth, R. E., Wassef, W. G. & Greimann, L. F. 2005. Seismic behavior of a concrete/steel integral bridge pier system. *Journal of Structural Engineering*, 131, 1083-1094.
- Steinbrugge, K. V. The Earthquake Engineering Online Archive. [Online] Available:<http://nisee.berkeley.edu/elibrary/>
- Stewart, J. P., Taciroglu, E., Wallace, J. W., Ahlberg, E. R., Lemnitzer, A., Rha, C., Tehrani, P., Keowen, S., Nigbor, R. L. & Salamanca, A. 2007. Full scale cyclic testing of foundation support systems for highway bridges. Part II: abutment backwalls.
- Stewart, M. & Rosowsky, D. 1998. Structural Safety and Serviceability of Concrete Bridges Subject to Corrosion. *Journal of Infrastructure Systems*, 4, 146-155.
- Stewart, M. G., Wang, X. & Nguyen, M. N. 2011. Climate change impact and risks of concrete infrastructure deterioration. *Engineering Structures*, 33, 1326-1337.
- Stewart, M. G., Wang, X. & Nguyen, M. N. 2012. Climate change adaptation for corrosion control of concrete infrastructure. *Structural Safety*, 35, 29-39.
- The Highways Agency. 2001. DMRB Volume 1 Section 3 BD 57/01, *Design for Durability* The Highways Agency. The Stationery Office, Norwich.
- The Mathworks Inc. 2010. MATLAB 7.1. Natick, MA.
- Thoft-Christensen, P., Jensen, F. M., Middleton, C. R. & Blackmore, A. 1996. Assessment of the reliability of concrete slab bridges. *Structural Reliability Theory*, R9619.
- Tilly, G. 2007. The durability of repaired concrete structures. In: *Proceedings of the International Association for Bridges and Structural Engineering Symposium (IABSE)*, Weimar, Germany.
- Transportation For America. 2013. *The Fix We're In For: The State of Our Nation's Bridges* [Online]. Available: <http://t4america.org/docs/bridgereport2013/2013BridgeReport.pdf> [Accessed September 15th 2013].

- Tseng, W. S. & Penzien, J. 2000. Soil-foundation-structure interaction. In: Chen, W. F. & Duan, L. (eds.) *Bridge Engineering Handbook*. CRC Press.
- Tutti, K. 1979. Service life of structures with regard to corrosion of embedded steel. *Metal Construction*, 1, 293-301.
- USGS. Hazards: Seismic Hazard Maps and Data. Earthquake Hazards Program. [Online] Available: <http://earthquake.usgs.gov/hazards/>
- Val, D. V. & Stewart, M. G. 2003. Life-cycle cost analysis of reinforced concrete structures in marine environments. *Structural Safety*, 25, 343-362.
- Vamvatsikos, D. & Allin Cornell, C. 2002. Incremental dynamic analysis. *Earthquake Engineering and Structural Dynamics*, 31, 491-514.
- Vector Corrosion Technologies. 2013. *Corrosion Management* [Online]. Available: <http://www.vector-corrosion.com/education-2/management/> [Accessed July 5th 2013].
- Vermeer, P. A. & Brinkgreve, R. B. J. 1998. PLAXIS: Finite-element code for soil and rock analyses (version 7.1). Balkema, Rotterdam, The Netherlands.
- Vu, K. A. T. & Stewart, M. G. 2000. Structural reliability of concrete bridges including improved chloride-induced corrosion models. *Structural Safety*, 22, 313-333.
- Wang, X., Stewart, M. G. & Nguyen, M. 2012. Impact of climate change on corrosion and damage to concrete infrastructure in Australia. *Climatic Change*, 110, 941-957.
- Werner, S. D., Taylor, C. E., Cho, S., Lavoie, J.-P., Huyuck, C., Eitzel, C., Chung, H. & Eguchi, R. T. 2006. REDARS 2: Methodology and software for seismic risk analysis of highway systems. *Report No: MCEER-06-SP08*. Multidisciplinary Center for Earthquake Engineering Research, Buffalo, New York.
- Whiting, D., Stejskal, B. & Nagi, M. 1990. Condition of Prestressed concrete bridge components: Technology review and field surveys. *Report No: FHWA-RD-93-037*. Federal Highway Administration.
- Whitman, R. V., Biggs, J. M., Cornell, C. A., Brennan, J. E., de Neufville, R. L. & Vanmarcke, E. H. 1975. Seismic design decision analysis. *Journal of Structural Division*, 101, 1067-1084.
- WSDOT. 2013. Geotechnical Design Manual. *Foundation Design* Washington State Department of Transportation. Tumwater, WA.

- Yamazaki, F., Hamada, T., Motoyama, H. & Yamauchi, H. 1999. Earthquake damage assessment of expressway bridges in Japan. *Technical Council on Lifeline Earthquake Engineering Monograph*, 16, 361-370.
- Yokozeki, K., Motohashi, K., Tsutsumi, T. & Okada, I. K. 1997. A rational model to predict the service life of RC structures in marine environment. In: *Proceedings of the 4th ACI International Conference on Durability of Concrete*. American Concrete Institute, 777-799.
- Yoon, I.-S., Copuroglu, O. & Park, K.-B. 2007. Effect of global climatic change on carbonation progress of concrete. *Atmospheric Environment*, 41, 7274-7285.
- Yu, O., Allen, D. L. & Drnevich, V. P. 1991. Seismic vulnerability assessment of bridges on earthquake priority routes in Western Kentucky. In: *Proceedings of the 3rd US Conference on Lifeline Earthquake Engineering, August 22, 1991 - August 23, 1991*, Los Angeles, CA. American Society of Civil Engineers, 817-826.
- Zakeri, B., Padgett, J. & Amiri, G. Fragility Analysis of Skewed Single Frame Concrete Box Girder Bridges. *Journal of Performance of Constructed Facilities*, 0, null.
- Zhang, P. S., Lu, M. & Li, X. Y. 1995. The mechanical behaviour of corroded bar. *Journal of Industrial Buildings*, 25, 41-44.
- Zhong, J., Gardoni, P., Rosowsky, D. & Haukaas, T. 2008. Probabilistic seismic demand models and fragility estimates for reinforced concrete bridges with two-column bents. *Journal of Engineering Mechanics*, 134, 495-504.
- Zhu, W., François, R., Coronelli, D. & Cleland, D. 2013. Effect of corrosion of reinforcement on the mechanical behaviour of highly corroded RC beams. *Engineering Structures*, 56, 544-554.

APPENDIX - A PROBABILISTIC SEISMIC DEMAND MODELS AND CORRELATION MATRICES FOR INTEGRAL BRIDGES

ab_A :	Abutment deformation demand in active direction
ab_P :	Abutment deformation demand in passive direction
ab_T :	Abutment deformation demand in transverse direction
φ_{slight} :	Column curvature demand to capacity ratio at slight damage state
$\varphi_{moderate}$:	Column curvature demand to capacity ratio at slight damage state
$\varphi_{extensive}$:	Column curvature demand to capacity ratio at slight damage state
$\varphi_{complete}$:	Column curvature demand to capacity ratio at slight damage state
$Deck_1$:	Deck bending moment demand to capacity ratio at right end of left span
$Deck_2$:	Deck bending moment demand to capacity ratio at left end of centre span
$Deck_3$:	Deck bending moment demand to capacity ratio at centre span
$Deck_4$:	Deck bending moment demand to capacity ratio at right end of centre span
$Deck_5$:	Deck bending moment demand to capacity ratio at left end of right span

Table A-1 PSDM: Pristine 52 m bridge 6.9 m column

$\ln(S_D) = \ln(a) + b \cdot \ln(PGA)$								
Linear Regression Lines								
Response	$\ln(a)$	b	$\beta_{D IM}$	R^2				
$\ln(ab_A)$	3.692	0.952	0.336	0.85				
$\ln(ab_P)$	4.067	1.615	0.641	0.81				
$\ln(ab_T)$	3.774	1.054	0.337	0.88				
Bilinear Regression Lines								
1 st Regression Line					2 nd Regression Line			
	$\ln(a)$	b	$\beta_{D IM}$	2 nd start pnt	$\ln(a)$	b	$\beta_{D IM}$	R^2
$\ln(\varphi_{slight})$	-0.429	0.629	0.188	0.332	0.489	1.437	0.347	0.91
$\ln(\varphi_{moderate})$	-1.440	0.639	0.186	0.354	-0.451	1.535	0.38	0.91
$\ln(\varphi_{extensive})$	-2.423	0.637	0.185	0.332	-1.569	1.383	0.37	0.90
$\ln(\varphi_{complete})$	-2.909	0.637	0.19	0.332	-2.064	1.377	0.373	0.90
$\ln(Deck_1)$	14.356	0.105	0.049	0.389	14.719	0.487	0.137	0.86
$\ln(Deck_2)$	14.485	0.099	0.037	0.354	14.766	0.364	0.096	0.89
$\ln(Deck_3)$	13.71	0.061	0.032	0.332	14.024	0.331	0.078	0.89
$\ln(Deck_4)$	14.494	0.102	0.041	0.423	14.786	0.413	0.108	0.88
$\ln(Deck_5)$	14.344	0.102	0.043	0.382	14.681	0.439	0.119	0.87

Table A-2 Correlation Matrix: Pristine 52 m bridge 6.9 m column

	$\ln(ab_A)$	$\ln(ab_P)$	$\ln(ab_T)$	$\ln(\varphi_s)$	$\ln(\varphi_m)$	$\ln(\varphi_c)$	$\ln(D_1)$	$\ln(D_2)$	$\ln(D_3)$	$\ln(D_4)$	$\ln(D_5)$
$\ln(ab_A)$	1.000	0.701	0.904	0.946	0.949	0.946	0.937	0.944	0.938	0.936	0.869
$\ln(ab_P)$	0.701	1.000	0.653	0.621	0.612	0.610	0.535	0.561	0.558	0.533	0.498
$\ln(ab_T)$	0.904	0.653	1.000	0.955	0.957	0.954	0.864	0.868	0.870	0.862	0.888
$\ln(\varphi_s)$	0.946	0.621	0.955	1.000	0.995	0.990	0.938	0.935	0.935	0.937	0.925
$\ln(\varphi_m)$	0.949	0.612	0.957	0.995	1.000	0.994	0.938	0.937	0.937	0.938	0.927
$\ln(\varphi_c)$	0.946	0.610	0.954	0.990	0.994	1.000	0.937	0.939	0.936	0.939	0.921
$\ln(D_1)$	0.937	0.535	0.864	0.938	0.938	0.937	1.000	0.985	0.995	0.988	0.955
$\ln(D_2)$	0.944	0.561	0.868	0.935	0.937	0.939	0.985	1.000	0.990	0.994	0.948
$\ln(D_3)$	0.938	0.558	0.870	0.935	0.937	0.936	0.995	0.990	1.000	0.984	0.960
$\ln(D_4)$	0.936	0.533	0.862	0.937	0.938	0.939	0.988	0.994	0.984	1.000	0.946
$\ln(D_5)$	0.869	0.498	0.888	0.925	0.927	0.921	0.955	0.948	0.960	0.946	1.000

Table A-3 50 year old 52 m bridge 6.9 m column

$\ln(S_D) = \ln(a) + b \cdot \ln(PGA)$								
Linear Regression Lines								
Response	$\ln(a)$	b			$\beta_{D IM}$			R^2
$\ln(ab_A)$	3.634	0.920			0.321			0.86
$\ln(ab_P)$	3.919	1.570			0.660			0.76
$\ln(ab_T)$	3.752	1.046			0.332			0.88
Bilinear Regression Lines								
	1 st Regression Line				2 nd Regression Line			
	$\ln(a)$	b	$\beta_{D IM}$	2 nd strt pnt	$\ln(a)$	b	$\beta_{D IM}$	R^2
$\ln(\varphi_{slight})$	-0.369	0.590	0.175	0.297	0.625	1.409	0.382	0.90
$\ln(\varphi_{moderate})$	-1.552	0.596	0.178	0.332	-0.518	1.510	0.389	0.90
$\ln(\varphi_{extensive})$	-2.191	0.608	0.180	0.332	-1.167	1.529	0.368	0.91
$\ln(\varphi_{complete})$	-2.578	0.611	0.186	0.332	-1.563	1.522	0.367	0.91
$\ln(Deck_1)$	14.282	0.101	0.048	0.382	14.674	0.504	0.138	0.86
$\ln(Deck_2)$	14.456	0.102	0.037	0.354	14.730	0.357	0.097	0.89
$\ln(Deck_3)$	13.746	0.051	0.029	0.332	14.091	0.352	0.080	0.90
$\ln(Deck_4)$	14.468	0.106	0.042	0.382	14.749	0.398	0.101	0.87
$\ln(Deck_5)$	14.262	0.094	0.041	0.382	14.609	0.442	0.118	0.86

Table A-4 Correlation Matrix: 50 year old 52 m bridge 6.9 m column

	$\ln(ab_A)$	$\ln(ab_P)$	$\ln(ab_T)$	$\ln(\varphi_s)$	$\ln(\varphi_m)$	$\ln(\varphi_c)$	$\ln(D_1)$	$\ln(D_2)$	$\ln(D_3)$	$\ln(D_4)$	$\ln(D_5)$
$\ln(ab_A)$	1.000	0.699	0.901	0.945	0.947	0.952	0.939	0.957	0.946	0.937	0.863
$\ln(ab_P)$	0.699	1.000	0.670	0.610	0.612	0.610	0.525	0.574	0.566	0.520	0.475
$\ln(ab_T)$	0.901	0.670	1.000	0.951	0.950	0.951	0.866	0.874	0.875	0.865	0.882
$\ln(\varphi_s)$	0.945	0.610	0.951	1.000	0.996	0.993	0.936	0.938	0.935	0.942	0.925
$\ln(\varphi_m)$	0.947	0.612	0.950	0.996	1.000	0.995	0.941	0.941	0.939	0.946	0.930
$\ln(\varphi_c)$	0.952	0.610	0.951	0.993	0.995	1.000	0.943	0.944	0.942	0.945	0.924
$\ln(D_1)$	0.939	0.525	0.866	0.936	0.941	0.943	1.000	0.983	0.993	0.989	0.949
$\ln(D_2)$	0.957	0.574	0.874	0.938	0.941	0.944	0.983	1.000	0.989	0.988	0.939
$\ln(D_3)$	0.946	0.566	0.875	0.935	0.939	0.942	0.993	0.989	1.000	0.981	0.950
$\ln(D_4)$	0.937	0.520	0.865	0.942	0.946	0.945	0.989	0.988	0.981	1.000	0.945
$\ln(D_5)$	0.863	0.475	0.882	0.925	0.930	0.924	0.949	0.939	0.950	0.945	1.000

Table A-5 75 year old 52 m bridge 6.9 m column

$\ln(S_D) = \ln(a) + b \cdot \ln(PGA)$								
Linear Regression Lines								
Response	$\ln(a)$		b		$\beta_{D IM}$		R^2	
$\ln(ab_A)$	3.639		0.916		0.334		0.85	
$\ln(ab_P)$	3.898		1.609		0.655		0.77	
$\ln(ab_T)$	3.737		1.041		0.330		0.88	
Bilinear Regression Lines								
	1 st Regression Line				2 nd Regression Line			
	$\ln(a)$	b	$\beta_{D IM}$	2 nd strt pnt	$\ln(a)$	b	$\beta_{D IM}$	R^2
$\ln(\varphi_{slight})$	-0.366	0.545	0.181	0.332	0.858	1.648	0.443	0.89
$\ln(\varphi_{moderate})$	-1.585	0.579	0.187	0.354	-0.433	1.642	0.411	0.90
$\ln(\varphi_{extensive})$	-2.048	0.587	0.173	0.332	-1.004	1.475	0.394	0.90
$\ln(\varphi_{complete})$	-2.364	0.589	0.175	0.332	-1.323	1.459	0.400	0.90
$\ln(Deck_1)$	14.219	0.093	0.048	0.423	14.629	0.532	0.130	0.87
$\ln(Deck_2)$	14.439	0.105	0.038	0.382	14.691	0.361	0.085	0.90
$\ln(Deck_3)$	13.771	0.045	0.025	0.332	14.125	0.353	0.084	0.89
$\ln(Deck_4)$	14.442	0.106	0.042	0.423	14.731	0.426	0.093	0.89
$\ln(Deck_5)$	14.203	0.087	0.042	0.389	14.546	0.444	0.093	0.88

Table A-6 Correlation Matrix: 75 year old 52 m bridge 6.9 m column

	$\ln(ab_A)$	$\ln(ab_P)$	$\ln(ab_T)$	$\ln(\varphi_s)$	$\ln(\varphi_m)$	$\ln(\varphi_c)$	$\ln(D_1)$	$\ln(D_2)$	$\ln(D_3)$	$\ln(D_4)$	$\ln(D_5)$
$\ln(ab_A)$	1.000	0.704	0.898	0.943	0.945	0.947	0.935	0.956	0.948	0.933	0.863
$\ln(ab_P)$	0.704	1.000	0.691	0.603	0.631	0.625	0.520	0.585	0.571	0.519	0.483
$\ln(ab_T)$	0.898	0.691	1.000	0.939	0.938	0.944	0.859	0.874	0.872	0.863	0.882
$\ln(\varphi_s)$	0.943	0.603	0.939	1.000	0.991	0.992	0.940	0.933	0.936	0.949	0.927
$\ln(\varphi_m)$	0.945	0.631	0.938	0.991	1.000	0.993	0.933	0.931	0.934	0.941	0.922
$\ln(\varphi_c)$	0.947	0.625	0.944	0.992	0.993	1.000	0.940	0.939	0.941	0.949	0.927
$\ln(D_1)$	0.935	0.520	0.859	0.940	0.933	0.940	1.000	0.975	0.991	0.989	0.949
$\ln(D_2)$	0.956	0.585	0.874	0.933	0.931	0.939	0.975	1.000	0.987	0.981	0.935
$\ln(D_3)$	0.948	0.571	0.872	0.936	0.934	0.941	0.991	0.987	1.000	0.981	0.946
$\ln(D_4)$	0.933	0.519	0.863	0.949	0.941	0.949	0.989	0.981	0.981	1.000	0.949
$\ln(D_5)$	0.863	0.483	0.882	0.927	0.922	0.927	0.949	0.935	0.946	0.949	1.000

Table A-7 100 year old 52 m bridge 6.9 m column

$\ln(S_D) = \ln(a) + b \cdot \ln(PGA)$								
Linear Regression Lines								
Response	$\ln(a)$	b			$\beta_{D IM}$			R^2
$\ln(ab_A)$	3.617	0.898			0.332			0.84
$\ln(ab_P)$	3.940	1.722			0.733			0.76
$\ln(ab_T)$	3.723	1.036			0.328			0.88
Bilinear Regression Lines								
	1 st Regression Line				2 nd Regression Line			
	$\ln(a)$	b	$\beta_{D IM}$	2 nd strt pnt	$\ln(a)$	b	$\beta_{D IM}$	R^2
$\ln(\varphi_{slight})$	-0.260	0.548	0.174	0.332	0.907	1.582	0.381	0.90
$\ln(\varphi_{moderate})$	-1.663	0.553	0.174	0.354	-0.384	1.720	0.395	0.91
$\ln(\varphi_{extensive})$	-2.002	0.554	0.162	0.354	-0.729	1.721	0.402	0.91
$\ln(\varphi_{complete})$	-2.253	0.555	0.167	0.354	-0.984	1.721	0.409	0.91
$\ln(Deck_1)$	14.155	0.085	0.045	0.354	14.578	0.480	0.141	0.85
$\ln(Deck_2)$	14.439	0.118	0.043	0.556	14.662	0.455	0.073	0.91
$\ln(Deck_3)$	13.801	0.040	0.024	0.332	14.160	0.347	0.091	0.88
$\ln(Deck_4)$	14.417	0.107	0.042	0.423	14.691	0.416	0.082	0.90
$\ln(Deck_5)$	14.145	0.081	0.040	0.382	14.496	0.444	0.088	0.89

Table A-8 Correlation Matrix: 100 year old 52 m bridge 6.9 m column

	$\ln(ab_A)$	$\ln(ab_P)$	$\ln(ab_T)$	$\ln(\varphi_s)$	$\ln(\varphi_m)$	$\ln(\varphi_c)$	$\ln(D_1)$	$\ln(D_2)$	$\ln(D_3)$	$\ln(D_4)$	$\ln(D_5)$
$\ln(ab_A)$	1.000	0.714	0.895	0.944	0.942	0.943	0.934	0.961	0.950	0.931	0.864
$\ln(ab_P)$	0.714	1.000	0.709	0.628	0.623	0.625	0.523	0.608	0.588	0.524	0.489
$\ln(ab_T)$	0.895	0.709	1.000	0.943	0.936	0.943	0.857	0.877	0.873	0.865	0.878
$\ln(\varphi_s)$	0.944	0.628	0.943	1.000	0.993	0.993	0.942	0.937	0.942	0.951	0.935
$\ln(\varphi_m)$	0.942	0.623	0.936	0.993	1.000	0.994	0.941	0.934	0.940	0.949	0.938
$\ln(\varphi_c)$	0.943	0.625	0.943	0.993	0.994	1.000	0.942	0.937	0.942	0.949	0.936
$\ln(D_1)$	0.934	0.523	0.857	0.942	0.941	0.942	1.000	0.970	0.988	0.989	0.952
$\ln(D_2)$	0.961	0.608	0.877	0.937	0.934	0.937	0.970	1.000	0.986	0.976	0.926
$\ln(D_3)$	0.950	0.588	0.873	0.942	0.940	0.942	0.988	0.986	1.000	0.980	0.942
$\ln(D_4)$	0.931	0.524	0.865	0.951	0.949	0.949	0.989	0.976	0.980	1.000	0.953
$\ln(D_5)$	0.864	0.489	0.878	0.935	0.938	0.936	0.952	0.926	0.942	0.953	1.000

Table A-9 Correlation Matrix: 100 year old 52 m bridge 6.9 m column

$\ln(S_D) = \ln(a) + b \cdot \ln(PGA)$								
Linear Regression Lines								
Response	$\ln(a)$	b			$\beta_{D IM}$		R^2	
$\ln(ab_A)$	3.840	0.972			0.325		0.87	
$\ln(ab_P)$	4.209	1.550			0.602		0.81	
$\ln(ab_T)$	3.939	1.081			0.341		0.88	
Bilinear Regression Lines								
	1 st Regression Line				2 nd Regression Line			
	$\ln(a)$	b	$\beta_{D IM}$	2 nd strt pnt	$\ln(a)$	b	$\beta_{D IM}$	R^2
$\ln(\varphi_{slight})$	-1.043	0.533	0.161	0.258	-0.058	1.202	0.320	0.90
$\ln(\varphi_{moderate})$	-1.914	0.598	0.179	0.332	-1.025	1.343	0.348	0.91
$\ln(\varphi_{extensive})$	-2.900	0.597	0.182	0.332	-2.038	1.332	0.363	0.90
$\ln(\varphi_{complete})$	-3.387	0.596	0.188	0.332	-2.530	1.330	0.368	0.90
$\ln(Deck_1)$	14.299	0.077	0.035	0.389	14.615	0.406	0.099	0.88
$\ln(Deck_2)$	14.481	0.102	0.037	0.423	14.651	0.286	0.064	0.90
$\ln(Deck_3)$	13.706	0.052	0.022	0.229	13.985	0.235	0.079	0.86
$\ln(Deck_4)$	14.478	0.100	0.037	0.556	14.738	0.454	0.116	0.89
$\ln(Deck_5)$	14.294	0.075	0.034	0.389	14.568	0.360	0.087	0.88

Table A-10 Correlation Matrix: Pristine 52 m bridge 9.5 m column

	$\ln(ab_A)$	$\ln(ab_P)$	$\ln(ab_T)$	$\ln(\varphi_s)$	$\ln(\varphi_m)$	$\ln(\varphi_c)$	$\ln(D_1)$	$\ln(D_2)$	$\ln(D_3)$	$\ln(D_4)$	$\ln(D_5)$
$\ln(ab_A)$	1.000	0.694	0.909	0.948	0.954	0.950	0.909	0.937	0.926	0.906	0.854
$\ln(ab_P)$	0.694	1.000	0.643	0.603	0.596	0.593	0.500	0.573	0.552	0.501	0.505
$\ln(ab_T)$	0.909	0.643	1.000	0.955	0.955	0.948	0.864	0.884	0.873	0.865	0.881
$\ln(\varphi_s)$	0.948	0.603	0.955	1.000	0.995	0.990	0.930	0.926	0.926	0.932	0.893
$\ln(\varphi_m)$	0.954	0.596	0.955	0.995	1.000	0.993	0.931	0.928	0.927	0.930	0.895
$\ln(\varphi_c)$	0.950	0.593	0.948	0.990	0.993	1.000	0.928	0.926	0.926	0.929	0.885
$\ln(D_1)$	0.909	0.500	0.864	0.930	0.931	0.928	1.000	0.972	0.987	0.991	0.949
$\ln(D_2)$	0.937	0.573	0.884	0.926	0.928	0.926	0.972	1.000	0.986	0.980	0.951
$\ln(D_3)$	0.926	0.552	0.873	0.926	0.927	0.926	0.987	0.986	1.000	0.984	0.956
$\ln(D_4)$	0.906	0.501	0.865	0.932	0.930	0.929	0.991	0.980	0.984	1.000	0.948
$\ln(D_5)$	0.854	0.505	0.881	0.893	0.895	0.885	0.949	0.951	0.956	0.948	1.000

Table A-11 PSDM: 100 year old 52 m bridge 9.5 m column

$\ln(S_D) = \ln(a) + b \cdot \ln(PGA)$								
Linear Regression Lines								
Response	$\ln(a)$	b			$\beta_{D IM}$			R^2
$\ln(ab_A)$	3.734	0.902			0.342			0.84
$\ln(ab_P)$	4.205	1.697			0.681			0.78
$\ln(ab_T)$	3.911	1.076			0.342			0.88
Bilinear Regression Lines								
	1 st Regression Line				2 nd Regression Line			
	$\ln(a)$	b	$\beta_{D IM}$	2 nd strt pnt	$\ln(a)$	b	$\beta_{D IM}$	R^2
$\ln(\varphi_{slight})$	-0.749	0.450	0.175	0.332	0.381	1.453	0.392	0.89
$\ln(\varphi_{moderate})$	-2.163	0.493	0.162	0.332	-1.013	1.454	0.393	0.90
$\ln(\varphi_{extensive})$	-2.481	0.504	0.166	0.332	-1.367	1.441	0.385	0.90
$\ln(\varphi_{complete})$	-2.723	0.510	0.173	0.332	-1.627	1.434	0.383	0.90
$\ln(Deck_1)$	14.174	0.074	0.031	0.382	14.489	0.389	0.087	0.90
$\ln(Deck_2)$	14.433	0.119	0.042	0.556	14.641	0.455	0.066	0.92
$\ln(Deck_3)$	13.846	0.051	0.026	0.354	14.152	0.330	0.083	0.89
$\ln(Deck_4)$	14.426	0.115	0.042	0.556	14.647	0.440	0.094	0.89
$\ln(Deck_5)$	14.174	0.074	0.031	0.382	14.436	0.331	0.069	0.91

Table A-12 Correlation Matrix: 100 year old 52 m bridge 9.5 m column

	$\ln(ab_A)$	$\ln(ab_P)$	$\ln(ab_T)$	$\ln(\varphi_s)$	$\ln(\varphi_m)$	$\ln(\varphi_c)$	$\ln(D_1)$	$\ln(D_2)$	$\ln(D_3)$	$\ln(D_4)$	$\ln(D_5)$
$\ln(ab_A)$	1.000	0.705	0.905	0.948	0.945	0.949	0.911	0.951	0.939	0.904	0.862
$\ln(ab_P)$	0.705	1.000	0.685	0.606	0.597	0.604	0.517	0.601	0.591	0.524	0.490
$\ln(ab_T)$	0.905	0.685	1.000	0.940	0.942	0.945	0.874	0.885	0.879	0.879	0.883
$\ln(\varphi_s)$	0.948	0.606	0.940	1.000	0.994	0.993	0.931	0.921	0.917	0.931	0.907
$\ln(\varphi_m)$	0.945	0.597	0.942	0.994	1.000	0.997	0.932	0.918	0.915	0.931	0.909
$\ln(\varphi_c)$	0.949	0.604	0.945	0.993	0.997	1.000	0.933	0.921	0.919	0.930	0.906
$\ln(D_1)$	0.911	0.517	0.874	0.931	0.932	0.933	1.000	0.969	0.978	0.983	0.964
$\ln(D_2)$	0.951	0.601	0.885	0.921	0.918	0.921	0.969	1.000	0.988	0.971	0.943
$\ln(D_3)$	0.939	0.591	0.879	0.917	0.915	0.919	0.978	0.988	1.000	0.976	0.948
$\ln(D_4)$	0.904	0.524	0.879	0.931	0.931	0.930	0.983	0.971	0.976	1.000	0.960
$\ln(D_5)$	0.862	0.490	0.883	0.907	0.909	0.906	0.964	0.943	0.948	0.960	1.000

Table A-13 PSDM: Pristine 52 m bridge 12 m column

$\ln(S_D) = \ln(a) + b \cdot \ln(PGA)$								
Linear Regression Lines								
Response	$\ln(a)$	b			$\beta_{D IM}$	R^2		
$\ln(ab_A)$	3.945	0.999			0.315	0.88		
$\ln(ab_P)$	4.346	1.579			0.581	0.82		
$\ln(ab_T)$	4.056	1.121			0.354	0.88		
Bilinear Regression Lines								
	1 st Regression Line				2 nd Regression Line			
	$\ln(a)$	b	$\beta_{D IM}$	2 nd strt pnt	$\ln(a)$	b	$\beta_{D IM}$	R^2
$\ln(\varphi_{slight})$	-1.560	0.459	0.152	0.229	-0.319	1.261	0.311	0.92
$\ln(\varphi_{moderate})$	-2.770	0.390	0.158	0.176	-1.374	1.188	0.273	0.92
$\ln(\varphi_{extensive})$	-3.764	0.387	0.169	0.172	-2.404	1.152	0.276	0.92
$\ln(\varphi_{complete})$	-4.259	0.384	0.179	0.172	-2.901	1.141	0.280	0.91
$\ln(Deck_1)$	14.302	0.069	0.029	0.382	14.54	0.307	0.071	0.89
$\ln(Deck_2)$	14.465	0.100	0.037	0.556	14.612	0.343	0.056	0.89
$\ln(Deck_3)$	13.722	0.053	0.023	0.203	13.981	0.215	0.073	0.87
$\ln(Deck_4)$	14.459	0.097	0.038	0.556	14.657	0.384	0.094	0.87
$\ln(Deck_5)$	14.230	0.067	0.027	0.382	14.519	0.286	0.073	0.89

Table A-14 Correlation Matrix: Pristine 52 m bridge 12 m column

	$\ln(ab_A)$	$\ln(ab_P)$	$\ln(ab_T)$	$\ln(\varphi_s)$	$\ln(\varphi_m)$	$\ln(\varphi_c)$	$\ln(D_1)$	$\ln(D_2)$	$\ln(D_3)$	$\ln(D_4)$	$\ln(D_5)$
$\ln(ab_A)$	1.000	0.687	0.913	0.946	0.952	0.948	0.910	0.940	0.934	0.908	0.867
$\ln(ab_P)$	0.687	1.000	0.631	0.578	0.571	0.568	0.508	0.567	0.557	0.511	0.517
$\ln(ab_T)$	0.913	0.631	1.000	0.954	0.955	0.949	0.874	0.886	0.874	0.883	0.882
$\ln(\varphi_s)$	0.946	0.578	0.954	1.000	0.995	0.990	0.928	0.924	0.923	0.939	0.886
$\ln(\varphi_m)$	0.952	0.571	0.955	0.995	1.000	0.993	0.930	0.928	0.925	0.938	0.890
$\ln(\varphi_c)$	0.948	0.568	0.949	0.990	0.993	1.000	0.927	0.925	0.924	0.936	0.880
$\ln(D_1)$	0.910	0.508	0.874	0.928	0.930	0.927	1.000	0.983	0.985	0.993	0.953
$\ln(D_2)$	0.940	0.567	0.886	0.924	0.928	0.925	0.983	1.000	0.994	0.979	0.955
$\ln(D_3)$	0.934	0.557	0.874	0.923	0.925	0.924	0.985	0.994	1.000	0.985	0.949
$\ln(D_4)$	0.908	0.511	0.883	0.939	0.938	0.936	0.993	0.979	0.985	1.000	0.949
$\ln(D_5)$	0.867	0.517	0.882	0.886	0.890	0.880	0.953	0.955	0.949	0.949	1.000

Table A-15 PSDM: 100 year old 52 m bridge 12 m column

$\ln(S_D) = \ln(a) + b \cdot \ln(PGA)$								
Linear Regression Lines								
Response	$\ln(a)$	b			$\beta_{D IM}$	R^2		
$\ln(ab_A)$	3.797	0.910			0.346	0.84		
$\ln(ab_P)$	4.211	1.664			0.709	0.76		
$\ln(ab_T)$	4.016	1.112			0.356	0.88		
Bilinear Regression Lines								
	1 st Regression Line				2 nd Regression Line			
	$\ln(a)$	b	$\beta_{D IM}$	2 nd strt pnt	$\ln(a)$	b	$\beta_{D IM}$	R^2
$\ln(\varphi_{slight})$	-1.430	0.357	0.157	0.229	-0.048	1.260	0.297	0.91
$\ln(\varphi_{moderate})$	-2.857	0.347	0.147	0.229	-1.440	1.263	0.284	0.92
$\ln(\varphi_{extensive})$	-3.165	0.362	0.149	0.229	-1.804	1.242	0.276	0.92
$\ln(\varphi_{complete})$	-3.400	0.371	0.155	0.229	-2.069	1.232	0.282	0.92
$\ln(Deck_1)$	14.231	0.084	0.032	0.382	14.45	0.307	0.055	0.92
$\ln(Deck_2)$	14.414	0.115	0.040	0.556	14.606	0.343	0.067	0.91
$\ln(Deck_3)$	13.873	0.057	0.027	0.332	14.150	0.307	0.071	0.91
$\ln(Deck_4)$	14.415	0.115	0.042	0.556	14.573	0.382	0.070	0.88
$\ln(Deck_5)$	14.258	0.094	0.033	0.556	14.428	0.365	0.068	0.90

Table A-16 Correlation Matrix: 100 year old 52 m bridge 12 m column

	$\ln(ab_A)$	$\ln(ab_P)$	$\ln(ab_T)$	$\ln(\varphi_s)$	$\ln(\varphi_m)$	$\ln(\varphi_c)$	$\ln(D_1)$	$\ln(D_2)$	$\ln(D_3)$	$\ln(D_4)$	$\ln(D_5)$
$\ln(ab_A)$	1.000	0.702	0.909	0.947	0.943	0.947	0.922	0.949	0.946	0.918	0.874
$\ln(ab_P)$	0.702	1.000	0.676	0.586	0.575	0.582	0.558	0.603	0.600	0.565	0.511
$\ln(ab_T)$	0.909	0.676	1.000	0.941	0.945	0.946	0.897	0.890	0.884	0.900	0.895
$\ln(\varphi_s)$	0.947	0.586	0.941	1.000	0.994	0.993	0.929	0.915	0.919	0.926	0.903
$\ln(\varphi_m)$	0.943	0.575	0.945	0.994	1.000	0.997	0.928	0.912	0.914	0.925	0.904
$\ln(\varphi_c)$	0.947	0.582	0.946	0.993	0.997	1.000	0.927	0.914	0.918	0.924	0.901
$\ln(D_1)$	0.922	0.558	0.897	0.929	0.928	0.927	1.000	0.981	0.976	0.985	0.962
$\ln(D_2)$	0.949	0.603	0.890	0.915	0.912	0.914	0.981	1.000	0.987	0.978	0.945
$\ln(D_3)$	0.946	0.600	0.884	0.919	0.914	0.918	0.976	0.987	1.000	0.985	0.939
$\ln(D_4)$	0.918	0.565	0.900	0.926	0.925	0.924	0.985	0.978	0.985	1.000	0.958
$\ln(D_5)$	0.874	0.511	0.895	0.903	0.904	0.901	0.962	0.945	0.939	0.958	1.000

Table A-17 PSDM: Pristine 69 m bridge 6.9 m column

$\ln(S_D) = \ln(a) + b \cdot \ln(PGA)$								
Linear Regression Lines								
Response	$\ln(a)$		b		$\beta_{D IM}$		R^2	
$\ln(ab_A)$	3.389		0.865		0.294		0.87	
$\ln(ab_P)$	4.137		1.455		0.541		0.83	
$\ln(ab_T)$	4.020		1.098		0.378		0.87	
Bilinear Regression Lines								
	1 st Regression Line				2 nd Regression Line			
	$\ln(a)$	b	$\beta_{D IM}$	strt pnt	$\ln(a)$	b	$\beta_{D IM}$	R^2
$\ln(\varphi_{slight})$	-0.119	0.546	0.166	0.382	1.129	1.820	0.319	0.93
$\ln(\varphi_{moderate})$	-1.2326	0.518	0.157	0.382	0.059	1.772	0.309	0.93
$\ln(\varphi_{extensive})$	-2.162	0.511	0.161	0.382	-0.836	1.825	0.296	0.93
$\ln(\varphi_{complete})$	-2.631	0.510	0.168	0.382	-1.242	1.869	0.348	0.92
$\ln(Deck_1)$	15.006	0.078	0.036	0.382	15.281	0.348	0.010	0.85
$\ln(Deck_2)$	15.067	0.054	0.019	0.382	15.236	0.220	0.067	0.86
$\ln(Deck_3)$	14.140	0.050	0.031	0.353	14.303	0.199	0.059	0.82
$\ln(Deck_4)$	15.066	0.054	0.024	0.382	15.280	0.270	0.075	0.86
$\ln(Deck_5)$	15.004	0.077	0.032	0.354	15.240	0.302	0.079	0.87

Table A-18 Correlation Matrix: Pristine 69 m bridge 6.9 m column

	$\ln(ab_A)$	$\ln(ab_P)$	$\ln(ab_T)$	$\ln(\varphi_s)$	$\ln(\varphi_m)$	$\ln(\varphi_c)$	$\ln(D_1)$	$\ln(D_2)$	$\ln(D_3)$	$\ln(D_4)$	$\ln(D_5)$
$\ln(ab_A)$	1.000	0.653	0.904	0.940	0.940	0.939	0.948	0.910	0.934	0.935	0.875
$\ln(ab_P)$	0.653	1.000	0.593	0.552	0.549	0.543	0.503	0.487	0.494	0.500	0.492
$\ln(ab_T)$	0.904	0.593	1.000	0.947	0.952	0.950	0.858	0.825	0.847	0.847	0.856
$\ln(\varphi_s)$	0.940	0.552	0.947	1.000	0.995	0.993	0.942	0.905	0.933	0.925	0.907
$\ln(\varphi_m)$	0.940	0.549	0.952	0.995	1.000	0.993	0.942	0.908	0.933	0.927	0.912
$\ln(\varphi_c)$	0.939	0.543	0.950	0.993	0.993	1.000	0.941	0.907	0.934	0.926	0.910
$\ln(D_1)$	0.948	0.503	0.858	0.942	0.942	0.941	1.000	0.971	0.996	0.984	0.930
$\ln(D_2)$	0.910	0.487	0.825	0.905	0.908	0.907	0.971	1.000	0.977	0.994	0.945
$\ln(D_3)$	0.934	0.494	0.847	0.933	0.933	0.934	0.996	0.977	1.000	0.984	0.937
$\ln(D_4)$	0.935	0.500	0.847	0.925	0.927	0.926	0.984	0.994	0.984	1.000	0.939
$\ln(D_5)$	0.875	0.492	0.856	0.907	0.912	0.910	0.930	0.945	0.937	0.939	1.000

Table A-19 PSDM: 100 year old 69 m bridge 6.9 m column

$\ln(S_D) = \ln(a) + b \cdot \ln(PGA)$								
Linear Regression Lines								
Response	$\ln(a)$	b			$\beta_{D IM}$	R^2		
$\ln(ab_A)$	3.802	0.827			0.275	0.87		
$\ln(ab_P)$	4.081	1.553			0.564	0.81		
$\ln(ab_T)$	4.004	1.101			0.374	0.87		
Bilinear Regression Lines								
	1 st Regression Line				2 nd Regression Line			
	$\ln(a)$	b	$\beta_{D IM}$	2 nd strt pnt	$\ln(a)$	b	$\beta_{D IM}$	R^2
$\ln(\varphi_{slight})$	0.072	0.477	0.147	0.382	1.795	2.198	0.384	0.93
$\ln(\varphi_{moderate})$	-1.379	0.468	0.152	0.382	0.339	2.180	0.392	0.93
$\ln(\varphi_{extensive})$	-1.694	0.461	0.151	0.382	0.046	2.165	0.397	0.93
$\ln(\varphi_{complete})$	-1.934	0.457	0.153	0.382	-0.179	2.157	0.401	0.93
$\ln(Deck_1)$	14.904	0.071	0.032	0.382	15.236	0.396	0.092	0.89
$\ln(Deck_2)$	15.031	0.058	0.021	0.382	15.215	0.245	0.056	0.90
$\ln(Deck_3)$	14.217	0.041	0.025	0.354	14.441	0.245	0.065	0.86
$\ln(Deck_4)$	15.028	0.057	0.022	0.382	15.252	0.278	0.072	0.88
$\ln(Deck_5)$	14.908	0.072	0.031	0.382	15.170	0.331	0.067	0.90

Table A-20 Correlation Matrix: 100 year old 69 m bridge 6.9 m column

	$\ln(ab_A)$	$\ln(ab_P)$	$\ln(ab_T)$	$\ln(\varphi_s)$	$\ln(\varphi_m)$	$\ln(\varphi_c)$	$\ln(D_1)$	$\ln(D_2)$	$\ln(D_3)$	$\ln(D_4)$	$\ln(D_5)$
$\ln(ab_A)$	1.000	0.672	0.903	0.936	0.936	0.935	0.952	0.935	0.945	0.943	0.879
$\ln(ab_P)$	0.672	1.000	0.630	0.543	0.542	0.539	0.513	0.526	0.526	0.513	0.473
$\ln(ab_T)$	0.903	0.630	1.000	0.939	0.943	0.941	0.875	0.850	0.861	0.862	0.869
$\ln(\varphi_s)$	0.936	0.543	0.939	1.000	0.995	0.995	0.957	0.934	0.945	0.940	0.938
$\ln(\varphi_m)$	0.936	0.542	0.943	0.995	1.000	0.999	0.961	0.936	0.947	0.944	0.938
$\ln(\varphi_c)$	0.935	0.539	0.941	0.995	0.999	1.000	0.961	0.939	0.947	0.946	0.940
$\ln(D_1)$	0.952	0.513	0.875	0.957	0.961	0.961	1.000	0.978	0.994	0.987	0.942
$\ln(D_2)$	0.935	0.526	0.850	0.934	0.936	0.939	0.978	1.000	0.982	0.994	0.953
$\ln(D_3)$	0.945	0.526	0.861	0.945	0.947	0.947	0.994	0.982	1.000	0.985	0.942
$\ln(D_4)$	0.943	0.513	0.862	0.940	0.944	0.946	0.987	0.994	0.985	1.000	0.948
$\ln(D_5)$	0.879	0.473	0.869	0.938	0.938	0.940	0.942	0.953	0.942	0.948	1.000

Table A-21 PSDM: Pristine 69 m bridge 9.5 m column

$\ln(S_D) = \ln(a) + b \cdot \ln(PGA)$								
Linear Regression Lines								
Response	$\ln(a)$		b		$\beta_{D IM}$		R^2	
$\ln(ab_A)$	4.003		0.892		0.331		0.84	
$\ln(ab_P)$	4.437		1.539		0.591		0.83	
$\ln(ab_T)$	4.276		1.161		0.394		0.87	
Bilinear Regression Lines								
	1 st Regression Line				2 nd Regression Line			
	$\ln(a)$	b	$\beta_{D IM}$	2 nd strt pnt	$\ln(a)$	b	$\beta_{D IM}$	R^2
$\ln(\varphi_{slight})$	-0.541	0.516	0.175	0.382	0.773	1.861	0.357	0.91
$\ln(\varphi_{moderate})$	-1.658	0.488	0.177	0.382	-0.321	1.802	0.337	0.91
$\ln(\varphi_{extensive})$	-2.583	0.481	0.175	0.382	-1.225	1.827	0.331	0.91
$\ln(\varphi_{complete})$	-3.053	0.479	0.181	0.382	-1.692	1.832	0.331	0.91
$\ln(Deck_1)$	14.960	0.053	0.026	0.382	15.289	0.381	0.093	0.88
$\ln(Deck_2)$	15.041	0.047	0.018	0.382	15.236	0.240	0.064	0.88
$\ln(Deck_3)$	14.166	0.057	0.033	0.354	14.395	0.272	0.067	0.87
$\ln(Deck_4)$	15.045	0.049	0.020	0.382	15.296	0.304	0.078	0.88
$\ln(Deck_5)$	14.954	0.051	0.024	0.354	15.223	0.310	0.082	0.87

Table A-22 Correlation Matrix: Pristine 69 m bridge 9.5 m column

	$\ln(ab_A)$	$\ln(ab_P)$	$\ln(ab_T)$	$\ln(\varphi_s)$	$\ln(\varphi_m)$	$\ln(\varphi_c)$	$\ln(D_1)$	$\ln(D_2)$	$\ln(D_3)$	$\ln(D_4)$	$\ln(D_5)$
$\ln(ab_A)$	1.000	0.697	0.898	0.942	0.940	0.939	0.927	0.908	0.924	0.916	0.890
$\ln(ab_P)$	0.697	1.000	0.648	0.597	0.587	0.588	0.528	0.535	0.541	0.523	0.546
$\ln(ab_T)$	0.898	0.648	1.000	0.949	0.952	0.948	0.868	0.831	0.863	0.839	0.883
$\ln(\varphi_s)$	0.942	0.597	0.949	1.000	0.995	0.993	0.948	0.893	0.937	0.907	0.911
$\ln(\varphi_m)$	0.940	0.587	0.952	0.995	1.000	0.994	0.947	0.894	0.936	0.911	0.916
$\ln(\varphi_c)$	0.939	0.588	0.948	0.993	0.994	1.000	0.946	0.893	0.937	0.908	0.912
$\ln(D_1)$	0.927	0.528	0.868	0.948	0.947	0.946	1.000	0.971	0.995	0.977	0.940
$\ln(D_2)$	0.908	0.535	0.831	0.893	0.894	0.893	0.971	1.000	0.976	0.994	0.950
$\ln(D_3)$	0.924	0.541	0.863	0.937	0.936	0.937	0.995	0.976	1.000	0.976	0.946
$\ln(D_4)$	0.916	0.523	0.839	0.907	0.911	0.908	0.977	0.994	0.976	1.000	0.943
$\ln(D_5)$	0.890	0.546	0.883	0.911	0.916	0.912	0.940	0.950	0.946	0.943	1.000

Table A-23 PSDM: 100 year old 69 m bridge 9.5 m column

$\ln(S_D) = \ln(a) + b \cdot \ln(PGA)$								
Linear Regression Lines								
Response	$\ln(a)$	b			$\beta_{D IM}$			R^2
$\ln(ab_A)$	3.985	0.865			0.324			0.84
$\ln(ab_P)$	4.328	1.564			0.580			0.82
$\ln(ab_T)$	4.249	1.165			0.409			0.86
Bilinear Regression Lines								
	1 st Regression Line				2 nd Regression Line			
	$\ln(a)$	b	$\beta_{D IM}$	strt pnt	$\ln(a)$	b	$\beta_{D IM}$	R^2
$\ln(\varphi_{slight})$	-0.362	0.442	0.156	0.354	1.102	1.829	0.361	0.91
$\ln(\varphi_{moderate})$	-1.793	0.442	0.167	0.382	-0.328	1.870	0.391	0.90
$\ln(\varphi_{extensive})$	-2.108	0.435	0.165	0.354	-0.609	1.873	0.385	0.90
$\ln(\varphi_{complete})$	-2.351	0.429	0.167	0.354	-0.829	1.872	0.388	0.90
$\ln(Deck_1)$	14.878	0.050	0.023	0.354	15.225	0.384	0.091	0.89
$\ln(Deck_2)$	15.015	0.055	0.021	0.382	15.225	0.263	0.071	0.87
$\ln(Deck_3)$	14.245	0.048	0.029	0.354	14.514	0.292	0.077	0.87
$\ln(Deck_4)$	15.014	0.054	0.020	0.382	15.257	0.288	0.075	0.88
$\ln(Deck_5)$	14.875	0.049	0.022	0.354	15.173	0.323	0.095	0.85

Table A-24 Correlation Matrix: 100 year old 69 m bridge 9.5 m column

	$\ln(ab_A)$	$\ln(ab_P)$	$\ln(ab_T)$	$\ln(\varphi_s)$	$\ln(\varphi_m)$	$\ln(\varphi_c)$	$\ln(D_1)$	$\ln(D_2)$	$\ln(D_3)$	$\ln(D_4)$	$\ln(D_5)$
$\ln(ab_A)$	1.000	0.670	0.892	0.933	0.934	0.933	0.926	0.929	0.942	0.916	0.882
$\ln(ab_P)$	0.670	1.000	0.631	0.537	0.542	0.530	0.477	0.513	0.522	0.472	0.481
$\ln(ab_T)$	0.892	0.631	1.000	0.935	0.939	0.937	0.874	0.848	0.869	0.852	0.879
$\ln(\varphi_s)$	0.933	0.537	0.935	1.000	0.995	0.995	0.952	0.923	0.941	0.928	0.917
$\ln(\varphi_m)$	0.934	0.542	0.939	0.995	1.000	0.999	0.957	0.926	0.944	0.933	0.919
$\ln(\varphi_c)$	0.933	0.530	0.937	0.995	0.999	1.000	0.959	0.930	0.946	0.936	0.922
$\ln(D_1)$	0.926	0.477	0.874	0.952	0.957	0.959	1.000	0.981	0.991	0.991	0.956
$\ln(D_2)$	0.929	0.513	0.848	0.923	0.926	0.930	0.981	1.000	0.984	0.992	0.961
$\ln(D_3)$	0.942	0.522	0.869	0.941	0.944	0.946	0.991	0.984	1.000	0.985	0.951
$\ln(D_4)$	0.916	0.472	0.852	0.928	0.933	0.936	0.991	0.992	0.985	1.000	0.964
$\ln(D_5)$	0.882	0.481	0.879	0.917	0.919	0.922	0.956	0.961	0.951	0.964	1.000

Table A-25 PSDM: Pristine 69 m bridge 12 m column

$\ln(S_D) = \ln(a) + b \cdot \ln(PGA)$								
Linear Regression Lines								
Response	$\ln(a)$	b			$\beta_{D IM}$	R ²		
$\ln(ab_A)$	4.130	0.930			0.361	0.83		
$\ln(ab_P)$	4.533	1.550			0.558	0.84		
$\ln(ab_T)$	4.431	1.206			0.439	0.85		
Bilinear Regression Lines								
	1 st Regression Line				2 nd Regression Line			
	$\ln(a)$	b	$\beta_{D IM}$	strt pnt	$\ln(a)$	b	$\beta_{D IM}$	R ²
$\ln(\varphi_{slight})$	-0.940	0.482	0.181	0.354	1.641	0.325	0.379	0.89
$\ln(\varphi_{moderate})$	-2.041	0.461	0.184	0.332	-0.749	1.633	0.386	0.89
$\ln(\varphi_{extensive})$	-2.981	0.447	0.182	0.332	-1.673	1.619	0.354	0.89
$\ln(\varphi_{complete})$	-3.455	0.445	0.188	0.332	-2.145	1.615	0.349	0.89
$\ln(Deck_1)$	14.030	0.037	0.019	0.382	15.244	0.340	0.081	0.89
$\ln(Deck_2)$	15.021	0.042	0.017	0.354	15.210	0.217	0.064	0.86
$\ln(Deck_3)$	14.179	0.058	0.033	0.354	14.402	0.261	0.081	0.85
$\ln(Deck_4)$	15.024	0.043	0.017	0.382	15.261	0.276	0.067	0.90
$\ln(Deck_5)$	14.927	0.036	0.018	0.332	15.186	0.271	0.080	0.85

Table A-26 Correlation Matrix: Pristine 69 m bridge 12 m column

	$\ln(ab_A)$	$\ln(ab_P)$	$\ln(ab_T)$	$\ln(\varphi_s)$	$\ln(\varphi_m)$	$\ln(\varphi_c)$	$\ln(D_1)$	$\ln(D_2)$	$\ln(D_3)$	$\ln(D_4)$	$\ln(D_5)$
$\ln(ab_A)$	1.000	0.671	0.888	0.940	0.937	0.939	0.891	0.888	0.902	0.874	0.866
$\ln(ab_P)$	0.671	1.000	0.592	0.565	0.553	0.554	0.483	0.512	0.521	0.477	0.508
$\ln(ab_T)$	0.888	0.592	1.000	0.949	0.950	0.944	0.879	0.855	0.884	0.856	0.885
$\ln(\varphi_s)$	0.940	0.565	0.949	1.000	0.994	0.992	0.936	0.901	0.932	0.905	0.891
$\ln(\varphi_m)$	0.937	0.553	0.950	0.994	1.000	0.993	0.940	0.904	0.934	0.912	0.898
$\ln(\varphi_c)$	0.939	0.554	0.944	0.992	0.993	1.000	0.935	0.902	0.930	0.907	0.890
$\ln(D_1)$	0.891	0.483	0.879	0.936	0.940	0.935	1.000	0.978	0.993	0.983	0.941
$\ln(D_2)$	0.888	0.512	0.855	0.901	0.904	0.902	0.978	1.000	0.977	0.993	0.954
$\ln(D_3)$	0.902	0.521	0.884	0.932	0.934	0.930	0.993	0.977	1.000	0.971	0.948
$\ln(D_4)$	0.874	0.477	0.856	0.905	0.912	0.907	0.983	0.993	0.971	1.000	0.945
$\ln(D_5)$	0.866	0.508	0.885	0.891	0.898	0.890	0.941	0.954	0.948	0.945	1.000

Table A-27 PSDM: 100 year old 69 m bridge 12 m column

$\ln(S_D) = \ln(a) + b \cdot \ln(PGA)$								
Linear Regression Lines								
Response	$\ln(a)$		b		$\beta_{D IM}$		R^2	
$\ln(ab_A)$	4.107		0.903		0.359		0.82	
$\ln(ab_P)$	4.421		1.568		0.570		0.82	
$\ln(ab_T)$	4.397		1.207		0.454		0.85	
Bilinear Regression Lines								
	1 st Regression Line				2 nd Regression Line			
	$\ln(a)$	b	$\beta_{D IM}$	strt pnt	$\ln(a)$	b	$\beta_{D IM}$	R^2
$\ln(\varphi_{slight})$	-0.758	0.410	0.166	0.332	1.684	0.680	0.420	0.88
$\ln(\varphi_{moderate})$	-2.189	0.410	0.174	0.354	-0.756	1.711	0.444	0.87
$\ln(\varphi_{extensive})$	-2.495	0.407	0.172	0.332	-1.030	1.738	0.441	0.88
$\ln(\varphi_{complete})$	-2.736	0.403	0.172	0.332	-1.248	1.743	0.437	0.88
$\ln(Deck_1)$	14.866	0.036	0.017	0.354	15.139	0.298	0.080	0.87
$\ln(Deck_2)$	14.999	0.051	0.021	0.382	15.222	0.268	0.081	0.86
$\ln(Deck_3)$	14.222	0.036	0.021	0.257	14.464	0.206	0.069	0.84
$\ln(Deck_4)$	15.000	0.051	0.019	0.382	15.192	0.238	0.067	0.87
$\ln(Deck_5)$	14.866	0.036	0.016	0.354	15.111	0.271	0.074	0.87

Table A-28 Correlation Matrix 100 year old 69 m bridge 12 m column

	$\ln(ab_A)$	$\ln(ab_P)$	$\ln(ab_T)$	$\ln(\varphi_s)$	$\ln(\varphi_m)$	$\ln(\varphi_c)$	$\ln(D_1)$	$\ln(D_2)$	$\ln(D_3)$	$\ln(D_4)$	$\ln(D_5)$
$\ln(ab_A)$	1.000	0.660	0.886	0.929	0.930	0.930	0.860	0.908	0.898	0.863	0.844
$\ln(ab_P)$	0.660	1.000	0.611	0.521	0.526	0.514	0.444	0.506	0.509	0.446	0.468
$\ln(ab_T)$	0.886	0.611	1.000	0.936	0.939	0.937	0.864	0.865	0.869	0.855	0.880
$\ln(\varphi_s)$	0.929	0.521	0.936	1.000	0.995	0.995	0.913	0.917	0.909	0.908	0.895
$\ln(\varphi_m)$	0.930	0.526	0.939	0.995	1.000	0.998	0.919	0.921	0.913	0.914	0.899
$\ln(\varphi_c)$	0.930	0.514	0.937	0.995	0.998	1.000	0.923	0.925	0.917	0.917	0.902
$\ln(D_1)$	0.860	0.444	0.864	0.913	0.919	0.923	1.000	0.976	0.985	0.992	0.971
$\ln(D_2)$	0.908	0.506	0.865	0.917	0.921	0.925	0.976	1.000	0.974	0.983	0.959
$\ln(D_3)$	0.898	0.509	0.869	0.909	0.913	0.917	0.985	0.974	1.000	0.976	0.959
$\ln(D_4)$	0.863	0.446	0.855	0.908	0.914	0.917	0.992	0.983	0.976	1.000	0.969
$\ln(D_5)$	0.844	0.468	0.880	0.895	0.899	0.902	0.971	0.959	0.959	0.969	1.000

Table A-29 PSDM: Pristine 52 m bridge 6.9 m column & 2.5 cm settlement

$\ln(S_D) = \ln(a) + b \cdot \ln(PGA)$								
Linear Regression Lines								
Response	$\ln(a)$	b		$\beta_{D IM}$		R ²		
$\ln(ab_A)$	3.586	0.832		0.348		0.81		
$\ln(ab_P)$	4.000	1.679		0.745		0.74		
$\ln(ab_T)$	3.819	1.043		0.334		0.88		
Bilinear Regression Lines								
	1 st Regression Line				2 nd Regression Line			
	$\ln(a)$	b	$\beta_{D IM}$	strt pnt	$\ln(a)$	b	$\beta_{D IM}$	R ²
$\ln(\varphi_{slight})$	-0.789	0.159	0.119	0.332	0.519	1.240	0.357	0.84
$\ln(\varphi_{moderate})$	-1.796	0.169	0.103	0.332	-0.492	1.278	0.345	0.86
$\ln(\varphi_{extensive})$	-2.783	0.167	0.107	0.332	-1.518	1.249	0.347	0.85
$\ln(\varphi_{complete})$	-3.269	0.167	0.117	0.332	-2.013	1.242	0.349	0.84
$\ln(Deck_1)$	13.473	0.181	0.097	0.382	14.258	0.936	0.263	0.86
$\ln(Deck_2)$	13.595	0.231	0.136	0.382	14.233	0.136	0.219	0.85
$\ln(Deck_3)$	13.954	0.037	0.024	0.229	14.197	0.197	0.065	0.85
$\ln(Deck_4)$	14.782	0.068	0.032	0.354	14.983	0.258	0.076	0.85
$\ln(Deck_5)$	14.647	0.079	0.046	0.354	14.936	0.341	0.106	0.83

Table A-30 Correlation Matrix: Pristine 52 m bridge 6.9 m column & 2.5 cm settlement

	$\ln(ab_A)$	$\ln(ab_P)$	$\ln(ab_T)$	$\ln(\varphi_s)$	$\ln(\varphi_m)$	$\ln(\varphi_c)$	$\ln(D_1)$	$\ln(D_2)$	$\ln(D_3)$	$\ln(D_4)$	$\ln(D_5)$
$\ln(ab_A)$	1.000	0.683	0.890	0.884	0.894	0.887	0.940	0.952	0.934	0.949	0.880
$\ln(ab_P)$	0.683	1.000	0.716	0.461	0.463	0.449	0.542	0.572	0.564	0.533	0.536
$\ln(ab_T)$	0.890	0.716	1.000	0.851	0.857	0.847	0.857	0.855	0.849	0.836	0.857
$\ln(\varphi_s)$	0.884	0.461	0.851	1.000	0.991	0.981	0.937	0.900	0.899	0.903	0.872
$\ln(\varphi_m)$	0.894	0.463	0.857	0.991	1.000	0.988	0.942	0.908	0.901	0.910	0.878
$\ln(\varphi_c)$	0.887	0.449	0.847	0.981	0.988	1.000	0.933	0.900	0.899	0.909	0.868
$\ln(D_1)$	0.940	0.542	0.857	0.937	0.942	0.933	1.000	0.975	0.974	0.965	0.934
$\ln(D_2)$	0.952	0.572	0.855	0.900	0.908	0.900	0.975	1.000	0.951	0.974	0.929
$\ln(D_3)$	0.934	0.564	0.849	0.899	0.901	0.899	0.974	0.951	1.000	0.964	0.950
$\ln(D_4)$	0.949	0.533	0.836	0.903	0.910	0.909	0.965	0.974	0.964	1.000	0.937
$\ln(D_5)$	0.880	0.536	0.857	0.872	0.878	0.868	0.934	0.929	0.950	0.937	1.000

Table A-31 PSDM: Pristine 52 m bridge 6.9 m column & 5 cm settlement

$\ln(S_D) = \ln(a) + b \cdot \ln(PGA)$								
Linear Regression Lines								
Response	$\ln(a)$	b			$\beta_{D IM}$	R^2		
$\ln(ab_A)$	3.491	0.679			0.335	0.75		
$\ln(ab_P)$	3.824	1.812			0.742	0.75		
$\ln(ab_T)$	3.855	1.052			0.337	0.88		
Bilinear Regression Lines								
	1 st Regression Line				2 nd Regression Line			
	$\ln(a)$	b	$\beta_{D IM}$	strt pnt	$\ln(a)$	b	$\beta_{D IM}$	R^2
$\ln(\varphi_{slight})$	-0.544	0.086	0.118	0.332	0.555	1.046	0.316	0.78
$\ln(\varphi_{moderate})$	-1.552	0.096	0.093	0.332	-0.456	1.083	0.292	0.82
$\ln(\varphi_{extensive})$	-2.539	0.094	0.010	0.332	-1.482	1.054	0.290	0.81
$\ln(\varphi_{complete})$	-3.026	0.093	0.111	0.332	-1.978	1.045	0.292	0.79
$\ln(Deck_1)$	13.438	0.202	0.127	0.229	13.798	0.434	0.212	0.76
$\ln(Deck_2)$	13.710	0.066	0.095	0.229	14.127	0.336	0.170	0.68
$\ln(Deck_3)$	14.259	0.017	0.028	0.229	14.403	0.115	0.229	0.71
$\ln(Deck_4)$	15.014	0.039	0.025	0.229	15.162	0.139	0.055	0.79
$\ln(Deck_5)$	14.884	0.055	0.039	0.382	15.097	0.264	0.086	0.79

Table A-32 Correlation Matrix: Pristine 52 m bridge 6.9 m column & 5 cm settlement

	$\ln(ab_A)$	$\ln(ab_P)$	$\ln(ab_T)$	$\ln(\varphi_s)$	$\ln(\varphi_m)$	$\ln(\varphi_c)$	$\ln(D_1)$	$\ln(D_2)$	$\ln(D_3)$	$\ln(D_4)$	$\ln(D_5)$
$\ln(ab_A)$	1.000	0.706	0.858	0.876	0.890	0.881	0.853	0.823	0.896	0.954	0.770
$\ln(ab_P)$	0.706	1.000	0.783	0.464	0.483	0.485	0.672	0.519	0.631	0.584	0.512
$\ln(ab_T)$	0.858	0.783	1.000	0.804	0.817	0.803	0.806	0.761	0.816	0.808	0.751
$\ln(\varphi_s)$	0.876	0.464	0.804	1.000	0.986	0.971	0.761	0.821	0.843	0.889	0.762
$\ln(\varphi_m)$	0.890	0.483	0.817	0.986	1.000	0.981	0.746	0.803	0.840	0.894	0.755
$\ln(\varphi_c)$	0.881	0.485	0.803	0.971	0.981	1.000	0.732	0.786	0.832	0.887	0.733
$\ln(D_1)$	0.853	0.672	0.806	0.761	0.746	0.732	1.000	0.934	0.940	0.895	0.907
$\ln(D_2)$	0.823	0.519	0.761	0.821	0.803	0.786	0.934	1.000	0.941	0.913	0.962
$\ln(D_3)$	0.896	0.631	0.816	0.843	0.840	0.832	0.940	0.941	1.000	0.950	0.906
$\ln(D_4)$	0.954	0.584	0.808	0.889	0.894	0.887	0.895	0.913	0.950	1.000	0.858
$\ln(D_5)$	0.770	0.512	0.751	0.762	0.755	0.733	0.907	0.962	0.906	0.858	1.000

Table A-33 100 year old 52 m bridge 6.9 m column & 2.5 cm settlement

$\ln(S_D) = \ln(a) + b \cdot \ln(PGA)$								
Linear Regression Lines								
Response	$\ln(a)$	b			$\beta_{D IM}$			R^2
$\ln(ab_A)$	3.544	0.789			0.347			0.79
$\ln(ab_P)$	3.883	1.670			0.630			0.78
$\ln(ab_T)$	3.768	1.038			0.327			0.88
Bilinear Regression Lines								
	1 st Regression Line				2 nd Regression Line			
	$\ln(a)$	b	$\beta_{D IM}$	strt pnt	$\ln(a)$	b	$\beta_{D IM}$	R^2
$\ln(\varphi_{slight})$	-0.479	0.163	0.124	0.332	0.943	1.411	0.364	0.85
$\ln(\varphi_{moderate})$	-1.894	0.157	0.113	0.332	-0.436	1.434	0.379	0.85
$\ln(\varphi_{extensioe})$	-2.212	0.168	0.117	0.332	-0.784	1.430	0.371	0.85
$\ln(\varphi_{complete})$	-2.454	0.174	0.125	0.332	-1.040	1.428	0.368	0.85
$\ln(Deck_1)$	13.469	0.145	0.089	0.354	14.192	0.823	0.229	0.86
$\ln(Deck_2)$	13.618	0.159	0.096	0.423	14.182	0.756	0.143	0.88
$\ln(Deck_3)$	14.036	0.041	0.026	0.332	14.304	0.280	0.062	0.90
$\ln(Deck_4)$	14.666	0.041	0.025	0.423	14.883	0.343	0.081	0.87
$\ln(Deck_5)$	14.392	0.064	0.040	0.382	14.707	0.378	0.084	0.88

Table A-34 Correlation Matrix: 100 year old 52 m bridge 6.9 m column & 2.5 cm settlement

	$\ln(ab_A)$	$\ln(ab_P)$	$\ln(ab_T)$	$\ln(\varphi_s)$	$\ln(\varphi_m)$	$\ln(\varphi_c)$	$\ln(D_1)$	$\ln(D_2)$	$\ln(D_3)$	$\ln(D_4)$	$\ln(D_5)$
$\ln(ab_A)$	1.000	0.711	0.880	0.899	0.898	0.905	0.941	0.952	0.942	0.936	0.877
$\ln(ab_P)$	0.711	1.000	0.760	0.489	0.481	0.497	0.562	0.598	0.632	0.556	0.550
$\ln(ab_T)$	0.880	0.760	1.000	0.849	0.847	0.855	0.850	0.845	0.856	0.839	0.864
$\ln(\varphi_s)$	0.899	0.489	0.849	1.000	0.990	0.988	0.940	0.911	0.894	0.923	0.898
$\ln(\varphi_m)$	0.898	0.481	0.847	0.990	1.000	0.995	0.938	0.909	0.888	0.920	0.895
$\ln(\varphi_c)$	0.905	0.497	0.855	0.988	0.995	1.000	0.943	0.915	0.896	0.926	0.899
$\ln(D_1)$	0.941	0.562	0.850	0.940	0.938	0.943	1.000	0.974	0.969	0.978	0.952
$\ln(D_2)$	0.952	0.598	0.845	0.911	0.909	0.915	0.974	1.000	0.948	0.963	0.923
$\ln(D_3)$	0.942	0.632	0.856	0.894	0.888	0.896	0.969	0.948	1.000	0.976	0.956
$\ln(D_4)$	0.936	0.556	0.839	0.923	0.920	0.926	0.978	0.963	0.976	1.000	0.950
$\ln(D_5)$	0.877	0.550	0.864	0.898	0.895	0.899	0.952	0.923	0.956	0.950	1.000

Table A-35 PSDM: 100 year old 52 m bridge 6.9 m column & 5 cm settlement

$\ln(S_D) = \ln(a) + b \cdot \ln(PGA)$								
Linear Regression Lines								
Response	$\ln(a)$		b	$\beta_{D IM}$	R^2			
$\ln(ab_A)$	3.471		0.641		0.329		0.74	
$\ln(ab_P)$	3.863		2.149		1.007		0.69	
$\ln(ab_T)$	3.792		1.042		0.332		0.88	
Bilinear Regression Lines								
	1 st Regression Line				2 nd Regression Line			
	$\ln(a)$	b	$\beta_{D IM}$	strt pnt	$\ln(a)$	b	$\beta_{D IM}$	R^2
$\ln(\varphi_{slight})$	-0.184	0.140	0.127	0.354	0.943	1.411	0.317	0.82
$\ln(\varphi_{moderate})$	-1.591	0.137	0.101	0.354	-0.402	1.249	0.324	0.84
$\ln(\varphi_{extensive})$	-1.911	0.148	0.102	0.354	-0.749	1.245	0.317	0.84
$\ln(\varphi_{complete})$	-2.153	0.154	0.111	0.354	-1.006	1.243	0.316	0.83
$\ln(Deck_1)$	13.321	0.236	0.161	0.138	13.668	0.411	0.194	0.77
$\ln(Deck_2)$	13.395	0.120	0.163	0.229	13.834	0.410	0.217	0.63
$\ln(Deck_3)$	14.316	0.039	0.033	0.332	14.437	0.149	0.055	0.75
$\ln(Deck_4)$	14.877	0.069	0.037	0.556	15.023	0.281	0.062	0.84
$\ln(Deck_5)$	14.660	0.065	0.047	0.556	14.878	0.411	0.080	0.82

Table A-36 Correlation Matrix: 100 year old 52 m bridge 6.9 m column & 5 cm settlement

	$\ln(ab_A)$	$\ln(ab_P)$	$\ln(ab_T)$	$\ln(\varphi_s)$	$\ln(\varphi_m)$	$\ln(\varphi_c)$	$\ln(D_1)$	$\ln(D_2)$	$\ln(D_3)$	$\ln(D_4)$	$\ln(D_5)$
$\ln(ab_A)$	1.000	0.742	0.846	0.899	0.903	0.913	0.800	0.751	0.878	0.925	0.810
$\ln(ab_P)$	0.742	1.000	0.785	0.557	0.560	0.581	0.680	0.528	0.673	0.620	0.602
$\ln(ab_T)$	0.846	0.785	1.000	0.821	0.824	0.835	0.798	0.714	0.809	0.795	0.782
$\ln(\varphi_s)$	0.899	0.557	0.821	1.000	0.987	0.984	0.713	0.715	0.828	0.866	0.755
$\ln(\varphi_m)$	0.903	0.560	0.824	0.987	1.000	0.993	0.702	0.708	0.823	0.867	0.757
$\ln(\varphi_c)$	0.913	0.581	0.835	0.984	0.993	1.000	0.717	0.718	0.834	0.876	0.765
$\ln(D_1)$	0.800	0.680	0.798	0.713	0.702	0.717	1.000	0.934	0.936	0.875	0.914
$\ln(D_2)$	0.751	0.528	0.714	0.715	0.708	0.718	0.934	1.000	0.917	0.898	0.935
$\ln(D_3)$	0.878	0.673	0.809	0.828	0.823	0.834	0.936	0.917	1.000	0.953	0.919
$\ln(D_4)$	0.925	0.620	0.795	0.866	0.867	0.876	0.875	0.898	0.953	1.000	0.905
$\ln(D_5)$	0.810	0.602	0.782	0.755	0.757	0.765	0.914	0.935	0.919	0.905	1.000

Table 1. Comparison of the results of the two methods for the determination of the concentration of the components in the mixture.

Component	Concentration (%)	Method 1 (%)	Method 2 (%)	Method 3 (%)	Method 4 (%)	Method 5 (%)	Method 6 (%)	Method 7 (%)	Method 8 (%)	Method 9 (%)	Method 10 (%)
Component A	10.0	10.0	10.0	10.0	10.0	10.0	10.0	10.0	10.0	10.0	10.0
Component B	20.0	20.0	20.0	20.0	20.0	20.0	20.0	20.0	20.0	20.0	20.0
Component C	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0
Component D	40.0	40.0	40.0	40.0	40.0	40.0	40.0	40.0	40.0	40.0	40.0
Component E	50.0	50.0	50.0	50.0	50.0	50.0	50.0	50.0	50.0	50.0	50.0
Component F	60.0	60.0	60.0	60.0	60.0	60.0	60.0	60.0	60.0	60.0	60.0
Component G	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0
Component H	80.0	80.0	80.0	80.0	80.0	80.0	80.0	80.0	80.0	80.0	80.0
Component I	90.0	90.0	90.0	90.0	90.0	90.0	90.0	90.0	90.0	90.0	90.0
Component J	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0

Table 2. Comparison of the results of the two methods for the determination of the concentration of the components in the mixture.

Component	Concentration (%)	Method 1 (%)	Method 2 (%)	Method 3 (%)	Method 4 (%)	Method 5 (%)	Method 6 (%)	Method 7 (%)	Method 8 (%)	Method 9 (%)	Method 10 (%)
Component A	10.0	10.0	10.0	10.0	10.0	10.0	10.0	10.0	10.0	10.0	10.0
Component B	20.0	20.0	20.0	20.0	20.0	20.0	20.0	20.0	20.0	20.0	20.0
Component C	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0
Component D	40.0	40.0	40.0	40.0	40.0	40.0	40.0	40.0	40.0	40.0	40.0
Component E	50.0	50.0	50.0	50.0	50.0	50.0	50.0	50.0	50.0	50.0	50.0
Component F	60.0	60.0	60.0	60.0	60.0	60.0	60.0	60.0	60.0	60.0	60.0
Component G	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0
Component H	80.0	80.0	80.0	80.0	80.0	80.0	80.0	80.0	80.0	80.0	80.0
Component I	90.0	90.0	90.0	90.0	90.0	90.0	90.0	90.0	90.0	90.0	90.0
Component J	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0

APPENDIX - B AGEING FRAGILITY ESTIMATES FOR INTEGRAL BRIDGES

Table B-1 52 m bridge 6.9 m column

	Slight		Moderate		Extensive		Complete	
	0yrs	100yrs	0yrs	100yrs	0yrs	100yrs	0yrs	100yrs
10% in 50 yr Earthquake								
Columns	0.15	0.36	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01
Abutment Active	0.34	0.31	<0.01	<0.01	N/A	N/A	N/A	N/A
Abutment Passive	0.03	0.03	<0.01	<0.01	N/A	N/A	N/A	N/A
Abutment Transverse	<0.01	<0.01	<0.01	<0.01	N/A	N/A	N/A	N/A
Deck	<0.01	0.29	<0.01	0.03	<0.01	<0.01	<0.01	<0.01
System	0.40	0.63	<0.01	0.03	<0.01	<0.01	<0.01	<0.01
2% in 50 yr Earthquake								
Columns	0.81	0.95	0.17	0.21	<0.01	0.07	<0.01	0.02
Abutment Active	0.81	0.77	0.03	0.02	N/A	N/A	N/A	N/A
Abutment Passive	0.33	0.29	0.03	0.03	N/A	N/A	N/A	N/A
Abutment Transverse	0.11	0.09	<0.01	<0.01	N/A	N/A	N/A	N/A
Deck	0.03	0.64	<0.01	0.17	<0.01	0.03	<0.01	<0.01
System	0.93	0.99	0.19	0.34	<0.01	0.09	<0.01	0.02

Table B-2 52 m bridge 9.5 m column

	Slight		Moderate		Extensive		Complete	
	0yrs	100yrs	0yrs	100yrs	0yrs	100yrs	0yrs	100yrs
10% in 50 yr Earthquake								
Columns	0.02	0.08	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01
Abutment Active	0.38	0.34	<0.01	<0.01	N/A	N/A	N/A	N/A
Abutment Passive	0.03	0.03	<0.01	<0.01	N/A	N/A	N/A	N/A
Abutment Transverse	<0.01	<0.01	<0.01	<0.01	N/A	N/A	N/A	N/A
Deck	<0.01	0.23	<0.01	0.02	<0.01	<0.01	<0.01	<0.01
System	0.39	0.50	<0.01	0.02	<0.01	<0.01	<0.01	<0.01
2% in 50 yr Earthquake								
Columns	0.37	0.69	0.01	0.02	<0.01	<0.01	<0.01	<0.01
Abutment Active	0.86	0.81	0.04	0.03	N/A	N/A	N/A	N/A
Abutment Passive	0.36	0.36	0.03	0.04	N/A	N/A	N/A	N/A
Abutment Transverse	0.16	0.15	<0.01	<0.01	N/A	N/A	N/A	N/A
Deck	0.01	0.56	<0.01	0.11	<0.01	0.02	<0.01	<0.01
System	0.89	0.95	0.07	0.18	<0.01	0.02	<0.01	<0.01

Table B-3 52 m bridge 12 m column

	Slight		Moderate		Extensive		Complete	
	0yrs	100yrs	0yrs	100yrs	0yrs	100yrs	0yrs	100yrs
10% in 50 yr Earthquake								
Columns	<0.01	0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01
Abutment Active	0.44	0.39	<0.01	<0.01	N/A	N/A	N/A	N/A
Abutment Passive	0.04	0.04	<0.01	<0.01	N/A	N/A	N/A	N/A
Abutment Transverse	0.01	0.01	<0.01	<0.01	N/A	N/A	N/A	N/A
Deck	<0.01	0.22	<0.01	0.02	<0.01	<0.01	<0.01	<0.01
System	0.46	0.52	<0.01	0.02	<0.01	<0.01	<0.01	<0.01
2% in 50 yr Earthquake								
Columns	0.18	0.37	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01
Abutment Active	0.91	0.83	0.06	0.05	N/A	N/A	N/A	N/A
Abutment Passive	0.43	0.37	0.04	<0.01	N/A	N/A	N/A	N/A
Abutment Transverse	0.23	0.21	<0.01	<0.01	N/A	N/A	N/A	N/A
Deck	<0.01	0.51	<0.01	0.10	<0.01	0.01	<0.01	<0.01
System	0.93	0.93	0.09	0.16	<0.01	0.01	<0.01	<0.01

Table B-4 69 m bridge 6.9 m column

	Slight		Moderate		Extensive		Complete	
	0yrs	100yrs	0yrs	100yrs	0yrs	100yrs	0yrs	100yrs
10% in 50 yr Earthquake								
Columns	0.27	0.57	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01
Abutment Active	0.44	0.43	<0.01	<0.01	N/A	N/A	N/A	N/A
Abutment Passive	0.02	0.01	<0.01	<0.01	N/A	N/A	N/A	N/A
Abutment Transverse	0.01	0.01	<0.01	<0.01	N/A	N/A	N/A	N/A
Deck	<0.01	0.04	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01
System	0.55	0.70	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01
2% in 50 yr Earthquake								
Columns	0.98	1.00	0.42	0.62	0.02	0.40	<0.01	0.25
Abutment Active	0.88	0.87	0.03	0.03	N/A	N/A	N/A	N/A
Abutment Passive	0.31	0.28	0.02	0.02	N/A	N/A	N/A	N/A
Abutment Transverse	0.22	0.21	<0.01	<0.01	N/A	N/A	N/A	N/A
Deck	<0.01	0.13	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01
System	0.99	1.0	0.44	0.62	0.03	0.40	<0.01	0.25

Table B-5 69 m bridge 9.5 m column

	Slight		Moderate		Extensive		Complete	
	0yrs	100yrs	0yrs	100yrs	0yrs	100yrs	0yrs	100yrs
10% in 50 yr Earthquake								
Columns	0.09	0.26	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01
Abutment Active	0.56	0.56	<0.01	<0.01	N/A	N/A	N/A	N/A
Abutment Passive	0.06	0.04	<0.01	<0.01	N/A	N/A	N/A	N/A
Abutment Transverse	0.03	0.03	<0.01	<0.01	N/A	N/A	N/A	N/A
Deck	<0.01	0.04	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01
System	0.59	0.64	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01
2% in 50 yr Earthquake								
Columns	0.89	0.97	0.17	0.18	<0.01	0.07	<0.01	0.03
Abutment Active	0.93	0.92	0.08	0.07	N/A	N/A	N/A	N/A
Abutment Passive	0.49	0.42	0.06	0.04	N/A	N/A	N/A	N/A
Abutment Transverse	0.39	0.37	0.01	0.01	N/A	N/A	N/A	N/A
Deck	<0.01	0.14	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01
System	0.98	0.99	0.24	0.24	<0.01	0.07	<0.01	0.03

Table B-6 69 m bridge 12 m column

	Slight		Moderate		Extensive		Complete	
	0yrs	100yrs	0yrs	100yrs	0yrs	100yrs	0yrs	100yrs
10% in 50 yr Earthquake								
Columns	0.03	0.13	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01
Abutment Active	0.63	0.63	<0.01	<0.01	N/A	N/A	N/A	N/A
Abutment Passive	0.06	0.05	<0.01	<0.01	N/A	N/A	N/A	N/A
Abutment Transverse	0.07	0.06	<0.01	<0.01	N/A	N/A	N/A	N/A
Deck	<0.01	0.03	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01
System	0.65	0.67	<0.01	0.01	<0.01	<0.01	<0.01	<0.01
2% in 50 yr Earthquake								
Columns	0.64	0.84	0.04	0.06	<0.01	0.02	<0.01	<0.01
Abutment Active	0.95	0.94	0.14	0.13	N/A	N/A	N/A	N/A
Abutment Passive	0.54	0.47	0.07	0.05	N/A	N/A	N/A	N/A
Abutment Transverse	0.50	0.47	0.03	0.03	N/A	N/A	N/A	N/A
Deck	<0.01	0.11	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01
System	0.97	0.98	0.21	0.20	<0.01	0.02	<0.01	<0.01

Table B-7 52 m bridge 6.9 m column & settlement

	Slight			Moderate			Extensive			Complete		
	100yrs	100	100	100yrs	100	100	100yrs	100	100	100yrs	100	100
		yrs 2.5	yrs		yrs 2.5	yrs		yrs 2.5	yrs		yrs 2.5	yrs
		cm	5 cm		cm	5 cm		cm	5 cm		cm	5 cm
10% in 50 yr Earthquake												
Columns	0.36	0.47	0.61	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01
Abutment Active	0.31	0.32	0.33	<0.01	<0.01	<0.01	N/A	N/A	N/A	N/A	N/A	N/A
Abutment Passive	0.03	0.01	0.03	<0.01	<0.01	<0.01	N/A	N/A	N/A	N/A	N/A	N/A
Abutment Transverse	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	N/A	N/A	N/A	N/A	N/A	N/A
Deck	0.29	0.60	0.81	0.03	0.14	0.32	<0.01	0.02	0.08	<0.01	<0.01	<0.01
System	0.63	0.81	0.93	0.03	0.15	0.32	<0.01	0.02	0.8	<0.01	<0.01	<0.01
2% in 50 yr Earthquake												
Columns	0.95	0.96	0.98	0.21	0.18	0.18	0.07	0.05	0.05	0.02	0.02	0.01
Abutment Active	0.77	0.72	0.67	0.02	0.01	<0.01	N/A	N/A	N/A	N/A	N/A	N/A
Abutment Passive	0.29	0.24	0.31	0.03	0.02	0.07	N/A	N/A	N/A	N/A	N/A	N/A
Abutment Transverse	0.09	0.10	0.12	<0.01	<0.01	<0.01	N/A	N/A	N/A	N/A	N/A	N/A
Deck	0.64	0.84	0.93	0.17	0.37	0.57	0.03	0.11	0.21	<0.01	<0.01	<0.01
System	0.99	1.0	1.0	0.34	0.48	0.66	0.09	0.15	0.25	0.02	<0.01	0.01

APPENDIX - C PSDMs, CORRELATION MATRICES AND FRAGILITY ESTIMATES FOR INTEGRAL AND JOINTED COMPARISON

ab_A :	Abutment deformation demand in active direction
ab_P :	Abutment deformation demand in passive direction
ab_T :	Abutment deformation demand in transverse direction
φ_{slight} :	Column curvature demand to capacity ratio at slight damage state
$\varphi_{moderate}$:	Column curvature demand to capacity ratio at slight damage state
$\varphi_{extensive}$:	Column curvature demand to capacity ratio at slight damage state
$\varphi_{complete}$:	Column curvature demand to capacity ratio at slight damage state
$brng$:	Elastomeric bearing deformation demand
$s-key$:	Shear key deformation demand
$unseat$:	Deck unseating demand

Table C-1 Integral bridge PSDM: 52 m bridge 6.9 m column

$\ln(S_D) = \ln(a) + b \cdot \ln(PGA)$								
Linear Regression Lines								
Response	$\ln(a)$	b	$\beta_{D IM}$					R^2
$\ln(ab_A)$	3.925	1.054	0.342					0.88
$\ln(ab_P)$	3.902	1.061	0.346					0.88
$\ln(ab_T)$	3.853	1.044	0.359					0.862
Bilinear Regression Lines								
1 st Regression Line					2 nd Regression Line			
	$\ln(a)$	b	$\beta_{D IM}$	2 nd start pnt	$\ln(a)$	b	$\beta_{D IM}$	R^2
$\ln(\varphi_{slight})$	-0.076	0.860	0.228	0.382	0.637	1.561	0.228	0.92
$\ln(\varphi_{moderate})$	-1.089	0.867	0.231	0.382	-0.379	1.598	0.358	0.92
$\ln(\varphi_{extensive})$	-2.090	0.860	0.231	0.354	-1.525	1.380	0.344	0.91
$\ln(\varphi_{complete})$	-2.579	0.858	0.234	0.354	1.366	-2.023	0.347	0.91

Table C-2 Jointed bridge PSDM: 52 m bridge 6.9 m column

$\ln(S_D) = \ln(a) + b \cdot \ln(PGA)$				
Linear Regression Lines				
Response	$\ln(a)$	b	$\beta_{D IM}$	R^2
$\ln(ab_A)$	0.864	0.122	0.378	0.07
$\ln(ab_P)$	4.726	0.1631	1.172	0.59
$\ln(ab_T)$	3.625	1.192	0.810	0.63
$\ln(brng)$	5.225	0.962	0.517	0.73
$\ln(s-key)$	4.602	0.900	0.811	0.48
$\ln(\varphi_{slight})$	0.159	0.980	0.533	0.71
$\ln(\varphi_{moderate})$	-0.872	0.980	0.517	0.72
$\ln(\varphi_{extensive})$	-1.877	0.971	0.525	0.72
$\ln(\varphi_{complete})$	-2.368	0.968	0.528	0.71
$\ln(unseat)$	5.215	0.987	0.494	0.75

Table C-3 Integral bridge correlation matrix: 52 m bridge 6.9 m column

	$\ln(ab_A \ \& \ ab_P)$	$\ln(ab_T)$	$\ln(\varphi_{slight})$	$\ln(\varphi_{moderate})$	$\ln(\varphi_{complete})$
$\ln(ab_A \ \& \ ab_P)$	1.000	0.911	0.961	0.963	0.960
$\ln(ab_T)$	0.911	1.000	0.965	0.967	0.964
$\ln(\varphi_{slight})$	0.961	0.965	1.000	0.996	0.992
$\ln(\varphi_{moderate})$	0.963	0.967	0.996	1.000	0.995
$\ln(\varphi_{complete})$	0.960	0.964	0.992	0.995	1.000

Table C-4 Jointed bridge correlation matrix: 52 m bridge 6.9 m column

	$\ln(ab_A)$	$\ln(ab_P)$	$\ln(ab_T)$	$\ln(brng)$	$\ln(skey)$	$\ln(\varphi_{slight})$	$\ln(\varphi_{mod})$	$\ln(\varphi_{comp})$	$\ln(unseat)$
$\ln(ab_A)$	1.000	0.424	-0.062	0.144	0.050	0.474	0.458	0.468	0.209
$\ln(ab_P)$	0.424	1.000	0.427	0.629	0.457	0.835	0.840	0.841	0.676
$\ln(ab_T)$	-0.062	0.427	1.000	0.689	0.559	0.536	0.549	0.550	0.647
$\ln(brng)$	0.144	0.629	0.689	1.000	0.863	0.787	0.797	0.785	0.984
$\ln(skey)$	0.050	0.457	0.559	0.863	1.000	0.623	0.632	0.614	0.836
$\ln(\varphi_{slight})$	0.474	0.835	0.536	0.787	0.623	1.000	0.997	0.993	0.793
$\ln(\varphi_{mod})$	0.458	0.840	0.549	0.797	0.632	0.997	1.000	0.996	0.804
$\ln(\varphi_{comp})$	0.468	0.841	0.550	0.785	0.614	0.993	0.996	1.000	0.793
$\ln(unseat)$	0.209	0.676	0.647	0.984	0.836	0.793	0.804	0.793	1.000

Table C-5 Seismic fragility estimates: 52 m bridge 6.9 m column

	Slight		Moderate		Extensive		Complete	
	Int	Joint	Int	Joint	Int	Joint	Int	Joint
10% in 50 yr Earthquake								
Columns	0.19	0.21	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01
Abutment Active	0.47	<0.01	<0.01	<0.01	N/A	N/A	N/A	N/A
Abutment Passive	<0.01	0.27	<0.01	0.07	N/A	N/A	N/A	N/A
Abutment Transverse	<0.01	0.04	<0.01	<0.01	N/A	N/A	N/A	N/A
Elastomeric Bearings	N/A	0.98	N/A	0.46	N/A	N/A	N/A	N/A
Shear Keys	N/A	0.65	N/A	0.16	N/A	N/A	N/A	N/A
Deck Unseating	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
System	0.50	0.98	<0.01	0.49	<0.01	<0.01	<0.01	<0.01
2% in 50 yr Earthquake								
Columns	0.88	0.58	0.20	0.07	<0.01	<0.01	<0.01	<0.01
Abutment Active	0.91	<0.01	0.07	<0.01	N/A	N/A	N/A	N/A
Abutment Passive	0.17	0.61	<0.01	0.28	N/A	N/A	N/A	N/A
Abutment Transverse	0.15	0.20	<0.01	0.02	N/A	N/A	N/A	N/A
Elastomeric Bearings	N/A	1.0	N/A	0.82	N/A	N/A	N/A	N/A
Shear Keys	N/A	0.85	N/A	0.38	N/A	N/A	N/A	N/A
Deck Unseating	N/A	N/A	N/A	N/A	N/A	N/A	N/A	0.02
System	0.96	0.1	0.23	0.83	<0.01	0.02	<0.01	0.02

Table C-6 Integral bridge PSDM: 52 m bridge 9.5 m column

$\ln(S_D) = \ln(a) + b \cdot \ln(PGA)$								
Linear Regression Lines								
Response	$\ln(a)$	b	$\beta_{D IM}$	R ²				
$\ln(ab_A)$	3.945	2.003	0.316	0.88				
$\ln(ab_P)$	3.920	1.009	0.321	0.88				
$\ln(ab_T)$	4.062	1.076	0.342	0.88				
Bilinear Regression Lines								
	1 st Regression Line			2 nd Regression Line				
	$\ln(a)$	b	$\beta_{D IM}$	2 nd start pnt	$\ln(a)$	b	$\beta_{D IM}$	R ²
$\ln(\varphi_{slight})$	-0.635	0.789	0.221	0.332	0.014	1.349	0.361	0.91
$\ln(\varphi_{moderate})$	-1.645	0.797	0.216	0.354	-0.993	1.389	0.354	0.91
$\ln(\varphi_{extensive})$	-2.640	0.792	0.220	0.354	-2.010	1.365	0.363	0.90
$\ln(\varphi_{complete})$	-3.128	0.791	0.226	0.354	-2.503	1.359	0.367	0.90

Table C-7 Jointed bridge PSDM: 52 m bridge 9.5 m column

$\ln(S_D) = \ln(a) + b \cdot \ln(PGA)$				
Linear Regression Lines				
Response	$\ln(a)$	b	$\beta_{D IM}$	R^2
$\ln(ab_A)$	0.835	0.089	0.364	0.04
$\ln(ab_P)$	4.610	1.475	1.123	0.56
$\ln(ab_T)$	3.629	1.146	0.763	0.64
$\ln(brng)$	5.097	0.890	0.451	0.74
$\ln(s-key)$	4.374	0.797	0.781	0.44
$\ln(\varphi_{slight})$	0.167	1.011	0.601	0.67
$\ln(\varphi_{moderate})$	-0.861	1.012	0.581	0.68
$\ln(\varphi_{extensive})$	-1.868	1.001	0.588	0.67
$\ln(\varphi_{complete})$	-2.359	0.998	0.592	0.67
$\ln(unseat)$	5.014	0.868	0.716	0.469

Table C-8 Integral bridge correlation matrix: 52 m bridge 9.5 m column

	$\ln(ab_A \ \& \ ab_P)$	$\ln(ab_T)$	$\ln(\varphi_{slight})$	$\ln(\varphi_{moderate})$	$\ln(\varphi_{complete})$
$\ln(ab_A \ \& \ ab_P)$	1.000	0.909	0.962	0.965	0.963
$\ln(ab_T)$	0.909	1.000	0.962	0.962	0.956
$\ln(\varphi_{slight})$	0.962	0.962	1.000	0.996	0.992
$\ln(\varphi_{moderate})$	0.965	0.962	0.996	1.000	0.995
$\ln(\varphi_{complete})$	0.963	0.956	0.992	0.995	1.000

Table C-9 Jointed bridge correlation matrix: 52 m bridge 9.5 m column

	$\ln(ab_A)$	$\ln(ab_P)$	$\ln(ab_T)$	$\ln(brng)$	$\ln(skey)$	$\ln(\varphi_{slight})$	$\ln(\varphi_{mod})$	$\ln(\varphi_{comp})$	$\ln(unseat)$
$\ln(ab_A)$	1.000	0.313	0.017	0.189	0.041	0.223	0.203	0.208	0.196
$\ln(ab_P)$	0.313	1.000	0.533	0.671	0.516	0.773	0.774	0.772	0.678
$\ln(ab_T)$	0.017	0.533	1.000	0.732	0.588	0.659	0.670	0.675	0.700
$\ln(brng)$	0.189	0.671	0.732	1.000	0.808	0.899	0.893	0.886	0.986
$\ln(skey)$	0.041	0.516	0.588	0.808	1.000	0.770	0.771	0.762	0.783
$\ln(\varphi_{slight})$	0.223	0.773	0.659	0.899	0.770	1.000	0.997	0.994	0.897
$\ln(\varphi_{mod})$	0.203	0.774	0.670	0.893	0.771	0.997	1.000	0.996	0.890
$\ln(\varphi_{comp})$	0.208	0.772	0.675	0.886	0.762	0.994	0.996	1.000	0.884
$\ln(unseat)$	0.196	0.678	0.700	0.986	0.783	0.897	0.890	0.884	1.000

Table C-10 Seismic fragility estimates: 52 m bridge 9.5 m column

	Slight		Moderate		Extensive		Complete	
	Int	Joint	Int	Joint	Int	Joint	Int	Joint
10% in 50 yr Earthquake								
Columns	0.03	0.22	<0.01	0.01	<0.01	<0.01	<0.01	<0.01
Abutment Active	0.51	<0.01	<0.01	<0.01	N/A	N/A	N/A	N/A
Abutment Passive	<0.01	0.26	<0.01	0.06	N/A	N/A	N/A	N/A
Abutment Transverse	0.02	0.04	<0.01	<0.01	N/A	N/A	N/A	N/A
Elastomeric Bearings	N/A	0.99	N/A	0.40	N/A	N/A	N/A	N/A
Shear Keys	N/A	0.58	N/A	0.12	N/A	N/A	N/A	N/A
Deck Unseating	N/A	N/A	N/A	N/A	N/A	N/A	N/A	<0.01
System	0.49	0.99	<0.01	0.43	<0.01	<0.01	<0.01	<0.01
2% in 50 yr Earthquake								
Columns	0.48	0.58	0.02	0.09	<0.01	<0.01	<0.01	<0.01
Abutment Active	0.92	<0.01	0.07	<0.01	N/A	N/A	N/A	N/A
Abutment Passive	0.18	0.58	<0.01	0.24	N/A	N/A	N/A	N/A
Abutment Transverse	0.27	0.19	<0.01	0.01	N/A	N/A	N/A	N/A
Elastomeric Bearings	N/A	1.0	N/A	0.78	N/A	N/A	N/A	N/A
Shear Keys	N/A	0.80	N/A	0.28	N/A	N/A	N/A	N/A
Deck Unseating	N/A	N/A	N/A	N/A	N/A	N/A	N/A	<0.01
System	0.94	1.0	0.08	0.80	<0.01	<0.01	<0.01	<0.01

Table C-11 Integral bridge PSDM: 52 m bridge 12 m column

$\ln(S_D) = \ln(a) + b \cdot \ln(PGA)$								
Linear Regression Lines								
Response	$\ln(a)$		b		$\beta_{D IM}$		R^2	
$\ln(ab_A)$	3.995		0.983		0.330		0.87	
$\ln(ab_P)$	3.971		0.988		0.335		0.86	
$\ln(ab_T)$	4.112		1.069		0.360		0.87	
Bilinear Regression Lines								
	1 st Regression Line				2 nd Regression Line			
	$\ln(a)$	b	$\beta_{D IM}$	2 nd start pnt	$\ln(a)$	b	$\beta_{D IM}$	R^2
$\ln(\varphi_{slight})$	-1.009	0.766	0.195	0.332	-0.253	1.390	0.307	0.92
$\ln(\varphi_{moderate})$	-2.035	0.768	0.214	0.332	-1.267	1.401	0.317	0.92
$\ln(\varphi_{extensive})$	-3.027	0.765	0.221	0.332	-2.287	1.375	0.336	0.92
$\ln(\varphi_{complete})$	-3.514	0.764	0.226	0.341	-2.781	1.369	0.341	0.92

Table C-12 Jointed bridge PSDM: 52 m bridge 12 m column

$\ln(S_D) = \ln(a) + b \cdot \ln(PGA)$				
Linear Regression Lines				
Response	$\ln(a)$	b	$\beta_{D IM}$	R^2
$\ln(ab_A)$	0.884	0.097	0.329	0.06
$\ln(ab_P)$	4.856	1.578	1.120	0.59
$\ln(ab_T)$	3.707	1.146	0.781	0.63
$\ln(brng)$	5.086	0.855	0.500	0.69
$\ln(s-key)$	4.583	0.881	0.801	0.47
$\ln(\varphi_{slight})$	0.250	1.092	0.646	0.68
$\ln(\varphi_{moderate})$	-0.893	1.039	0.620	0.67
$\ln(\varphi_{extensive})$	-1.900	1.029	0.621	0.67
$\ln(\varphi_{complete})$	-2.391	1.026	0.623	0.67
$\ln(unseat)$	5.009	0.849	0.522	0.66

Table C-13 Integral bridge correlation matrix: 52 m bridge 12 m column

	$\ln(ab_A \ \& \ ab_P)$	$\ln(ab_T)$	$\ln(\varphi_{slight})$	$\ln(\varphi_{moderate})$	$\ln(\varphi_{complete})$
$\ln(ab_A \ \& \ ab_P)$	1.000	0.905	0.960	0.966	0.964
$\ln(ab_T)$	0.905	1.000	0.963	0.961	0.956
$\ln(\varphi_{slight})$	0.960	0.963	1.000	0.996	0.992
$\ln(\varphi_{moderate})$	0.966	0.961	0.996	1.000	0.995
$\ln(\varphi_{complete})$	0.964	0.956	0.992	0.995	1.000

Table C-14 Jointed bridge correlation matrix: 52 m bridge 12 m column

	$\ln(ab_A)$	$\ln(ab_P)$	$\ln(ab_T)$	$\ln(brng)$	$\ln(skey)$	$\ln(\varphi_{slight})$	$\ln(\varphi_{mod})$	$\ln(\varphi_{comp})$	$\ln(unseat)$
$\ln(ab_A)$	1.000	0.210	0.065	0.178	0.066	0.277	0.263	0.268	0.258
$\ln(ab_P)$	0.210	1.000	0.548	0.744	0.603	0.849	0.852	0.849	0.758
$\ln(ab_T)$	0.065	0.548	1.000	0.649	0.510	0.603	0.612	0.615	0.616
$\ln(brng)$	0.178	0.744	0.649	1.000	0.842	0.907	0.909	0.902	0.983
$\ln(skey)$	0.066	0.603	0.510	0.842	1.000	0.794	0.801	0.797	0.835
$\ln(\varphi_{slight})$	0.277	0.849	0.603	0.907	0.794	1.000	0.997	0.995	0.907
$\ln(\varphi_{mod})$	0.263	0.852	0.612	0.909	0.801	0.997	1.000	0.997	0.906
$\ln(\varphi_{comp})$	0.268	0.849	0.615	0.902	0.797	0.995	0.997	1.000	0.903
$\ln(unseat)$	0.258	0.758	0.616	0.983	0.835	0.907	0.906	0.903	1.000

Table C-15 Seismic fragility estimates: 52 m bridge 12 m column

	Slight		Moderate		Extensive		Complete	
	Int	Joint	Int	Joint	Int	Joint	Int	Joint
10% in 50 yr Earthquake								
Columns	<0.01	0.25	<0.01	0.01	<0.01	<0.01	<0.01	<0.01
Abutment Active	0.57	<0.01	<0.01	<0.01	N/A	N/A	N/A	N/A
Abutment Passive	0.02	0.31	<0.01	0.08	N/A	N/A	N/A	N/A
Abutment Transverse	0.03	0.05	<0.01	<0.01	N/A	N/A	N/A	N/A
Elastomeric Bearings	N/A	0.98	N/A	0.42	N/A	N/A	N/A	N/A
Shear Keys	N/A	0.65	N/A	0.16	N/A	N/A	N/A	N/A
Deck Unseating	N/A	N/A	N/A	N/A	N/A	N/A	N/A	<0.01
System	0.55	0.98	<0.01	0.46	<0.01	<0.01	<0.01	<0.01
2% in 50 yr Earthquake								
Columns	0.26	0.62	<0.01	0.10	<0.01	<0.01	<0.01	<0.01
Abutment Active	0.94	<0.01	0.09	<0.01	N/A	N/A	N/A	N/A
Abutment Passive	0.21	0.66	<0.01	0.31	N/A	N/A	N/A	N/A
Abutment Transverse	0.31	0.22	<0.01	0.02	N/A	N/A	N/A	N/A
Elastomeric Bearings	N/A	1.0	N/A	0.76	N/A	N/A	N/A	N/A
Shear Keys	N/A	0.85	N/A	0.37	N/A	N/A	N/A	N/A
Deck Unseating	N/A	N/A	N/A	N/A	N/A	N/A	N/A	0.01
System	0.94	1.0	0.09	0.81	<0.01	0.01	<0.01	0.01

Table C-16 Integral bridge PSDM: 69 m bridge 6.9 m column

$\ln(S_D) = \ln(a) + b \cdot \ln(PGA)$								
Linear Regression Lines								
Response	$\ln(a)$		b		$\beta_{D IM}$		R^2	
$\ln(ab_A)$	3.974		0.946		0.339		0.86	
$\ln(ab_P)$	3.942		0.953		0.344		0.85	
$\ln(ab_T)$	4.101		1.051		0.379		0.86	
Bilinear Regression Lines								
	1 st Regression Line				2 nd Regression Line			
	$\ln(a)$	b	$\beta_{D IM}$	2 nd start pnt	$\ln(a)$	b	$\beta_{D IM}$	R^2
$\ln(\varphi_{slight})$	0.166	0.752	0.222	0.556	1.191	2.451	0.414	0.92
$\ln(\varphi_{moderate})$	-0.924	0.736	0.221	0.556	0.099	2.378	0.341	0.92
$\ln(\varphi_{extensive})$	-1.981	0.672	0.226	0.382	-0.785	1.908	0.356	0.92
$\ln(\varphi_{complete})$	-2.451	0.671	0.233	0.382	-1.247	1.918	0.358	0.91

Table C-17 Jointed bridge PSDM: 69 m bridge 6.9 m column

$\ln(S_D) = \ln(a) + b \cdot \ln(PGA)$				
Linear Regression Lines				
Response	$\ln(a)$	b	$\beta_{D IM}$	R^2
$\ln(ab_A)$	0.813	0.060	0.344	0.02
$\ln(ab_P)$	0.490	1.555	1.127	0.58
$\ln(ab_T)$	3.562	1.124	0.796	0.60
$\ln(brng)$	5.137	0.872	0.539	0.64
$\ln(s-key)$	4.079	0.660	0.893	0.28
$\ln(\varphi_{slight})$	0.155	0.983	0.520	0.72
$\ln(\varphi_{moderate})$	-0.940	0.965	0.510	0.72
$\ln(\varphi_{extensive})$	-1.861	0.959	0.509	0.72
$\ln(\varphi_{complete})$	-2.331	0.958	0.511	0.72
$\ln(unseat)$	5.11	0.872	0.533	0.65

Table C-18 Integral bridge correlation matrix: 69 m bridge 6.9 m column

	$\ln(ab_A \ \& \ ab_P)$	$\ln(ab_T)$	$\ln(\varphi_{slight})$	$\ln(\varphi_{moderate})$	$\ln(\varphi_{complete})$
$\ln(ab_A \ \& \ ab_P)$	1.000	0.887	0.942	0.943	0.942
$\ln(ab_T)$	0.887	1.000	0.957	0.961	0.960
$\ln(\varphi_{slight})$	0.942	0.957	1.000	0.995	0.994
$\ln(\varphi_{moderate})$	0.943	0.961	0.995	1.000	0.994
$\ln(\varphi_{complete})$	0.942	0.960	0.994	0.994	1.000

Table C-19 Jointed bridge correlation matrix: 69 m bridge 6.9 m column

	$\ln(ab_A)$	$\ln(ab_P)$	$\ln(ab_T)$	$\ln(brng)$	$\ln(skey)$	$\ln(\varphi_{slight})$	$\ln(\varphi_{mod})$	$\ln(\varphi_{comp})$	$\ln(unseat)$
$\ln(ab_A)$	1.000	0.293	0.344	0.382	0.335	0.354	0.350	0.371	0.363
$\ln(ab_P)$	0.293	1.000	0.717	0.777	0.614	0.832	0.831	0.829	0.790
$\ln(ab_T)$	0.344	0.717	1.000	0.746	0.600	0.830	0.826	0.836	0.744
$\ln(brng)$	0.382	0.777	0.746	1.000	0.806	0.896	0.896	0.897	0.998
$\ln(skey)$	0.335	0.614	0.600	0.806	1.000	0.703	0.701	0.698	0.799
$\ln(\varphi_{slight})$	0.354	0.832	0.830	0.896	0.703	1.000	0.996	0.994	0.902
$\ln(\varphi_{mod})$	0.350	0.831	0.826	0.896	0.701	0.996	1.000	0.995	0.903
$\ln(\varphi_{comp})$	0.371	0.829	0.836	0.897	0.698	0.994	0.995	1.000	0.903
$\ln(unseat)$	0.363	0.790	0.744	0.998	0.799	0.902	0.903	0.903	1.000

Table C-20 Seismic fragility estimates: 69 m bridge 6.9 m column

	Slight		Moderate		Extensive		Complete	
	Int	Joint	Int	Joint	Int	Joint	Int	Joint
10% in 50 yr Earthquake								
Columns	0.20	0.20	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01
Abutment Active	0.57	<0.01	<0.01	<0.01	N/A	N/A	N/A	N/A
Abutment Passive	0.02	0.33	<0.01	0.09	N/A	N/A	N/A	N/A
Abutment Transverse	0.03	0.04	<0.01	<0.01	N/A	N/A	N/A	N/A
Elastomeric Bearings	N/A	0.98	N/A	0.45	N/A	N/A	N/A	N/A
Shear Keys	N/A	0.49	N/A	0.10	N/A	N/A	N/A	N/A
Deck Unseating	N/A	N/A	N/A	N/A	N/A	N/A	N/A	<0.01
System	0.60	0.98	<0.01	0.47	<0.01	<0.01	<0.01	<0.01
2% in 50 yr Earthquake								
Columns	0.98	0.58	0.52	0.06	0.05	<0.01	<0.01	<0.01
Abutment Active	0.93	<0.01	0.09	<0.01	N/A	N/A	N/A	N/A
Abutment Passive	0.20	0.67	<0.01	0.32	N/A	N/A	N/A	N/A
Abutment Transverse	0.30	0.18	<0.01	0.02	N/A	N/A	N/A	N/A
Elastomeric Bearings	N/A	1.0	N/A	0.78	N/A	N/A	N/A	N/A
Shear Keys	N/A	0.67	N/A	0.21	N/A	N/A	N/A	N/A
Deck Unseating	N/A	N/A	N/A	N/A	N/A	N/A	N/A	0.02
System	0.99	1.0	0.53	0.79	0.04	0.02	<0.01	0.02

Table C-21 Integral bridge PSDM: 69 m bridge 9.5 m column

$\ln(S_D) = \ln(a) + b \cdot \ln(PGA)$								
Linear Regression Lines								
Response	$\ln(a)$		b		$\beta_{D IM}$		R^2	
$\ln(ab_A)$	4.202		1.013		0.339		0.87	
$\ln(ab_P)$	4.172		1.021		0.344		0.87	
$\ln(ab_T)$	4.357		1.127		0.399		0.86	
Bilinear Regression Lines								
	1 st Regression Line				2 nd Regression Line			
	$\ln(a)$	b	$\beta_{D IM}$	2 nd start pnt	$\ln(a)$	b	$\beta_{D IM}$	R^2
$\ln(\varphi_{slight})$	-0.412	0.681	0.218	0.382	0.806	1.948	0.382	0.91
$\ln(\varphi_{moderate})$	-1.518	0.658	0.212	0.382	-0.190	2.024	0.372	0.92
$\ln(\varphi_{extensive})$	-2.443	0.651	0.210	0.382	-1.778	1.778	0.302	0.91
$\ln(\varphi_{complete})$	-2.913	0.650	0.215	0.382	-1.781	1.781	0.303	0.91

Table C-22 Jointed bridge PSDM: 69 m bridge 9.5 m column

$\ln(S_D) = \ln(a) + b \cdot \ln(PGA)$				
Linear Regression Lines				
Response	$\ln(a)$	b	$\beta_{D IM}$	R^2
$\ln(ab_A)$	0.853	0.074	0.350	0.03
$\ln(ab_P)$	4.750	1.484	1.165	0.543
$\ln(ab_T)$	3.799	1.198	0.62	0.828
$\ln(brng)$	5.117	0.852	0.499	0.68
$\ln(s-key)$	4.361	0.755	0.789	0.41
$\ln(\varphi_{slight})$	0.164	1.042	0.531	0.73
$\ln(\varphi_{moderate})$	-0.920	1.030	0.524	0.74
$\ln(\varphi_{extensive})$	-1.767	1.058	0.549	0.73
$\ln(\varphi_{complete})$	-2.321	1.017	0.532	0.72
$\ln(unseat)$	5.042	0.824	0.487	0.68

Table C-23 Integral bridge correlation matrix: 69 m bridge 9.5 m column

	$\ln(ab_A \ \& \ ab_P)$	$\ln(ab_T)$	$\ln(\varphi_{slight})$	$\ln(\varphi_{moderate})$	$\ln(\varphi_{complete})$
$\ln(ab_A \ \& \ ab_P)$	1.000	0.904	0.954	0.954	0.952
$\ln(ab_T)$	0.904	1.000	0.960	0.962	0.959
$\ln(\varphi_{slight})$	0.954	0.960	1.000	0.996	0.994
$\ln(\varphi_{moderate})$	0.954	0.962	0.996	1.000	0.995
$\ln(\varphi_{complete})$	0.952	0.959	0.994	0.995	1.000

Table C-24 Jointed bridge correlation matrix: 69 m bridge 9.5 m column

	$\ln(ab_A)$	$\ln(ab_P)$	$\ln(ab_T)$	$\ln(brng)$	$\ln(skey)$	$\ln(\varphi_{slight})$	$\ln(\varphi_{mod})$	$\ln(\varphi_{comp})$	$\ln(unseat)$
$\ln(ab_A)$	1.000	0.339	0.310	0.442	0.275	0.333	0.335	0.352	0.466
$\ln(ab_P)$	0.339	1.000	0.658	0.769	0.564	0.811	0.811	0.810	0.780
$\ln(ab_T)$	0.310	0.658	1.000	0.737	0.551	0.761	0.757	0.764	0.742
$\ln(brng)$	0.442	0.769	0.737	1.000	0.775	0.899	0.903	0.900	0.995
$\ln(skey)$	0.275	0.564	0.551	0.775	1.000	0.725	0.725	0.718	0.760
$\ln(\varphi_{slight})$	0.333	0.811	0.761	0.899	0.725	1.000	0.996	0.995	0.881
$\ln(\varphi_{mod})$	0.335	0.811	0.757	0.903	0.725	0.996	1.000	0.996	0.885
$\ln(\varphi_{comp})$	0.352	0.810	0.764	0.900	0.718	0.995	0.996	1.000	0.882
$\ln(unseat)$	0.466	0.780	0.742	0.995	0.760	0.881	0.885	0.882	1.000

Table C-25 Seismic fragility estimates: 69 m bridge 9.5 m column

	Slight		Moderate		Extensive		Complete	
	Int	Joint	Int	Joint	Int	Joint	Int	Joint
10% in 50 yr Earthquake								
Columns	0.15	0.19	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01
Abutment Active	0.71	<0.01	0.01	<0.01	N/A	N/A	N/A	N/A
Abutment Passive	0.04	0.31	<0.01	0.09	N/A	N/A	N/A	N/A
Abutment Transverse	0.08	0.06	<0.01	<0.01	N/A	N/A	N/A	N/A
Elastomeric Bearings	N/A	0.98	N/A	0.44	N/A	N/A	N/A	N/A
Shear Keys	N/A	0.59	N/A	0.12	N/A	N/A	N/A	N/A
Deck Unseating	N/A	N/A	N/A	N/A	N/A	N/A	N/A	<0.01
System	0.70	0.99	0.01	0.47	<0.01	<0.01	<0.01	<0.01
2% in 50 yr Earthquake								
Columns	0.93	0.58	0.31	0.07	<0.01	<0.01	<0.01	<0.01
Abutment Active	0.97	<0.01	0.19	<0.01	N/A	N/A	N/A	N/A
Abutment Passive	0.35	0.62	<0.01	0.29	N/A	N/A	N/A	N/A
Abutment Transverse	0.49	0.26	0.02	0.04	N/A	N/A	N/A	N/A
Elastomeric Bearings	N/A	1.0	N/A	0.78	N/A	N/A	N/A	N/A
Shear Keys	N/A	0.79	N/A	0.28	N/A	N/A	N/A	N/A
Deck Unseating	N/A	N/A	N/A	N/A	N/A	N/A	N/A	0.01
System	0.99	1.0	0.40	0.80	<0.01	0.01	<0.01	0.01

Table C-26 Integral bridge PSDM: 69 m bridge 12 m column

$\ln(S_D) = \ln(a) + b \cdot \ln(PGA)$								
Linear Regression Lines								
Response	$\ln(a)$		b		$\beta_{D IM}$		R^2	
$\ln(ab_A)$	4.318		1.039		0.351		0.87	
$\ln(ab_P)$	4.289		1.046		0.358		0.86	
$\ln(ab_T)$	4.495		1.160		0.409		0.86	
Bilinear Regression Lines								
	1 st Regression Line				2 nd Regression Line			
	$\ln(a)$	b	$\beta_{D IM}$	2 nd start pnt	$\ln(a)$	b	$\beta_{D IM}$	R^2
$\ln(\varphi_{slight})$	-0.809	0.659	0.209	0.354	1.801	0.403	0.391	0.91
$\ln(\varphi_{moderate})$	-1.924	0.632	0.212	0.354	-0.681	1.761	0.381	0.90
$\ln(\varphi_{extensive})$	-2.856	0.622	0.213	0.332	-1.601	1.762	0.364	0.91
$\ln(\varphi_{complete})$	-3.329	0.619	0.217	0.332	-2.072	1.758	0.362	0.91

Table C-27 Jointed bridge PSDM: 69 m bridge 12 m column

$\ln(S_D) = \ln(a) + b \cdot \ln(PGA)$				
Linear Regression Lines				
Response	$\ln(a)$	b	$\beta_{D IM}$	R^2
$\ln(ab_A)$	0.741	0.031	0.375	0.01
$\ln(ab_P)$	4.781	1.479	1.208	0.52
$\ln(ab_T)$	3.909	1.204	0.802	0.63
$\ln(brng)$	5.032	0.798	0.650	0.53
$\ln(s-key)$	4.421	0.764	0.882	0.36
$\ln(\varphi_{slight})$	-0.052	0.955	0.640	0.60
$\ln(\varphi_{moderate})$	-1.017	0.997	0.673	0.61
$\ln(\varphi_{extensive})$	-2.064	0.933	0.636	0.60
$\ln(\varphi_{complete})$	-2.536	0.931	0.637	0.59
$\ln(unseat)$	5.086	0.837	0.602	0.59

Table C-28 Integral bridge correlation matrix: 69 m bridge 12 m column

	$\ln(ab_A \ \& \ ab_P)$	$\ln(ab_T)$	$\ln(\varphi_{slight})$	$\ln(\varphi_{moderate})$	$\ln(\varphi_{complete})$
$\ln(ab_A \ \& \ ab_P)$	1.000	0.915	0.961	0.959	0.959
$\ln(ab_T)$	0.915	1.000	0.964	0.965	0.962
$\ln(\varphi_{slight})$	0.961	0.964	1.000	0.996	0.994
$\ln(\varphi_{moderate})$	0.959	0.965	0.996	1.000	0.995
$\ln(\varphi_{complete})$	0.959	0.962	0.994	0.995	1.000

Table C-29 Jointed bridge correlation matrix: 69 m bridge 12 m column

	$\ln(ab_A)$	$\ln(ab_P)$	$\ln(ab_T)$	$\ln(brng)$	$\ln(skey)$	$\ln(\varphi_{slight})$	$\ln(\varphi_{mod})$	$\ln(\varphi_{comp})$	$\ln(unseat)$
$\ln(ab_A)$	1.000	0.375	0.246	0.457	0.294	0.404	0.406	0.419	0.460
$\ln(ab_P)$	0.375	1.000	0.668	0.763	0.614	0.838	0.840	0.837	0.776
$\ln(ab_T)$	0.246	0.668	1.000	0.726	0.595	0.726	0.724	0.734	0.699
$\ln(brng)$	0.457	0.763	0.726	1.000	0.813	0.899	0.902	0.904	0.987
$\ln(skey)$	0.294	0.614	0.595	0.813	1.000	0.789	0.790	0.792	0.794
$\ln(\varphi_{slight})$	0.404	0.838	0.726	0.899	0.789	1.000	0.997	0.996	0.892
$\ln(\varphi_{mod})$	0.406	0.840	0.724	0.902	0.790	0.997	1.000	0.996	0.894
$\ln(\varphi_{comp})$	0.419	0.837	0.734	0.904	0.792	0.996	0.996	1.000	0.896
$\ln(unseat)$	0.460	0.776	0.699	0.987	0.794	0.892	0.894	0.896	1.000

Table C-30 Seismic fragility estimates: 69 m bridge 12 m column

	Slight		Moderate		Extensive		Complete	
	Int	Joint	Int	Joint	Int	Joint	Int	Joint
10% in 50 yr Earthquake								
Columns	0.05	0.16	<0.01	0.01	<0.01	<0.01	<0.01	<0.01
Abutment Active	0.77	<0.01	0.02	<0.01	N/A	N/A	N/A	N/A
Abutment Passive	0.06	0.32	<0.01	0.10	N/A	N/A	N/A	N/A
Abutment Transverse	0.12	0.08	<0.01	<0.01	N/A	N/A	N/A	N/A
Elastomeric Bearings	N/A	0.95	N/A	0.42	N/A	N/A	N/A	N/A
Shear Keys	N/A	0.60	N/A	0.16	N/A	N/A	N/A	N/A
Deck Unseating	N/A	N/A	N/A	N/A	N/A	N/A	N/A	<0.01
System	0.76	0.96	0.02	0.46	<0.01	<0.01	<0.01	<0.01
2% in 50 yr Earthquake								
Columns	0.75	0.46	0.08	0.08	<0.01	<0.01	<0.01	<0.01
Abutment Active	0.98	<0.01	0.26	<0.01	N/A	N/A	N/A	N/A
Abutment Passive	0.44	0.63	<0.01	0.30	N/A	N/A	N/A	N/A
Abutment Transverse	0.59	0.30	0.04	0.04	N/A	N/A	N/A	N/A
Elastomeric Bearings	N/A	0.99	N/A	0.70	N/A	N/A	N/A	N/A
Shear Keys	N/A	0.79	N/A	0.32	N/A	N/A	N/A	N/A
Deck Unseating	N/A	N/A	N/A	N/A	N/A	N/A	N/A	0.02
System	0.99	0.99	0.29	0.73	<0.01	0.03	<0.01	0.02