Seismic Fragility Analysis for Highway Bridges with Consideration of Soil-Structure Interaction and Deterioration

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SEISMIC FRAGILITY ANALYSIS FOR HIGHWAY BRIDGES WITH
CONSIDERATION OF SOIL-STRUCTURE INTERACTION AND
DETERIORATION

By

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THE CITY UNIVERSITY OF NEW YORK
Abstract

Seismic Fragility Analysis for Highway Bridges with Consideration of Soil-Structure Interaction and Deterioration

By

Xin Zong

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Co-Supervisor: Professor Huabei Liu

Bridges are critical elements within the highway transportation network. It is very important for the owner or designer to perform the risk assessment of the highway bridges during extreme events, such as earthquakes, due to their importance to the network, commerce, economic vitality and mobility. Recent studies show that seismic fragility curves are useful tools for the seismic risk assessment of highway bridges.

Although general seismic fragility approach has been well established in the last two decades and numerous retrofit methods have been applied to highway bridges in New York City (NYC) metropolitan area, which has been classified as moderate seismic zone as per American Association of State Highway and Transportation Officials (AASHTO), there is a need to carry out detailed and further work on seismic fragility by considering soil-structure interaction (SSI) and deterioration effects because of the differences in ground motion characteristics, construction practices and inevitable deterioration of construction materials. The main objective of this research work is to refine existing methods for the development of analytical seismic fragility curves for bridges in NYC.
metropolitan area by introducing detailed consideration and modeling of the SSI and
deterioration of critical bridge elements.

To meet above objective, several sets of typical synthetic bedrock and ground surface
motions in NYC metropolitan region are developed in this thesis. A detailed soil material
modeling along with the sensitivity analysis of the soil properties has been investigated,
followed by the development of more reliable SSI model. General deterioration models,
for both elastomeric bearings and reinforcement steel deterioration have been constructed.

With these investigated and developed models, more realistic structural model for the
typical multi-span continuous (MSC) bridges in NYC metropolitan area has been
constructed. Nonlinear structural analysis as well as corresponding limit states and
probabilistic analysis, have been carried out using this detailed bridge model. Based on
analysis results, more realistic and reliable seismic fragility curves, which is the function
of peak ground acceleration, for bridge and its components have been developed.

The evaluation of seismic fragility curves constructed in this research work shows that
typical MSC bridges in NYC metropolitan area would benefit from the consideration of
the detailed SSI model and the risk of these bridges experiencing different extents of
damage under earthquake disaster decreases because of modeling of SSI effects.

However, when these bridges have been in service and have undergone deterioration for
20 years, the risk increases by the same level as the decrease because of inclusion of SSI
modeling.
DEDICATION

To my loving mother

and my late father, may he rest in heaven
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Chapter 1 Introduction

1.1 Background

Bridges are critical elements within the highway transportation network, supporting commerce, economic vitality, and mobility. Recent records show that unpredictable extreme events, such as earthquakes, can cause significant damage to bridges, resulting in significant loss of life and property. Considering that many existing bridges were designed without consideration of seismic effects, components of current highway transportation system are at risk of significant damages during earthquakes. This risk is increased further because of deterioration of these bridges. In order to mitigate potential life and economic losses during an earthquake, it is very important for the owner or designer of bridges to predict the extent of probable damage to highway bridges during such unexpected earthquakes.

Seismic fragility curve, defined as a conditional probability curve that gives the likelihood that a structure or its components will meet or exceed a specified level of damage during a given ground motion intensity measure, has been found to be useful tool for assessing potential damages. They are also an essential component of the seismic risk assessment procedures. Although seismic fragility has been investigated by researchers around the world, there have been very few studies that have considered the effects of deterioration and detailed soil-structure interaction (SSI) effects on seismic fragility, particularly in the New York City metropolitan region.
Although general seismic fragility analysis approach is well established, based on which numerous retrofit methods are being applied to highway bridges in NYC metropolitan area, which has been classified as moderate seismic zone as per American Association of State Highway and Transportation Officials (AASHTO), there is a need to carry out further work on seismic fragility by considering deterioration and SSI effects because of inevitable deterioration of materials, differences in ground motion characteristics and construction practices. The objective of this dissertation is to carry out detailed investigation on effects of SSI and deterioration on seismic fragility of bridges.

1.2 Objectives

The main objective of this dissertation is to refine existing methods for the development of analytical seismic fragility curves for bridges in New York City metropolitan area, which is classified as moderate seismic zone by considering the SSI effects and deterioration.

Although there have been number of studies on generating seismic fragility curves for bridges in moderate seismic zones, the majority of these studies have focused on originally designed or built bridges and bridges with simplified soil-ground model or even without consideration of SSI effects. More focus is now being placed on existing bridges, which are continuously exposed to natural environment experiencing unavoidable deterioration. Effect of soil structure interaction has not been considered during seismic design of these bridges. This dissertation combines the consideration of these effects and develops more reliable and realistic seismic fragility curves. This has been achieved through following tasks:
1. Identify the most common and typical bridge types in New York City metropolitan area. These bridges represent overall inventory for this region.

2. Identify parameters that represent effects of deterioration on bridge components and analytically evaluate these effects.

3. Generate several groups of synthetic ground motions that are representative of the seismic hazard for the New York City metropolitan region.

4. Generate detailed soil model to simulate SSI. The soil model needs to be calibrated with available experimental results.

5. Develop 3-D nonlinear bridge models using detailed parameters and component models mentioned above.

6. Construct improved seismic fragility curves based on analytical bridge models and synthetic ground motions for the New York City metropolitan region.

1.3 Outline of the Dissertation

Seismic fragility analysis with specific consideration of SSI and deterioration has been investigated in this dissertation for highway bridges in NYC metropolitan area. The results show the risk for deteriorated typical MSC bridges experiencing different extends of damage under earthquake disaster increases by the same level of decrease as benefited from the SSI consideration. The outline of the dissertation is as follows:

Chapter 1 presents a brief introduction of bridge seismic fragility analysis with consideration of SSI effects and deterioration, and describes the objectives of this dissertation.
Chapter 2 presents a detailed review of the state-of-the-art on commonly adopted bridge seismic fragility analysis approaches as well as deterioration and SSI models.

Chapter 3 presents detailed information on the generation of soil and SSI model. Soil is a very complex material with a broad spectrum of properties, and randomness of soil material properties is much higher than those of other engineering materials such as concrete and steel. Furthermore, seismic loadings are transmitted to bridges through soil. Hence, instead of using commonly adopted simplified models for soil, extensive work has been done to evaluate soil models for SSI effects.

Chapter 4 describes detailed statistical models for typical bridges in NYC metropolitan area and establishes finite element models for the structural analysis of those bridges. Uncertainties in parameters, which dominate the behavior of bridges with specific consideration of SSI effects and deterioration have been identified and quantified for the purpose of developing bridge samples to be analyzed. Also, uncertainty and parametric analysis for the soil material has been presented. The seismic demand has been estimated by nonlinear time history analyses while capacity estimation has been carried out through specific analyses such as moment-curvature analysis and push-over analysis, or by experimental data collected. Seismic fragility curves have been constructed using analytical models of bridges.

Chapter 5 presents detailed modeling of material deterioration in bridge components and applications of these models to existing bridges. Deterioration of bridges and its components is not avoidable since they are continuously exposed to the impact of natural environments such as chlorides induced corrosion and are subjected to unavoidable
degradation of materials. Fragility analyses for existing highway bridges should not be based on original design parameters and properties. Fragility curves have been developed by considering effects of deterioration because of corrosion and material degradation in elastomeric bearings.

Chapter 6 presents conclusions of this dissertation and provides a discussion on future research work.
Chapter 2 Review of Current Bridge Seismic Fragility Analysis

Because of risk of significant damages during an earthquake, there is an increased concern on the evaluation of seismic hazards and the quantification of potential losses to infrastructures. In particular, there is significant uncertainty on estimation of potential losses to bridges vulnerable to damages during earthquakes uncertainties in material, structural and earthquake hazard, including ground shaking, fault rupture, soil liquefaction, and lateral/vertical ground movement [Imbsen (2001)]. When these hazards occur, bridges may experience from minor to severe damages, depending on the severity of the seismic hazard. Fragility curves are essential component of risk assessment methodology during such hazards.

2.1 Introduction

Seismic fragility curves are useful tools for seismic risk assessment. Basoz and Kiremidjian (1996) have presented a seismic event time-line, shown in Fig. 2.1. This timeline shows actions that take place before and after an earthquake event. It is observed from Figure 2.1 that risk assessment is the first action in the entire seismic time-line. The risk assessment step estimates the risk of potential losses that may occur as a result of a seismic event. Depending on the outcome of risk assessment, actions such as mitigation (using seismic retrofits) and pre-earthquake planning may be carried out. Following a seismic event, planning for actions such as emergency response, short term recovery and long-term recovery, also depends on the risk assessment. Hence, risk assessment plays an important role during a seismic event, and seismic fragility curves are essential tool for the assessment of risk.
2.2 Fragility Analysis of Bridges

Seismic fragility is a conditional probability that gives the likelihood that a structure or its components will meet or exceed a specified level of damage during a given ground motion intensity measure. There are a number of different methodologies that have been employed for the determination of structural fragilities. These methodologies can be classified into three main categories of fragility functions: (1) expert based fragility functions, (2) empirical fragility functions and (3) analytical fragility functions.

The expert based fragility functions were developed in 1980’s and can be considered as the initiation of the concept of fragility analysis. These fragility functions only depend on the experience and number of experts involved. With the availability of extensive amount of damage data collected during earthquakes around the world and progress in analytical probabilistic methods, this kind of fragility functions are no longer being used. Consequently, very few recent references could be found, except for the work done by Padgett and DesRoches (2006).
Empirical fragility curves are developed based on the actual damage data collected during the past earthquakes such as 1989 Loma Prieta, 1994 Northridge and 1995 Kobe earthquakes [e.g., Basöz et al. (1997, 1999), Yamazaki et al. (1999, 2000), Shinozuka et al. (2000a, 2003), Karim and Yamazaki (2001), Rossetto and Elnashai (2003), Elnashai et al. (2004)]. The research on the development of empirical fragility curves still has its own limitations, such as the lack of number and different levels of earthquakes due to frequency of occurrence of earthquake. Even though these limitations exist, empirical fragility curves still serve as benchmark for analytical fragility curves described below. These curves also present more realistic risk of damages during earthquakes.

Analytical fragility curves are being developed rapidly for different types of bridges during the past decade. These fragility curves are usually used to assess the vulnerability of bridges under different levels of earthquakes when actual bridge damage and ground motion data are not available. However, when used with experimental or actual damage data, analytical fragility curves can also reliably predict the probability of different levels of bridge damages, even when there is no history of past earthquake in a region. Basic methodology and detailed procedure for generating analytical bridges fragility curves have been developed by researchers, such as Kiureghian (1996), Mander and Basöz (1999), Shinozuka et al. (2000b), Mackie and Stojadinović (2001, 2007), Choi (2002), Karim and Yamazaki (2003), Gardoni et al. (2003), Choi et al. (2004), Nielson (2005), Nielson and DesRoches (2007a, 2007b), Pan (2007), and De Felice and Giannini (2010) and Pan et al. (2010). Many researchers have focused on necessary techniques used in analytical fragility analysis, such as parameter and uncertainty analysis [e.g., Saiïdi et al. (1996), Kwon and Elnashai (2006), Padgett and DesRoches (2007)], alternative seismic
intensity measure rather than PGA [Kafali and Grigoriu (2007)] and capacity/demand analysis [Saadeghvaziri and Yazdani-Motlagh (2008)]. Analytical fragility analysis method has also been applied to other structure, such as RC buildings, wood shear-walls and RC structural walls by Sasani and Kiureghian (2001), Schotanus et al. (2004), Rossetto and Elnashai (2005), Kim and Rosowsky (2005), Lupoi et al. (2006), and Kinali and Ellingwood (2007). These developed seismic fragility curves make it possible to predict the potential damage to bridge and other structural systems. Fragility of bridges retrofitted with several retrofit strategies/measures have been also been investigated by researchers. Some of these methods include restrainer cable, elastomeric isolation bearing, shear key, seat extender and steel jacket, etc. [Shinozuka et al. (2002), Kim and Shinozuka (2004), Padgett and DesRoches (2008, 2009), Pan et al. (2010a, 2010b), Agrawal et al. (2010)]. Casiati et al. (2008) have constructed fragility curves for a cable-stayed bridge retrofitted with hysteretic devices. They have shown that an accurate estimation of the limit state is very important, since the fragility results are very sensitive to uncertainties in limit state of bridge components. Banerjee and Shinozuka (2008) have developed analytical fragility curves by calibrating analytical models with past earthquake damage data. Zhang and Huo (2010) have evaluated the effectiveness and optimum design of isolation devices for highway bridges using fragility function method. This study shows that isolation devices can drastically reduce the damage probability of bridges, and offers an efficient way to select optimum isolation design parameters based on structural properties and performance objective incorporating the uncertainties in ground motions and variability of structural properties.
Figure 2.2 illustrates the general procedure for the development of analytical fragility curves.

**Figure 2.2 Flowchart for the Generation of Analytical Bridge Fragility Curves.**
Figure 2.3 shows an example of a set of fragility curves. In this figure, the vertical axis represents the probability that the demand of the structure will meet or exceed certain limit state under certain condition. The horizontal axis in fragility curves generally varies within different pre-defined conditions. Figure 2.3 shows the case where fragility is function of the absolute value of response of structure and/or its components. Figure 2.4 shows more fragility curve with intensity of earthquake event as the horizontal axis, which is used more commonly.

Figure 2.3 Example Fragility Curves in HAZUS Damage Levels (FEMA 2003)
2.3 Consideration of Deterioration and Soil-Structure Interaction (SSI)

More recent studies on seismic fragility curves, focusing on the deterioration and aging behavior of bridges, have been presented based on recent research on deterioration mechanisms. There are various causes for deterioration of engineering materials used in bridges. Modeling complex deterioration mechanism due to chemical attacks, such as carbonation, chloride, corrosion, sulfate, as well as the reduction in capacity of bridge components, such as bearings and columns, have been addressed by several researchers, such as Tsopelas et al. (1996), Mori et al. (1996), Val et al. (1998), Chase and Gáspár (2000), Stewart and Val (2003), Du et al. (2005a, 2005b), Itoh et al. (2006a, 2006b), Parameswaran et al. (2008), Bertolini (2008), and Tapan and Aboutaha (2008). The findings in these studies have been introduced into the analytical seismic fragility analysis framework and several time-dependent fragility curves for bridge system and components levels have been developed.
Choe et al. (2008, 2009, 2010) have investigated the potential reduction in capacity and increase in fragility of a typical single-bent bridge in CA because of deterioration. Their work illustrates the potential importance of capturing the aging effects on seismic fragility, and identifying crucial materials and corrosion parameters that significantly affect bridge reliability. In these studies, time-dependent fragility curves, which account for uncertainties in the corrosion model as well as in the bridge components capacity models, have been developed. Based on this work, several researchers, such as Ghosh and Padgett (2010), Simon et al. (2010) and Alipour et al. (2010) have studied the effect of aging on system response and fragility by considering not only the vulnerability of multiple components, but also their simultaneous aging and constructed time-dependent fragility curves with the consideration of corrosion induced deterioration. Mullard and Stewart (2010) have extended the work to life-cycle assessment of maintenance strategies on the basis of time-dependent reliability model.

Most of these research studies focus on one factor of the deterioration, either reinforcement deterioration in columns or aging of rubber material used in the bridge bearing. Few studies have considered the combined effects of these deteriorations mechanisms (corrosion and material degradation).

Although consideration of soil-structure interaction (SSI) in models of bridges for the fragility analysis is very important, all such studies have considered simplified model of soil in the analytical models. Ghiocel et al. (1998) have investigated the seismic response and fragility evaluation for nuclear power plant (NPP) sitting on soft soil deposit with consideration the SSI effects. In the bridge engineering field, lumped spring model is the most commonly adopted approach to model a soil-foundation, as presented in Nielson
However, soil is a very complex material with a broad spectrum of properties, including friction, cohesion, cyclic-mobility/flow liquefaction, dilation/contraction and buildup/dissipation of pore water pressure. Besides, the randomness in soil material properties is much higher than those in other common engineering materials such as concrete and steel. Therefore, lumped springs may neither represent the complexity of soil behavior, nor efficiently model the uncertainties associated with soil properties.

Several constitutive models have been developed to simulate cyclic mobility and/or flow-liquefaction soil response, especially the shear deformation by Elgamal et al. (2002, 2003) and Yang et al. (2002, 2003), as well as the computational models for soil-pile and/or soil-abutment system by Ellis and Springman (2000), Mylonakis and Gazetas (2000), and Shamsabadi et al. (2007). Seismic response of bridges and bridge components have also been investigated. Saadeghvaziri et al. (2000) have studied the SSI effects on longitudinal response of multi-span simply supported bridges using equivalent translational and rotational springs. Elgamal et al. (2008) have investigated 3D seismic response of bridge-foundation system. Their studies indicate that the soil boundary conditions remain an area of ongoing research. Similarly, Jeremić et al. (2009) have conducted time domain simulation of soil-foundation-structure interaction in non-uniform soils. In their studies, soil element size determination, coupling of structural and soil models, and domain reduction method (DRM), which represent the only method that can consistently apply free field ground motion to finite element model, have been discussed. Kwon and Elnashai (2010) have investigated and compared four different modeling methods of abutments and foundations system of bridges, namely, (1) fixed foundation
assumption, (2) lumped springs derived from conventional methods, (3) lumped springs developed from 3D FE analysis and (4) Multiplatform 3D FE models with more realistic soil models. They have constructed fragility curves with consideration of SSI using these models. Due to lack of sufficient reliable data, it is difficult to draw conclusions on the most accurate method. Simply from the results presented in this work, one can see that the bridge is more fragile when analyzed using multiplatform than using fixed foundation. As indicated in this work, the fragility curves obtained from a multiplatform approach can be considered reliable as the method was verified from the measured response of an instrumented bridge. However, it is still to be noted that this research is for bridge located near the New Madrid Fault. Neither the bridge configuration nor the characteristics of the earthquake are typical of mid-US areas. Mwafy et al. (2010) have extended this work and have conducted seismic assessment of an existing non-seismically designed bridge-abutment-foundation system. Similar work has been presented by Aygün et al. (2011) which mainly focuses on multi-span continuous steel bridge on liquefiable soils. In this work, soil model as well as SSI model were developed. However, possible conflicts between soil model and near-field element in the SSI model haven’t been discussed in details.

### 2.4 Limitation of Current Approaches

For the material deterioration models, researchers have mainly focused on the deterioration of the bridge column, since a deteriorating bridge pier not only affects the capacity of the bridge/bridge components, but also the response quantities under seismic loads. It has been observed that the deterioration of bearings isn’t considered in the development of time-dependent fragility analysis, since the capacity of bearings isn’t
significantly affected because of deterioration. However, the deterioration of the elastomeric bearing can cause changes in the period of the bridge, which can affect seismic behavior of the bridge. Hence, seismic fragility curves considering deterioration in concrete and elastomeric materials are important to understand the effect of deterioration on seismic vulnerability of bridges.

Although SSI effects are included in analytical bridge models by using simplified methods such as p-y springs, there is no documented literature on fragility analysis by considering 3-dimensional SSI model of soil. Since the behavior of soil during a strong earthquake may be highly nonlinear because of plastic deformation and damage, simplified models may not be able to adequately capture this nonlinear behavior. Three-dimensional models can also be helpful in understanding and quantifying the effects of soil on the overall risk to the bridge during an earthquake. The main objective of research in this dissertation is to bridge this critical gap in the seismic fragility analysis of bridges.
Chapter 3 Soil-Structure Interaction Modeling for Bridge Seismic Response Analysis

3.1 Introduction

Earthquake waves, usually generated at the bedrock level, propagate from the bedrock, through soils layers, to the ground surface. Foundation system of a bridge, including soils and bridge components (piles for example) form important connection between soil and structure. Considering the importance of the foundation system on overall response of the bridge, soil-structure interaction (SSI) affects the seismic response of a bridge and its components significantly.

To evaluate SSI effects on seismic response of a bridge, several modeling methods have been investigated by different research groups. From structural engineering perspective, there are four types of analytical models that can represent SSI effects, namely, (1) fixed foundations by simply ignoring SSI effects (2) lumped springs developed from conventional pile analysis of piles at foundations, (3) lumped springs developed from three-dimensional finite element analysis of foundations and (4) detailed finite element model of foundation including soil conditions and soil-pile interactions. Geotechnical and structural engineers are usually involved in the estimation of bridge response using these analytical models except the first one, depending on the complexity of the bridge system. Models utilizing one of two lumped spring approaches mentioned above are generally developed by geotechnical and structural engineers through detailed geotechnical studies. For example, a geotechnical engineer, based on hazard levels of earthquake ground motions as well as the soil conditions, can develop a simplified spring model that a
structural engineer can use to model foundation effects. Although this could be an effective procedure to model foundation systems conservatively, lumped springs don’t represent the complexity of SSI. Soil is a very complex material with a broad spectrum of properties. For seismic fragility analysis, the randomness of soil material properties is much higher than those of other structural materials. Hence, lumped springs models aren’t sufficient to represent neither the complexity of soil behavior, nor uncertainties associated with soil properties.

Hence, the proposed work on fragility of the bridge with SSI effects has been carried out by including a detailed 3-D soil model around the bridge foundation. This model can simulate propagation of seismic waves from the rock motion to the foundation system and the uncertainties in soil properties.

3.2 Synthetic Earthquake Generation

The likelihood of an earthquake in NYC metropolitan area has been estimated as ‘moderate’ by the US Geological Survey (USGS), although there is no history of recorded ground motions during an earthquake capable of causing noticeable damages. Hence, synthetic ground motions time histories have to be generated and used as input in the seismic response analysis of bridges. Generally, the development of synthetic motions at ground surface is done in two steps: (i) Generation of ground motions at the outcrop of rock site based on the characteristics of seismic source (ii) Conversion of the rock motions into acceleration time histories at the ground surface level by site response analysis based on attenuation and local soil conditions.
In this study, the latter step has been merged into the finite element model of soil and SSI, so this subsection only focuses on the first step, which is performed using the computer program SIMQKE (Gasparini and Vanmarcke, 1976).

3.2.1 Generation of Design Spectra for Different Peak Ground Acceleration (PGA) Level at Rock Site

Seismic Design Guidelines for Bridges in Downstate Region (2014) by the New York City Department of Transportation recommends Seismic Hazard for downstate region of New York State that includes five boroughs of New York City and counties of Rockland, Westchester and Richmond. This seismic hazard is in the form of 5% damped horizontal Uniform Hazard Spectra (UHS) for earthquake return periods of 500, 1500 and 2500 years based on the detailed study of rock motion by Risk Engineering, Inc. (2002) These horizontal UHS for 500, 1500 and 2500 years return periods are presented in Tables 3.1 to 3.3 and Figure 3.1. The spectra in Tables 3.1 to 3.3 and Figure 3.1 represent an 85th percentile of ground motions corresponding to each one of the three return periods (median plus one standard deviation level). The motions denoted as VHR are for Very Hard Rock (VHR) in NYC, typical of the eastern United States (US), with a shear wave velocity of at least 2.83 km/sec (approximately 9,000 ft/sec). This 2.83 km/sec shear wave velocity is an average of eastern US continental crust. UHS in the horizontal direction for other softer rock conditions more frequently encountered in NYC are presented as Rock Class A and Rock Class B in Tables 3.1 to 3.3 and Figure 3.1. For detailed fragility analysis, design spectra need to be generated at different PGA levels through the interpolation of spectra corresponding to 500, 1500 and 2500 Yr return
periods. In this research, we have assumed that the soil is on rock class B. Hence, rock spectra corresponding to Rock Class B in Figure 3.1 have been used.

Table 3.1 500-year Return Period-NYC Rock Sites-Horizontal Design Spectra (g)

<table>
<thead>
<tr>
<th>Period(Sec)</th>
<th>VHR</th>
<th>Rock Class A</th>
<th>Rock Class B</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.01</td>
<td>5.34E-02</td>
<td>6.14E-02</td>
<td>8.81E-02</td>
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<tr>
<td>0.04</td>
<td>1.25E-01</td>
<td>1.44E-01</td>
<td>2.06E-01</td>
</tr>
<tr>
<td>0.1</td>
<td>9.98E-02</td>
<td>1.15E-01</td>
<td>1.65E-01</td>
</tr>
<tr>
<td>0.2</td>
<td>6.99E-02</td>
<td>8.04E-02</td>
<td>1.15E-01</td>
</tr>
<tr>
<td>0.5</td>
<td>2.56E-02</td>
<td>2.94E-02</td>
<td>4.22E-02</td>
</tr>
<tr>
<td>1</td>
<td>1.26E-02</td>
<td>1.45E-02</td>
<td>2.08E-02</td>
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<tr>
<td>2</td>
<td>6.66E-03</td>
<td>7.66E-03</td>
<td>1.10E-02</td>
</tr>
<tr>
<td>4</td>
<td>2.23E-03</td>
<td>2.56E-03</td>
<td>3.68E-03</td>
</tr>
<tr>
<td>5</td>
<td>1.61E-03</td>
<td>1.85E-03</td>
<td>2.66E-03</td>
</tr>
<tr>
<td>8</td>
<td>7.11E-04</td>
<td>8.18E-04</td>
<td>1.17E-03</td>
</tr>
<tr>
<td>10</td>
<td>5.37E-04</td>
<td>6.18E-04</td>
<td>8.86E-04</td>
</tr>
</tbody>
</table>
Table 3.2 1500-year Return Period-NYC Rock Sites-Horizontal Design Spectra (g)

<table>
<thead>
<tr>
<th>Period (Sec)</th>
<th>VHR</th>
<th>Rock Class A</th>
<th>Rock Class B</th>
</tr>
</thead>
<tbody>
<tr>
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<td>1.40E-01</td>
<td>1.61E-01</td>
<td>2.31E-01</td>
</tr>
<tr>
<td>0.04</td>
<td>3.36E-01</td>
<td>3.86E-01</td>
<td>5.54E-01</td>
</tr>
<tr>
<td>0.1</td>
<td>2.60E-01</td>
<td>2.99E-01</td>
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</tr>
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<td>1.82E-01</td>
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</tr>
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<td>0.5</td>
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<td>7.14E-02</td>
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<td>1</td>
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<td>9.74E-03</td>
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<td>5</td>
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<td>7.05E-03</td>
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<td>8</td>
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<td>3.32E-03</td>
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<td>10</td>
<td>1.54E-03</td>
<td>1.77E-03</td>
<td>2.54E-03</td>
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</tbody>
</table>
Table 3.3 2500-year Return Period-NYC Rock Sites-Horizontal Design Spectra (g)

<table>
<thead>
<tr>
<th>Period (Sec)</th>
<th>VHR</th>
<th>Rock Class A</th>
<th>Rock Class B</th>
</tr>
</thead>
<tbody>
<tr>
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<td>2.02E-01</td>
<td>2.32E-01</td>
<td>3.33E-01</td>
</tr>
<tr>
<td>0.04</td>
<td>5.27E-01</td>
<td>6.06E-01</td>
<td>8.70E-01</td>
</tr>
<tr>
<td>0.1</td>
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<td>4.19E-01</td>
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</tr>
<tr>
<td>0.2</td>
<td>2.31E-01</td>
<td>2.66E-01</td>
<td>3.81E-01</td>
</tr>
<tr>
<td>0.5</td>
<td>9.15E-02</td>
<td>1.05E-01</td>
<td>1.51E-01</td>
</tr>
<tr>
<td>1</td>
<td>4.35E-02</td>
<td>5.00E-02</td>
<td>7.18E-02</td>
</tr>
<tr>
<td>2</td>
<td>2.24E-02</td>
<td>2.58E-02</td>
<td>3.70E-02</td>
</tr>
<tr>
<td>4</td>
<td>9.05E-03</td>
<td>1.04E-02</td>
<td>1.49E-02</td>
</tr>
<tr>
<td>5</td>
<td>6.92E-03</td>
<td>7.96E-03</td>
<td>1.14E-02</td>
</tr>
<tr>
<td>8</td>
<td>3.23E-03</td>
<td>3.71E-03</td>
<td>5.33E-03</td>
</tr>
<tr>
<td>10</td>
<td>2.49E-03</td>
<td>2.86E-03</td>
<td>4.11E-03</td>
</tr>
</tbody>
</table>

It is observed from Figure 3.1 that the maximum value of design PGA in New York City area is approximately 0.3g. For fragility analysis, we need design PGA from 0.1g to 1.0g. This is done by extrapolation of spectra in Figure 3.1. These extrapolated spectra represent possible extreme events in the New York City area. Simple linear inter/extrapolation in terms of return period has been carried out to generate additional spectra representing different PGAs. In this inter/extrapolation, two pairs of period, acceleration spectra data from different return period are used to linearly determine one pair of period, acceleration spectra data. Figure 3.2 shows the examples for developing
different levels of PGA, and 10 levels of design spectra with different PGA, ranging from 0.1g to 1.0g.

Figure 3.1 NYC 3 Different Levels of Design Spectra
3.2.2 Generation of Acceleration Time Histories Based on Design Spectra

Once spectra for different PGA levels have been developed, the synthetic rock motion for each spectrum can be generated by using computer program SIMQKE. The basic idea of generation of ground motions is based on the fact that any periodic function can be expressed by a series of sinusoidal waves,

\[ X(t) = \sum_n A_n \sin(\omega_n t + \phi_n) \]  

(3.1)

where \( A_n \) is the amplitude and \( \phi_n \) is the phase angle of \( n^{th} \) contributing sinusoid. If amplitude is fixed and phase angle varies, different motions with same general
appearance but different phase can be developed. The computer program can generate a random number to produce strings of phase angles with uniform probability in the range of $0$ to $2\pi$.

In Eq.(3.1), amplitudes $A_n$ are related to the spectral density function $G(\omega)$ through the Eq.(3.2) below,

$$G(\omega_n)\Delta\omega = \frac{A_n^2}{2}$$

(3.2)

and the total power can be reached by sum of all $n^{th}$ sinusoidal parts with frequency of $\omega_n$. Once the number of sinusoids in the motion reach a certain large value, the total power will become the area under the continuous curve $G(\omega)$, as expressed by Eq. (3.3),

$$\sum_{n} \frac{A_n^2}{2} = \sum_{n} G(\omega_n)\Delta\omega \xrightarrow{n \to \infty} \int_{0}^{\infty} G(\omega) d\omega$$

(3.3)

Since the power of motion generated according to Equation (3.1) does not vary with time, in order to simulate the transient character of real earthquake, the steady-state motions are added by multiplying a deterministic envelope function $I(t)$. Then, the synthetic motion $Z(t)$ then becomes,

$$Z(t) = I(t)\sum_{n} A_n \sin(\omega_n t + \phi_n)$$

(3.4)

The resulting motion is stationary in frequency content with a peak acceleration close to the target peak acceleration. There are several different intensity envelop functions such as “Trapezoidal”, “Exponential” and “Compound”. In this study, we utilized
“Trapezoidal” intensity envelop function for computational simplification. Following this procedure, 3 different motions have been generated using each spectrum. Figure 3.3 shows the highest PGA level spectrum generated in this research and Figure 3.4 shows the corresponding ground motions.

Figure 3.3 Generated Design Spectrum with Highest PGA (1.0g)
Figure 3.4 Three Different Generated Motions for One Spectrum

3.3 Site Response Analysis of Layered Soil Columns

As discussed previously, ground motions are affected significantly by local soil conditions [e.g., Silva et al. (1988)]. Earthquake motions at the bedrock level are modified significantly, both in frequency and amplitude, as seismic waves propagate through the soil deposits. Hence, synthetic ground motions in Figure 3.4 need to be modified for local soil effects before they can be used as input ground motion for investigating seismic behavior of bridges. This can be done through nonlinear finite element program such as OpenSees (Mazzoni et al., 2005) in which bedrock motions can
be applied as input to the soil model between the footing and the bedrock. The nonlinear finite element analysis (FEA) approximates the effects of foundation components such as piles/pile groups, by equivalent springs whereas the modeling using nonlinear finite element program is capable of considering detailed model of pile/pile group and their interaction with surrounding soil. At the same time, the FEA simulates the behavior of soil material itself and it is equivalent to the site specific analysis which can be carried out by computer program such as ProShake (EduPro, ProShake Version 1.11, 2001) and DeepSoil (Hashash et al., 2009). The approach used in this research is based on detailed modeling of foundation using the OpenSees software.

3.3.1 Computational Model Description

Figure 3.5 shows soil deposits underlain by an elastic half-space, which simulates the finite rigidity of an underlying medium, such as bedrock. The soil is assumed to be saturated clay and its seismic responses is assumed to be undrained; therefore, total stress analysis method can be used.

In one direction, either longitudinal or transvers, the soil is modeled in two-dimensions with two degrees-of-freedom using the plane strain formulation of the quad element. To account for the finite rigidity of the underlying half-space, a Lysmer-Kuhlemeyer (1969) dashpot is incorporated at the base of the soil column and is assigned a dashpot coefficient equal to the product of the mass density ($\rho$) and shear wave velocity ($v_s$) of the underlying layer with the area of the base of the soil column. This dashpot coefficient is expressed as,

$$ c = \rho v_s $$

(3.5)
The soil column is excited at the base by a horizontal force time history, which is proportional to the known velocity time history of the previously generated bedrock ground motion. The horizontal force time history is obtained by multiplying the known velocity time history by a constant factor set as the product of the area of the base of the soil column (width x thickness) with the mass density and shear wave velocity of the underlying layer. This constant factor is expressed as,

\[ f = \rho v_y' \]  

(3.6)

The area of the soil column is included to ensure proportional loading for any desired horizontal element size.
The nodes at the base of soil column are fixed against displacement in the vertical direction in accordance with the assumption that the soil layers are underlain by bedrock. In order to achieve a simple shear deformation pattern, the remaining soil nodes are tied together by setting equal degree-of-freedom (DOF) for every pair of nodes in vertical direction. One of the dashpot nodes is fully fixed while the other one is fixed only against displacements in vertical direction, and this partially fixed dashpot node is linked with the horizontal DOF of the node at the base of the soil column.

To simulate the constitutive behavior of the soil and underlying rock, a series of material properties are required, such as mass density, the shear wave velocity and Poisson’s ratio. Besides, the elastic, shear and bulk modulus need to be computed according to those properties. Poisson’s ratio needs to be set as zero for the purpose of satisfying one-dimensional analysis, which means no vertical accelerations can be generated.

Furthermore, particular constitutive model should be defined for soil material. In OpenSees material library (Yang et al., 2008), there are several material models which can simulate different soil materials. For example, \textit{PressureIndependMultiYield} material is an elasto-plastic material in which plasticity exhibits only in the deviatoric stress-strain response. The volumetric stress-strain response is linear-elastic and is independent of the deviatoric response. This material has been developed to simulate monotonic or cyclic response of material whose shear behavior is insensitive to the confinement change. Organic soils or clay under fast (undrained) loading conditions can be modeled with this material (Yang et al., 2008). Figure 3.6 shows the schematic of stress-strain relationship using this material model.
Figure 3.6 Schematic of Soil Stress-Strain Relationship

One can define yield surface directly based on desired shear modulus reduction curve by defining pairs of shear strain ($\gamma$) and modulus ratio ($G_s$) values. Otherwise, by default, the shear stress $\tau$ (octahedral) – shear strain $\gamma$ (octahedral) nonlinearity can be defined by a hyperbolic curve (backbone curve) below,

$$
\tau = \frac{G\gamma}{1 + \frac{\gamma}{\gamma_r} \left( \frac{p'_r}{\gamma_r} \right)^d}
$$

(3.7)

where $\gamma_r$ satisfies the following equation at $p'_r$:

$$
\tau_f = \frac{2\sqrt{2}}{3 - \sin \phi} p'_r + \frac{2\sqrt{2}}{3} c = \frac{G_r \gamma_{max}}{1 + \gamma_{max}/\gamma_r}
$$

(3.8)

where

$G_r$ - Reference low-strain shear modulus, specified at a reference mean effective confining pressure of $p'_r$,

$p'_r$ - Reference mean effective confining pressure at which $G_r$ and $\gamma_{max}$ are defined,

$c$ - Apparent cohesion at zero effective confinement,
$d$ - Pressure dependency coefficient, it is an optional non-negative constant defining variations of reference low-strain shear modulus and reference bulk modulus as a function of initial effective confinement $p_i'$,

$\gamma_{\text{max}}$ - An octahedral shear strain at which the maximum shear strength is reached, specified at a reference mean effective confining pressure of $p_i'$,

$\phi$ - Friction angle at peak shear strength in degrees.

### 3.3.2 Verification Cases

Several analyses have been carried out for the verification of the model described above. Two ground motions are used in these analyses: one downloaded from the Peer NGA strong motion database (http://peer.berkeley.edu/nga/), and the other from the synthetic bedrock motions generated, as described previously.

For simplicity, a soil profile with single layer of 40 m thickness, 2 Mg/m$^3$ mass density and 300 m/s (985 ft/s) shear wave velocity is considered. The mass density of underlain rock is 2.4 Mg/m$^3$ while the shear wave velocity is 760 m/s (2494 ft/s). Other soil properties are taken as follow: Poisson’s ratio = 0.49, soil cohesion = 95 KPa, peak shear strain = 0.05, soil friction angle = 0, reference pressure = 80 KPa, and pressure dependency coefficient = 0.

Figure 3.7 and 3.8 show the response acceleration time histories based on different bedrock motions. It can be seen from these figures that OpenSees soil model is capable of simulating the wave attenuation well in this simple case.
Figure 3.9 shows comparisons between time history plots generated using OpenSees and DeepSoil software. It is observed from Figure 3.9 that the response acceleration time history calculated using OpenSees matches well that obtained by using DeepSoil. Likewise, spectra of time histories using the two software shown in Figure 3.10 match with each other very well. Hence, OpenSees soil model can be used directly to develop finite element model of soil for simulating the constitutive behavior of soil very well. Reduction and damping curves used in DeepSoil are based on Vuceti & Dobry, 1991 model, as shown in figure 3.11. Reduction curve adopted in OpeeSees is generated based on equation (3.7). Note that the backbone curve recorded in OpenSees PressureIndependMultiYield material is the secant shear modulus reduction curves at one or more given confinements. User can define their own confinements, however, by default, in this work, 20 yield surfaces are defined in this material, and correspondingly 20 confinements are presented. For each given confinement, there are two columns of data: shear strain ($\gamma$) and secant modulus ($G_s$) are recorded. Figure 3.12 and 3.13 show the comparison of modulus reduction (MR) curve and damping curve adopted in OpenSees and DeepSoil, respectively. As shown in the figures, the difference of the MR and damping curve is the source of the difference of the spectra shown in figure 3.10.
Figure 3.7 Response Acceleration Time History at the Ground Surface of the Soil Column (Motion from Peer NGA Database)
Figure 3.8 Response Acceleration Time History at the Ground Surface of the Soil Column (Motion generated in this work)
Figure 3.9 Comparison between OpenSees and DeepSoil (Time History)

Figure 3.10 Comparison between OpenSees and DeepSoil (Spectra)
Figure 3.11 Soil Modulus Reduction and Damping Curves Used in DeepSoil
Figure 3.12 Soil Modulus Reduction Curve Comparison

Figure 3.13 Damping Curve Comparison
3.4 Seismic Soil-Structure Interaction

Typical bridges consist of the superstructure (deck slab, girders, etc.), substructure (abutments, bents, footings and foundations) and bearings. Though supporting soil is usually not commonly considered a part of bridge foundation, the effects of soil play an important role in the seismic responses of bridges. During earthquakes, the individual components of bridge, as well as the supporting soil, interact with each other and affect the global response of the bridges. Relevant aspects related to SSI, mainly soil-pile interaction, will be discussed in this subsection, while soil-abutment interaction will be briefly mentioned.

3.4.1 Modeling of Soil-Pile Interaction

Methods of analyzing seismic SSI include modeling of the pile and soil continuum using Finite Element Analysis (FEA) methods, dynamic Beam-on-Nonlinear-Winkler-Foundation (BNWF) methods and simplified two-step methods that uncouple the structure and foundation portions of the analysis.
This subsection discusses the FEA method adopted in detail, while figure 3.14 shows the conceptual methods of BNWF and simplified two-step analysis.

The two-step uncoupled methods ignore or simplify certain details of effects from SSI and may not simulate the SSI effects quite appropriately. The dynamic BNWF (also called “dynamic p-y”) methods are commonly used in practice and are considerably less complex than comprehensive 3D FE models. These methods also offer several advantages over the two-step uncoupled method when dealing with soft soil conditions. However, these methods make too many simplifications to shear wave propagation within soil materials. In BNWF method, far-field (free-field) behavior is significantly simplified and only behavior of near-field, where SSI occurs, is investigated to generate p-y curves.
The comprehensive three-dimensional (3D) finite element model (FEM) of SSI system is computationally expensive. It requires not only significant simulation time, including pre/post-processing work, but also detailed hysteretic or other advanced constitutive models of soil materials capturing localized soil response and the use of complicated contact elements to represent soil and pile interaction effects, such as gapping and sliding. Besides this, spatial definition of the input seismic excitation and soil boundary conditions are topics of ongoing research.

Comprehensive 3D FEM methods for SSI (e.g., Petek (2006)) are quite complex and are beyond the scope of this research. In this research, we propose a simplified 3D FEM method where complexity of the comprehensive model is reduced by introducing link spring element and t-z (q-z) spring in BNWF (p-y) method.

When an earthquake occurs, shear waves propagate through the soil and apply kinematic forces on the piles/pile groups. The shaking of the underground system including bedrock and soils induces inertial forces, which will be transferred to the foundation as well as the structure above the ground surface. As a result, the soil behavior as well as the foundation (pile/pile groups) need to be properly considered when carrying out the structural analysis.

In Elgamal et al. (2008), a simplified 3D finite element model including soil and SSI model was constructed. For the soil model itself, this work presents a comprehensive soil model including constitutive and geometric model. However, the interface between soil and structure was simplified as fixed boundary condition to reduce computational time. The proposed SSI modeling method in this dissertation simplifies the geometric soil model by using two directions of 2D soil to represent 3D, keeps the constitutive soil
model and mainly focuses on the simulation of SSI model. For the SSI model, a link element has been introduced to account for the lateral motion/force transfer between soils and structures.

Figure 3.15 shows a schematic of the proposed SSI analysis model for single pile supported structure. As shown in the figure, link spring element simulates lateral force transmission behavior at the soil-structure interface in two-orthogonal-directions, while t-z and q-z springs simulate axial and tip behavior, respectively.
3.4.2 Link Spring Curve, Dynamic t-z and q-z Curves

As shown in Figure 3.15, horizontal force transmission between the pile and soil is modeled by link spring element. Figure 3.16 shows the simple constitutive curve for link springs. As shown in this model, if soil contacts pile, the link spring will apply compressive force from the pile through the link spring. On the other hand, when soil and pile are separated, zero tensile force will be applied on the pile. As shown in figure 3.16, negative stiffness of the link element is set to be a very small number, which should be 0 theoretically, for numerical convergence considerations, while positive stiffness, K, is set to be the same as the stiffness of the pile, as described below. Johnson (2001) stated in the previous researches that using contact elements in a finite element analysis (FEA) simulation is seldom a simple painless experience, and the changing contact between parts is a common phenomenon which, in some cases, can be treated with rigorous mathematical theory. Even with simplifying the link/contact element herein to a point-to-point contact element, which can be used where little or no sliding occurs, the determination of the stiffness of the contact interface would be a judgement call of the analyst. Generally, "all contact problems require a stiffness between the two contact surfaces. The amount of penetration between the two surfaces depends on this stiffness. Higher stiffness values decrease the amount of penetration but can lead to ill-condition of the global stiffness matrix and to convergence difficulties. Ideally, you want a high enough stiffness that contact penetration is acceptably small, but a low enough stiffness that the problem will be well-behaved in terms of convergence or matrix ill-condition", as stated in Johnson (2001).
In the FEA practice with ANSYS (2000) and LS-DYNA (2015), choosing of the contact stiffness is recommended to be within the range from the stiffness of the stiffer/stiffest part to the average stiffness of the contact parts. In this research work, the contact stiffness is set to be the stiffness of the pile because that, by trial and error method, (1) the stiffness of the pile controls the contact and force transferring behavior between pile and soil, (2) the pile stiffness is high enough to prevent the penetration, and (3) it is low enough, comparing to the stiffness of the whole structure, to allow the analysis going well in terms of convergence.

For vertical force transfer, the p-y method is adopted. The p-y curve (for axial one, commonly named as t-z) is the important constitutive curve for defining the BNWF model. In general, the relationships between vertical soil reaction force (t) and pile displacement (z) are based on field tests, laboratory tests and/or analytical derivations.

In OpenSees material library, there are several uniaxial materials, named as PyTzQz uniaxial materials, for modeling the constitutive behavior of BNWF model. Specially, for
non-liquefiable soils, “PySimple1”, “TzSimple1” and “QzSimple1” materials have been
developed (Boulanger, R. W. et al. 2003) to represent behavior of lateral (p-y), axial (t-z)
and tip resistance (q-z) soil springs, respectively.

In this research work, the lateral behavior is modeled by link spring model discussed
above, while vertical behavior, modeled by t-z and q-z model as summarized below, is
simulated by default OpenSees material library.

TzSimple1 has four input parameters, which are $t_{ult}$ (the ultimate capacity of the t-z
material), $z_{s0}$ (the displacement at which 50% of $t_{ult}$ is mobilized during monotonic
loading), $c$ (the viscous damping term on the far-field component of the material) and
soilType (an argument that identifies the choice of backbone t-z relation that is
approximated). QzSimple1 has five input parameters, which are $q_{ult}$ (the ultimate
capacity of the q-z material), $z_{s0}$ (the displacement at which 50% of $q_{ult}$ is mobilized
during monotonic loading), $c$ (same as defined in TzSimple1) and soilType (an
argument that identifies the choice of backbone q-z relation that is approximated). One
can construct constitutive behaviors of these soil springs by parameters mentioned above,
in particular, values of $t_{ult}$, $z_{s0}$ and $q_{ult}$ can be obtained from empirical equations. More
details about the empirical equations for determining those parameters and the equations,
which control the normalized hysteretic behaviors ($t/t_{ult}$ versus $z/z_{s0}$ and $q/q_{ult}$ versus
$z/z_{s0}$) of these springs, can be found in Boulanger et al. (2003).

Furthermore, when considering lateral force transmission, soil should not apply tensile
force on the pile, even they are not separated completely, since soil material itself can
hardly take any tensile force. To verify the validity of this feature of the soil model mentioned in subsection 3.3, a simple static analysis has been carried out by applying tensile force on a soil block. Figure 3.17 shows the model, dimensions of the soil and applied forces. It is observed from the analysis results in Figure 3.18 that the displacements of node 4 and node 6 are very large compared to dimensions of the soil block. This implies that the soil block undergoes large deformation under the tensile force because of its inability to resist this force.

![Figure 3.17 Model for Soil Material Verification Analysis](image-url)
3.4.3 Verification Case for Soil-Pile Interaction

As mentioned in previous sections, several modeling methods have been presented in the literature for evaluating SSI behavior. Results of these models have been verified through experiments. For example, Shaomin et al. (1998) have carried out centrifuge test to verify the p-y model. Figure 3.19 shows the configuration of the centrifuge model test. Tests were performed on samples of normally consolidated San Francisco Bay Mud (density, $\rho \approx 1.7\, \text{Mg/m}^3$) with a "crust" of dense sand ($\rho \approx 2.1\, \text{Mg/m}^3$) on the surface of
The superstructure was represented by a 11.5 gram mass (includes a 1.5 gram accelerometer) attached to an extension of the pile. Table 3.4 summarizes the centrifuge scaling laws used to convert model dimension to prototype dimension. For model test presented in Shaomin et al. (1998), a scale factor of $N = 50$ was used.
A scaled version of the ground motion recorded at Santa Cruz during the 1989 Loma Prieta Earthquake was used to excite the base of the centrifuge model.

A verification of the SSI model in this research has been carried out by modeling this experiment. The superstructure is the same as used in the test, 11.5 gram lumped mass. The saturated soil mass density is 1.8 Mg/m³, which is close to the San Francisco Bay Mud used in the test. Correspondingly, other soil properties are the recommended value by OpenSees material library for stiff clay. The reference low-strain shear modulus is 150000 kPa, the reference bulk modulus is 750000 kPa, the apparent cohesion at zero effective confinement is 75 kPa and the octahedral shear strain at which the maximum shear strength is reached is 0.1. For the bottom of the soil layer, the fixed boundary condition is adopted, since in the centrifuge test, there's no movement allowed on the horizontal and vertical directions. Sixteen ground motions, in PEER Strong Ground
Motion Database, that were recorded near Santa Cruz, have been investigated in this research and are shown in Figure 3.20. In this figure, Motion13, whose frequency contents are closest to the one used in Shaomin et al. (1998), has been chosen and scaled as the input motion to the analysis presented in the following.
Figure 3.20 Comparison of the Excitation Motion to the Centrifuge
The spectral accelerations at soil surface and superstructure from Shaomin et al. (1998) and those based on simulation in this research are presented in Figures 3.21 and 3.22. It is observed from these figures that the responses using SSI model proposed in this research are similar to those obtained from centrifuge test. This demonstrates that the proposed SSI model in this research can reliably simulate SSI of bridge foundations.

Figure 3.21 Response at Soil Surface
3.4.4 Soil-Abutment Interaction

Bridge abutments significantly affect the seismic response of the bridge deck by providing longitudinal and transverse resistance. Typical bridge abutment types include seat abutment, diaphragm abutment and cast-in-drilled-hole (CIDH) shaft-controlled abutment. Figure 3.23 shows the schematic of seat-type abutment. In fact, the comprehensive mechanism of soil-abutment interaction behavior is quite complicated and difficult to simulate accurately. Therefore, the soil-abutment interaction is modeled as beam elements supported on springs with spring stiffness coefficient derived on the basis of empirical relationships. Current CALTRANS (California Department of Transportation) practice recommends modelling initial abutment stiffness as (CALTRANS, 2009),

\[
K_{abut} = k_W \left( \frac{h_{abut}}{5.5} \right)
\] (3.9)
where $k_i$ is the suggested value for initial embankment fill stiffness, which is equal to 20.0 kip/in/ft, $w$ is the width of the backwall (ft) and $h_{abut}$ is the height of the backwall (ft). The ultimate abutment capacity due to the backfill soil can be expressed as,

$$P_{abut} = A_e \times 5.0\text{ksf} \times \left(\frac{h_{abut}}{5.5}\right) \quad \text{(ft, kip)} \quad (3.10)$$

where $A_e = h_{abut} \times w_{abut}$ is the effective abutment area. The passive pressure resisting the movement at the abutment increases linearly with the displacement, as shown in Figure 3.24. The maximum passive pressure of 5.0 ksf (239 kPa), used in Equation 3.10, is based on the ultimate static force of a typical embankment material.

---

**Figure 3.23 Schematic of Seat-Type Abutment (Kramer et al. 2008)**
3.5 Summary

This chapter presents a review of SSI effects on bridge components and various SSI modeling methods. The generation of synthetic ground motion in accordance with NYC metropolitan geological conditions has been discussed since recorded ground motions in NYC area aren’t available. A simplified 3-D SSI modeling method has been investigated for modeling of the bridge foundation for the fragility analysis. Capability of this method in modeling SSI effects has been verified through comparisons with experimental results available in the literature. Finally, the soil-abutment interaction has been briefly introduced. Because of complexity of soil-abutment interactions, conventional lumped spring method has been adopted for modeling soil-abutment interactions.
Chapter 4 Seismic Fragility Analysis of Typical NYC Metropolitan Area Bridges

4.1 Methodology

Mathematical speaking, fragility is defined as the conditional probability that a certain random variable will meet or exceed a predefined value under a given condition. In structural engineering, this certain random variable can be the response or performance of a structure or a structural component, the predefined value can be certain level of capacity of the structure or component, and the given condition can be various loads including man-made or natural hazards that the structure can be subjected to. Hence, seismic fragility of bridges is defined as the conditional probability that a bridge or bridge component (bearing, pier, etc.) would meet or exceed a certain limit state under the effects of a given earthquake event. The limit states of the bridge are chosen such that they have some relation to the operation or functionality of the bridge and are also represented as bridge capacity. This probability of exceedance is defined as,

\[
P_f = P\left[\frac{S_D}{S_C} \geq 1\right]
\]  

(4.1)

where \(P_f\) is the failure probability for a specific limit state, \(S_D\) is the demand and \(S_C\) is the capacity. It should be noted that Equation 4.1 only defines value for the probability under certain seismic load because bridge demand \((S_d)\) depends on earthquake ground motion intensity.
The systematic procedure to generate fragility curves for typical NYC bridges is presented in figure 4.1. As shown in the flow chart, synthetic ground motions are generated, as described in Chapter 3. Statistical samples of a bridge to be studied are created by considering variations in different random variables affecting the behavior of the bridge. Unique sets of bridge models are created by combining variations in different random variables using approaches such as Latin Hypercube Sampling method. Samples of bridges are modeled in OpenSees software. Each of these bridge samples is matched with certain number of earthquake ground motions to generate bridge-earthquake sample pairs. For each sample pair, nonlinear time-history analysis is carried out to generate seismic response data. Along with the deterministic bridge capacity when constructing bridge analytical model, a probabilistic analysis for bridge response (seismic demand) and capacity of certain limit state is carried out to generate the fragility curves as a function of earthquake characteristic parameter(s), for example, Peak Ground Acceleration (PGA).
Figure 4.1 Procedure of Generating Fragility Curves
4.2 Structural Modeling and Analysis of Typical Bridges

4.2.1 Typical Bridges in NYC Metropolitan Area

Pan (2007) has investigated the National Bridges Inventory data (2003) and has identified two types of bridges as typical for the New York State: (1) multi-span simply supported (MSSS) steel bridges and (2) multi-span continuous (MSC) steel bridges.

Based on the inventory data mentioned above, 58.2% of the highway bridges in New York State are multi-beam types and 65.6% of the bridges have steel superstructures. Typical bridges most commonly studied for seismic risk assessment are multi-span bridges with the consideration that no detailed seismic analysis is required for single span bridges [AASHTO (2009)]. Generally speaking, multi-span bridges are either simply supported or continuous with different superstructure types. Among all multi-span bridges in New York State, 70% are simply supported and 27% are continuous.

In terms of seismic fragility, there's no significant difference between the two typical types of bridges in New York State and MSC bridges are generally less fragile [Pan (2007)]. One recommended seismic retrofit strategy for MSSS bridges is to simultaneously increase the continuity of the spans, install elastomeric bearings to enhance its structural integrity and address water leakage issues. Hence, multi-span continuous steel bridges have been considered to carry out the fragility analysis by including soil-structure interaction (SSI) effects. Other types of bridges have not been included in the fragility analysis because of significant computational efforts required for the three dimensional (3D) SSI analysis.
4.2.2 Analytical Bridge Model

Figure 4.2 shows a hypothetical typical multi-span continuous bridge considered for fragility analysis in this research. The three-span continuous steel plate girder bridge with 98 ft end spans and 118 ft middle span has been selected on the basis of typical design details for each component obtained from the review of bridge design drawings provided by New York State Department of Transportation (NYSDOT). The superstructure of the bridge consists of a 48 ft wide, 10 inches thick continuous cast-in-place composite concrete deck with 6 steel plate I-girder equally spaced at 8 ft. Each column bent consists of 3 ft × 4 ft rectangular section cap beam and three 16 ft high, 3 ft diameter circular columns. Two sets of elastomeric bearings have been installed at the abutments and column bents, while seat-type cantilever abutments with U-shaped wing walls supported on cast-in-place concrete piles support the end bearings of the superstructure. Column bents are also supported by 24 cast-in-place piles. The material used for bridge pier is the concrete with nominal $f'_c = 3$ ksi compressive strength and reinforced with #8 Grade 40 bars (vertically) and #3@6" bars (transversely). (Pan, 2007)
A three-dimensional finite element model of the bridge has been developed using OpenSees software. OpenSees is an object-oriented framework for finite element analysis and its intended users are in the research community. A key feature of OpenSees is the interchangeability of components and the ability to integrate existing libraries and new components into the framework without the need to change the existing code. In terms of abilities of performing finite element analysis, the major advantages of OpenSees are (1) open source and (2) free-style programming. The most significant disadvantage is that the graphic user interface (GUI) isn’t sufficiently user-friendly. However, OpenSees can be used for modeling both soil and structure to develop analytical model of the bridge with SSI effects.
Figure 4.3 shows the analytical model of a typical multi-span continuous bridge. Parameters which can be treated as random variables are set to their mean values for developing basic bridge model.

Figure 4.3 Three-dimensional Finite Element Model of a Typical MSC Bridge

As shown in figure 4.3, the deck and steel girder are combined together and modeled as elastic beam elements. All elastomeric bearings are modeled using hysteretic link elements with the shear-displacement behavior (Pan, 2007) shown in the Figure 4.4 below. Locations of bearings EB1 and EB2 are indicated in Figure 4.2. Bridge piers are modeled using displacement based element available in OpenSees. Basically, OpenSees provides two types of nonlinear beam-column elements: (1) Force based elements, which include distributed plasticity and concentrated plasticity with elastic interior, and (2) Displacement based elements, which include distributed plasticity with linear curvature.
distribution. Since curvature of the pier is an important measure of the pier behavior, the nonlinearity of the whole pier and linearity of the curvature of the pier need to be considered simultaneously. Hence, nonlinear displacement based elements are chosen for modeling columns. It should be noted that instead of explicitly specifying the plastic hinge in the force based elements, nonlinear displacement based elements consider the nonlinearity in the pier implicitly. Furthermore, the nonlinear constitutive moment-curvature curve of the pier can be explicitly shown by section analysis or push-over analysis.

![Graphs showing shear force-displacement models of elastomeric bearings](image)

(a) Bearing EB1  
(b) Bearing EB2

**Figure 4.4 Shear Force-displacement Models of Elastomeric Bearings (Pan, 2007)**

Figure 4.5 shows moment-curvature relationship for piers. It is observed that the actual nonlinear behavior of columns can be idealized as elastic-perfectly-plastic bilinear model. In Figure 4.5, the equivalent yield curvature \( \phi_i \) is found by extrapolating the line joining the origin and the point corresponding to the first yield point of a reinforcement bar, up to the nominal moment capacity \( M_n \), \( M_n \) being the moment corresponding to a compressive strain of \( \varepsilon_c = 0.005 \) in the extreme concrete fiber.
More detailed information on modeling details of the bridge superstructure, bearings and piers can be found in Pan (2007). In this research work, the superstructure and substructure of the bridge is the same as that in Pan (2007), although Pan (2007) used SAP2000 to develop the bridge model for the fragility analysis. However, Pan (2007) didn’t consider detailed SSI in their research.

The approaches for the modeling of abutments and foundation have been discussed in Chapter 3. The concern that needs to be addressed in this chapter is "convergence" of soil model. As mentioned previously, two most important problems during the numerical soil-structure interaction analysis are: (1) determination of boundary conditions between soil and structural members and (2) numerical convergence of soil simulation. The simulation of boundary conditions is still an ongoing problem in the soil modeling field. Recent methods, such as domain reduction method (DRM), are increasingly allowing for a more accurate simulation of the 3D seismic wave propagation. In this research, since the main purpose is generating fragility curves with the consideration of SSI, absorbing boundary conditions presented in Section 3.3.1 have been adopted.
“Convergence” of soil model results may either imply that entire computation has converged or results have converged to a reliable value. The first one can be easily identified by judging if the computing has been completed successfully. The second one has to be fulfilled by continuously refining the element size until the difference between two simulations is relatively small or acceptable.

The accuracy of a numerical simulation of seismic wave propagation in a dynamic soil-structure interaction analysis is primarily affected by two parameters: (1) the spacing of nodes $\Delta h$ in finite element method and (2) length of time step $\Delta t$. Based to previous research work, the maximum grid spacing ($\Delta h$) should be controlled by Equation 4.1,

$$\Delta h \leq \frac{v_{\text{min}}}{10 f_{\text{max}}}$$

(4.1)

where $v_{\text{min}}$ is smallest wave velocity, and $f_{\text{max}}$ is the highest relevant frequency of input motions, typically assumed to be 10 Hz for seismic analysis. The time step ($\Delta t$) needs to be limited as per Equation 4.2,

$$\Delta t \leq \frac{\Delta h}{v_{\text{max}}}$$

(4.2)

In Eq.(4.2), $v_{\text{max}}$ is the highest wave velocity. Trial and error method has been implemented to obtain proper values for these two parameters. In this work, $\Delta h = 2$ ft and $\Delta t = 0.005$ s have been selected based on trial and error method.

In order to quantify effects of modeling SSI in the bridge model, an analytical model of the bridge without SSI part has also been developed. In this model, termed as “Fixed
Model”, the bottoms of piers are fixed. The ground motion applied to the bottom of piers has been developed through site-specific analysis by propagating the seismic waves from the rock surface through the soil medium to the bottom of bridge piers, as illustrated in Figure 4.6(b).

![Figure 4.6 Three-dimensional Finite Element Model of a Typical MSC Bridge without SSI effects](image)

(a) Bridge Model  (b) Soil Model

Tables 4.1 and 4.2 show the criteria to classify the rock under soil sites (Risk Engineering Inc., 2002). In NYC metropolitan area, the average shear wave velocities of rock and soil sites vary in the range of 2500 ft/s to 5000 ft/s and 600 ft/s to 1200 ft/s, respectively, as presented in Chapter 4. In section 4.2.1 of this dissertation, design spectra for the rock
sites Very Hard Rock (VHR), Rock Site A and Rock Site B recommended in the seismic design guideline by the NYCDOT are presented. However, ground motions applied to the bridge directly should follow the design spectra based on the soil site class.

Table 4.1 Rock Classification Under Soil Sites

<table>
<thead>
<tr>
<th>Rock Class</th>
<th>Name</th>
<th>Average Shear-Wave Velocity $\bar{V}_{s20}$ ft/sec</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>A / VHR</td>
<td>Hard Rock</td>
<td>$\bar{V}_{s20} &gt; 5,000$</td>
<td>Rock Class A / VHR shall be established only by measured $V_s$.</td>
</tr>
<tr>
<td>B</td>
<td>Rock (or Cemented or Very Dense Soil)</td>
<td>$2,500 &lt; \bar{V}_{s20} \leq 5,000$</td>
<td>Assignment of Rock Class B for cemented or very dense soil shall be based on shear wave velocity measurements. Rock Class B may be assigned for moderately fractured and weathered rock.</td>
</tr>
</tbody>
</table>
### Table 4.2 Soil Classification at Soil Sites

<table>
<thead>
<tr>
<th>Soil Class</th>
<th>Soil General Description</th>
<th>Average Shear-Wave Velocity $\bar{V}_{s100}$ ft/sec</th>
<th>Average Undrained Shear Strength $\bar{\sigma}_u$ psf</th>
<th>Average Penetration Resistance $\bar{N}$, $\bar{N}_{ch}$ blows/ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>C</td>
<td>Very Dense Soil</td>
<td>$1,200 &lt; \bar{V}_{s100} \leq 2,500 , \star$</td>
<td>$\bar{\sigma}_u &gt; 2,000 , (100)$</td>
<td>$(\bar{N} \text{ or } \bar{N}_{ch}) &gt; 50$</td>
</tr>
<tr>
<td>D</td>
<td>Stiff Soil</td>
<td>$600 \leq \bar{V}_{s100} \leq 1,200$</td>
<td>$1,000 \leq \bar{\sigma}_u \leq 2,000$</td>
<td>$15 \leq (\bar{N} \text{ or } \bar{N}_{ch}) \leq 50$</td>
</tr>
<tr>
<td>E</td>
<td>Non Special-Investigation Soft Soil</td>
<td>$\bar{V}_{s100} &lt; 600$</td>
<td>$\bar{\sigma}_u &lt; 1,000$</td>
<td>$(\bar{N} \text{ or } \bar{N}_{ch}) &lt; 15$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Any profile with more than 10 feet of soil having the following characteristics:</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>1. Plasticity index $PI &gt; 20$</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>2. Moisture content $w \geq 40%$</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>3. Undrained shear strength $\bar{\sigma}_u &lt; 500$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>F</td>
<td>Special-Investigation Soft Soil</td>
<td>Require Site-Specific Investigation/Analyses. Include any profile containing soils with one or more of the following characteristics:</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils (see Section 8), quick and highly sensitive clays, collapsible weakly cemented soils.</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>2. Peats and/or highly organic clays ($H &gt; 10$ ft, where $H =$ total thickness of peat and/or highly organic clay soil layers).</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>3. Very high plasticity clays ($H &gt; 25$ ft with plasticity index $PI &gt; 75$).</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>4. Very thick soft/medium stiff clays ($H &gt; 120$ ft) with $\bar{\sigma}_u &lt; 1,000$ psf.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Tables 4.3 to 4.5 (Risk Engineering Inc., 2002) present horizontal ground motion design spectra for soil sites of NYC metropolitan area for return periods of 500, 1500 and 2500 Yr, respectively. In this research, soil class D with 40 ft thick soil on top of rock class B has been considered. Hence, following the method discussed in Chapter 4, ten (10) design spectra with PGAs in the range of 0.1g to 1.0g have been generated using the spectra for soil site D on top of rock class B.

Table 4.3 500 yr Return Period - NYC Soil Sites - Horizontal Design Spectra (g)

<table>
<thead>
<tr>
<th>Period (sec)</th>
<th>Soil Class C</th>
<th>Soil Class D</th>
<th>Soil Class E</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>PGA</td>
<td>0.18</td>
<td>0.18</td>
<td>0.18</td>
<td></td>
</tr>
<tr>
<td>0.02</td>
<td>0.40</td>
<td>0.40</td>
<td>0.31</td>
<td></td>
</tr>
<tr>
<td>0.10</td>
<td>0.40</td>
<td>0.40</td>
<td>0.31</td>
<td></td>
</tr>
<tr>
<td>Tₛ</td>
<td>0.40</td>
<td>0.40</td>
<td>0.31</td>
<td></td>
</tr>
<tr>
<td>0.50</td>
<td>0.11</td>
<td>0.11</td>
<td>0.11</td>
<td>Tₛ= 0.20 seconds for Classes C,D</td>
</tr>
<tr>
<td>1.00</td>
<td>0.03</td>
<td>0.06</td>
<td>0.06</td>
<td>Tₛ= 0.29 seconds for Class E</td>
</tr>
<tr>
<td>2.00</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
<td></td>
</tr>
<tr>
<td>4.00</td>
<td>0.01</td>
<td>0.01</td>
<td>0.01</td>
<td></td>
</tr>
</tbody>
</table>

Soil on Top of Rock Class B - Hr<100 ft

<table>
<thead>
<tr>
<th>Period (sec)</th>
<th>Soil Class C</th>
<th>Soil Class D</th>
<th>Soil Class E</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>PGA</td>
<td>0.18</td>
<td>0.18</td>
<td>0.18</td>
<td></td>
</tr>
<tr>
<td>0.02</td>
<td>0.40</td>
<td>0.40</td>
<td>0.35</td>
<td>Tₛ= 0.20 seconds for Classes C,D</td>
</tr>
<tr>
<td>0.10</td>
<td>0.40</td>
<td>0.40</td>
<td>0.35</td>
<td>Tₛ= 0.27 seconds for Class E</td>
</tr>
<tr>
<td>Tₛ</td>
<td>0.40</td>
<td>0.40</td>
<td>0.35</td>
<td></td>
</tr>
<tr>
<td>0.50</td>
<td>0.16</td>
<td>0.16</td>
<td>0.16</td>
<td></td>
</tr>
<tr>
<td>1.00</td>
<td>0.05</td>
<td>0.08</td>
<td>0.08</td>
<td></td>
</tr>
<tr>
<td>2.00</td>
<td>0.03</td>
<td>0.03</td>
<td>0.03</td>
<td></td>
</tr>
<tr>
<td>4.00</td>
<td>0.01</td>
<td>0.01</td>
<td>0.01</td>
<td></td>
</tr>
</tbody>
</table>

Soil on Top of Deep Rock of Any Type - Hr>100 ft

<table>
<thead>
<tr>
<th>Period (sec)</th>
<th>Soil Class C</th>
<th>Soil Class D</th>
<th>Soil Class E</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>PGA</td>
<td>0.18</td>
<td>0.18</td>
<td>0.12</td>
<td></td>
</tr>
<tr>
<td>0.02</td>
<td>0.31</td>
<td>0.31</td>
<td>0.25</td>
<td>Tₛ= 0.20 seconds for Classes C,D</td>
</tr>
<tr>
<td>0.10</td>
<td>0.31</td>
<td>0.31</td>
<td>0.25</td>
<td>Tₛ= 0.33 seconds for Class E</td>
</tr>
<tr>
<td>Tₛ</td>
<td>0.31</td>
<td>0.31</td>
<td>0.25</td>
<td></td>
</tr>
<tr>
<td>0.50</td>
<td>0.17</td>
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<td>0.17</td>
<td></td>
</tr>
<tr>
<td>1.00</td>
<td>0.07</td>
<td>0.11</td>
<td>0.11</td>
<td></td>
</tr>
<tr>
<td>2.00</td>
<td>0.04</td>
<td>0.06</td>
<td>0.06</td>
<td></td>
</tr>
<tr>
<td>4.00</td>
<td>0.02</td>
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<td>Soil Class D</td>
<td>Soil Class E</td>
<td>Notes</td>
</tr>
<tr>
<td>-------------</td>
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</table>

Ts = period at which the spectral acceleration values start decreasing
$T_s$ = 0.20 seconds for Classes C,D
$T_s$ = 0.30 seconds for Class E

Soil on Top of Rock Class B - Hr<100 ft

<table>
<thead>
<tr>
<th>Period (sec)</th>
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<th>Soil Class D</th>
<th>Soil Class E</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>PGA</td>
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<td>0.42</td>
<td>0.35</td>
<td></td>
</tr>
<tr>
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<td>0.95</td>
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</tr>
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Ts = 0.20 seconds for Classes C,D
Ts = 0.27 seconds for Class E

Soil on Top of Deep Rock of Any Type - Hr>100 ft

<table>
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<th>Soil Class D</th>
<th>Soil Class E</th>
<th>Notes</th>
</tr>
</thead>
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Ts = 0.20 seconds for Class C
Ts = 0.25 seconds for Class D
Ts = 0.37 seconds for Class E
Table 4.5 2500 yr Return Period - NYC Soil Sites - Horizontal Design Spectra (g)

**Soil on Top of Rock Class A / VHR - Hr<100 ft**

<table>
<thead>
<tr>
<th>Period (sec)</th>
<th>Soil Class C</th>
<th>Soil Class D</th>
<th>Soil Class E</th>
<th>Notes</th>
</tr>
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<td>Ts= period at which the spectral acceleration values start decreasing</td>
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</tr>
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</table>

**Soil on Top of Rock Class B - Hr<100 ft**

<table>
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<th>Soil Class C</th>
<th>Soil Class D</th>
<th>Soil Class E</th>
<th>Notes</th>
</tr>
</thead>
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<tr>
<td>PGA</td>
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<td>0.58</td>
<td>0.51</td>
<td>Ts= 0.20 seconds for Classes C,D</td>
</tr>
<tr>
<td>0.02</td>
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**Soil on Top of Deep Rock of Any Type - Hr>100 ft**

<table>
<thead>
<tr>
<th>Period (sec)</th>
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<th>Soil Class D</th>
<th>Soil Class E</th>
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<td>0.46</td>
<td>0.31</td>
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<tr>
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<td>0.67</td>
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<td>0.09</td>
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</tbody>
</table>
Detailed numerical simulations have been carried out using the two cases of bridges: the bridge model with detailed soil model representing SSI effects, termed as “SSI Model” and the bridge model with fixed column bases where seismic grounds motion from site specific study is applied (termed as “Fixed Model”). For illustration purposes, Figure 4.7 presents the pier curvature time-history responses under the PGA of 1.0 g ground motion for these two cases of bridges. It is observed from Figure 4.7 that the pier curvature responses in case of the bridge with SSI effects have been reduced significantly compared to the case of the bridged with fixed column bases. It is recalled from the verification cases in Chapter 3 that this difference is more significant than similar cases of SSI and pure site specific analysis in Chapter 3. This is because structural frequency contents and specific base mode frequency, (fundamental period) are quite different for these two cases, although the difference between ground surface motion is small.

**Figure 4.7 Pier Response Comparison under PGA = 1.0 g Ground Motion**
4.2.3 Parametric and Uncertainty Analysis

For fragility analysis of a typical bridge without considering SSI effects, extensive parametric analyses have been carried out by several research groups. One typical result of the parametric study carried out by Pan (2007) is shown in Figure 4.8. The result indicates that the uncertainties associated with five parameters, concrete compressive strength, reinforcement yield strength, gap size, friction coefficient of expansion bearing and superstructure weight, must be considered in the development of fragility curves.
Figure 4.8 Results of Parametric Study of Multi-span Continuous Bridges (Pan, 2007)
Figure 4.9 Results of Parametric Study of Elastomer Shear Modulus for MSC Bridges (Pan, 2007)
While Pan et al. (2007) considered steel bearings in bridges, elastomeric bearings have been installed in place of steel bearings in this research. Hence, further parametric study has been carried out to study the sensitivity of bridge pier curvature ductility to variations in shear modulus of elastomers. Figure 4.9 shows plots of pier curvature ductility and deck displacement for variations in shear modulus of elastomers. It is observed that although the deck displacement is less sensitive to variations in the shear modulus of elastomers, pier ductility seems to be relatively more sensitive, particularly at higher PGAs. However, overall, the pier curvature ductility of piers can be considered insensitive to variations in shear modulus of elastomers.
Figure 4.10 Results of Soil Properties Parametric Study
Since the fragility analysis in this research also includes soil behavior, uncertainties in soil parameters also need to be included in the fragility analysis if the bridge response is found to be sensitive to variations in soil parameters. This has been done by considering uncertainties in soil shear modulus, soil shear strength and initial stiffness of the connection of soil and structure. Figure 4.10 shows the results of the sensitivity study. It is observed from Figure 4.10 that both shear modulus and shear strength of soil have a significant effect on pier ductility during strong motions, while initial stiffness of the connection between soil and structure is not a sensitive parameter, even for bridges subjected to earthquake with PGA up to 1.0 g. However, shear modulus and shear strength are not independent variables and they have similar trend, as observed from Figure 4.10. Hence, uncertainty only in the shear modulus of soil has been considered during the fragility analysis to account for uncertainty in the behavior of soils during earthquakes.

Based on the discussion presented above, seismic fragility analysis of the bridge with SSI effects has been carried out by considering uncertainties in the following five (5) parameters: concrete compressive strength, reinforcement yield strength, gap size, superstructure weight, and shear modulus of soil.

Fragility analysis of the bridge requires unique samples of bridges by considering range of uncertainties in five parameters identified above. This has been done through the Latin Hypercube Sampling (LHS) approach, which is a probabilistic simulation method used to obtain a set of parameter samples to achieve the high levels of accuracy while reducing the sample size. Probability distributions are assumed for each parameter, and the probability density function of each random variable is divided into a histogram with
equal probability intervals so that the corresponding cumulative distributions are graded linearly. For example, Figure 4.11 shows the probability density function of a normally distributed variable divided into six (6) strips of equal areas. The intersecting points on the horizontal axis are the Latin Hypercube Samples for this normal distribution variable and the cumulative distributions corresponding to these samples increase linearly from approximately 8.35% to 91.65% from left to right.

Figure 4.11 Generation of Parameter Samples by Latin Hypercube Sampling Method

The probability density and cumulative distribution functions for the normally distributed variable are expressed by the following equations,

\[
f_x(x) = \frac{1}{\sigma \sqrt{2\pi}} \exp \left[ -\frac{1}{2} \left( \frac{x - \mu}{\sigma} \right)^2 \right] \tag{4.3}
\]

\[
P(X \leq b) = \frac{1}{\sigma \sqrt{2\pi}} \int_{-\infty}^{b} \exp \left[ -\frac{1}{2} \left( \frac{x - \mu}{\sigma} \right)^2 \right] dx = \phi \left( \frac{b - \mu}{\sigma} \right) \tag{4.4}
\]

where \( \mu \) and \( \sigma \) are the mean and standard deviation of the variant.
For variables assumed to follow lognormal distribution, the LHS procedure is the same as in Figure 4.11, whereas the probability density function and cumulative distribution function are expressed by the following equations,

\[
f_X(x) = \frac{1}{\zeta_x \sqrt{2\pi}} \exp\left[-\frac{1}{2} \left(\frac{\ln x - \lambda}{\zeta}\right)^2\right]
\]

(4.5)

\[
P(X \leq b) = \int_0^b \frac{1}{\zeta_x \sqrt{2\pi}} \exp\left[-\frac{1}{2} \left(\frac{\ln x - \lambda}{\zeta}\right)^2\right] dx = \phi\left(\frac{\ln b - \lambda}{\zeta}\right)
\]

(4.6)

where \( \lambda = \ln \mu - \frac{1}{2} \zeta^2 \) and \( \zeta^2 = \ln\left(1 + \frac{\sigma^2}{\mu^2}\right) \), if \( \text{COV} \leq 0.03 \), \( \zeta \approx \text{COV} \); \( \lambda \) and \( \zeta \) are the mean and standard deviation of \( \ln x \); \( \mu \) and \( \sigma \) are the mean and standard deviation of \( x \).

Uncertainties in different bridge parameters except soil shear modulus are based on Pan (2007) and are summarized in the following.

**Bridge Superstructure Weight:** It is assumed that the superstructure weight is normally distributed with a bias \( \lambda = 1.05 \), and a coefficient of variation (COV) = 0.10. For the typical MSC bridges in Figure 4.2, the total nominal distributed weight for the superstructure is equal to 630 lb/in.

**Yield Strength of Reinforcement Steel:** Lognormal distribution is assumed. Nominal strength is 40 ksi with COV of 0.117.

**Concrete Compressive Strength:** Normal distribution is assumed. Nominal strength is 3 ksi with COV of 0.16.
**Gap Size:** Considerable variations in temperature conditions will cause large deviations in a gap's size from its nominal value. The change in gap size because of temperature variations is \( \alpha \times \Delta T \times L \). Coefficients of thermal expansion \( \alpha \) for steel and concrete are in the range of \((6.1 \sim 6.7) \times 10^{-6} \, /\, ^{\circ} F\) and \((4.1 \sim 7.3) \times 10^{-6} \, /\, ^{\circ} F\), respectively. For a composite deck, the coefficient of thermal expansion \( \alpha \) of the composite superstructure can be calculated as

\[
\alpha = \frac{\alpha_c \Delta TL + \frac{FL}{E_c A_c}}{\Delta TL}
\]

(4.7)

where \( F \) is the force transmitted between the concrete deck and steel beams to ensure compatibility of the displacement, and is determined by

\[
\alpha_c \Delta TL + \frac{FL}{E_c A_c} = \alpha_s \Delta TL - \frac{FL}{E_s A_s}
\]

(4.8)

By taking the mean values of coefficient \( \alpha \) for steel and concrete, the coefficient of thermal expansion of the composite superstructure, \( \alpha \), is calculated to be \(6.0 \times 10^{-6} \, /\, ^{\circ} F\). For the MSC bridge, the gap between the deck and abutment is assumed to be 3 inches.
for as-built conditions, $L = 66$ m for the right span and $L = 30$ m for the left span, as shown in figure 4.12. The values for the samples of gap sizes are calculated by subtracting $\alpha \Delta L$ from the nominal values of each gap. The uncertainties of gap size due to temperature changes are thus considered.

**Elastomer Shear Modulus:** Normal distribution is assumed. Mean value of 112.5 psi with COV of 0.095.

**Soil Shear Modulus:** Normal distribution is assumed. Based on Oh-Sung and Elnashai (2010), the nominal reference shear modulus considered is 21.76 ksi, with the COV of 0.38.

### 4.2.4 Bridge Model Sampling

Probability distributions of each of the variables can be divided into a number of regions of equal areas. Assuming six (6) divisions of probability distributions, as illustrated in Figure 4.11, there will be 36 divisions for six (6) random variables identified previously. A sample bridge model with uncertainties in these parameters can be developed by selection one of the divisions for each of the random variables randomly. Hence, large number of bridges representing uncertainties in six (6) random variables can be developed by selected these random variable divisions randomly. In order to simplify simulation efforts, LHS approach has been used to represent an equal probability distribution for a random variable in one of the six bridges only once.

After dividing probability distributions of each of the six (6) random variables into six divisions of equal areas, pairing approach is adopted to generate a bridge sample. It should be noted that even though two variables are sampled independently and paired randomly,
the sample correlation coefficient of the \( n \) pairs of variables in either a random sample or a Latin hypercube sample will, in general, be not equal to zero due to sampling fluctuations. In order to obtain a sample in which the sample correlations more nearly match assumed or intended correlations, Iman and Conover (1982a) proposed a method for restricting the way in which the variables are paired. The effect of this restriction on the statistical properties of the estimated distribution of \( Y \), its mean and percentiles, is believed to be small. The LHS supports both random pairing and restricted pairing of variables. We have adopted restricted pairing approach in this research, which is illustrated Table 4.6. The basic concept of restricted paring is to pair the parameters such that the correlation matrix is close to the actual correlation matrix. Since six (6) random variables are assumed to be independent, the correlation between any two parameters should be close to zero. This means the paring makes the correlation matrix close as an identity matrix, where the diagonal terms are unity and all off-diagonal terms are very small in magnitude.

Table 4.6 shows 6 sample bridges created by considering randomness in six variables discussed above. Each of the columns in Table 4.6 represents one bridge. For example, Bridge model 1 in Column 1 considers division 1 for steel strength, division 2 for concrete strength, division 1 for superstructure weight, division 3 for gap, division 6 for shear modulus of elastomers and division 5 for soil shear modulus. These divisions of random variables haven’t been considered into any other bridge model.
Each of the six bridge models in Table 4.6 are paired with 10 ground motions to generate sixty (6x10) bridge-earthquake pairs for the structural demand analysis. Although truly random combination of random variables and ground motions can lead to very large number of bridge-earthquake pairs, 60 bridge-earthquake pairs are statistically sufficient for the fragility analysis. It should be noted that one case of three dimensional dynamic time-history analysis of the bridge with the 3D soil model takes more than 2 days of
computational time. Large number of bridge samples will increase the computational time drastically. Also, unlike traditional Monte Carlo Sampling, LHS ensures that a relatively small number of samples account for sufficiently accurate representation of uncertainties in parameters.

4.3 Evaluation of Capacity of Bridge and Its Components

Statistical data on structural demand corresponding to peak ground accelerations (PGAs) from 0.1g to 1.0g can be generated through structural analysis of bridge-earthquake pairs. In order to carry out fragility analysis of the bridge, calculation of capacity data for different components of the bridge is described next.

4.3.1 Capacity Estimation of Bridge Components

It is well known from past research and experience during earthquakes that the vulnerabilities of bridges during an earthquake event are mainly due to damages to critical components, such as bearings and piers. For instance, large relative displacements at the bearing joints may result in unseating of the deck, resulting in unsupported superstructure, while excessive movements of piers, rotational or translational, may result in the failure of the pier in flexure or shear. The capacity of bridge components can be determined from the analytical model of the bridge for a particular level of damage. These capacities, also called as limit states of the bridge components, can be used for defining limit states of the entire bridge.

As discussed above, bearings and piers are critical and representative components of a bridge system. Though failure of superstructure, abutment and soil foundation have been observed to happen, most of the failures of bridges during earthquakes have been because
of failure of bearings and/or piers. Past research results by Pan (2007) also indicate that the probability of seismic damage to bridge superstructure and abutments is quite low and can be ignored for typical bridges in NYC metropolitan area. Furthermore, although SSI has been included in the analytical model of the bridge, failure of soil foundation is still beyond the scope of this research. Hence, calculations of capacity of bearings and piers are needed for the fragility analysis.

Elastomeric bearings usually experience relatively larger displacement under applied ground motions. The allowable seismic displacement for the rectangular elastomeric bearing is governed by Equation (4.9),

\[ \Delta_b = B \left( 1 - \frac{A'}{A} \right) \]  

(4.9)

where \( A' \) is overlapping of top and bottom area of a bearing at maximum displacement, and \( B \) is the side dimension of the bearing in the direction of the ground motion being applied as shown in figure 4.13.
In Eq. (4.9), the overlap factor $A'/A$ is taken as 0.6 [Pan (2007)]. Hence, the allowable value of $\Delta_b$ is $0.4B$. As presented later in Chapter 5, the longitudinal dimension, $B$, for elastomeric bearings EB1 and EB2 are 10 in and 14 in, respectively. Hence, the allowable displacement capacities against unseating are 4 in for EB1 and 5.6 in for EB2. Though the capacity is relatively high compared to steel bearings, the response of elastomeric bearing is much higher than steel bearing as well. However, the overall probability of failure of bearing is still low for probable earthquake events in NYC metropolitan area. [Pan (2007)]

Pier, as a bending element under axial compression, is usually damaged due to the flexural failure under seismic loads. Past research by Pan (2007) has shown that the piers of typical MSC bridges in NYC area are unlikely to undergo shear failure. Therefore, only flexure capacity analyses of the piers are carried out in this research.
Figure 4.5 shows moment-curvature relationship for the bridge piers. It is observed from Figure 4.5 that the moment-curvature relationship of the pier is idealized as bilinear elastic-perfectly plastic model with critical points associated with the curvature $\phi_y$, $\phi_y$, $\phi_y$, and $\phi_y$. These critical points are related to the extent of damage in a pier under the seismic loads corresponding to different limit states of the pier. The parameter $\phi_y$ indicates the initiation of yielding, the moment associated with this curvature can be found by $M_y = E I \phi_y$. The point at $\phi_y$ defines the formation of a plastic hinge in the pier and it can be calculated when idealized moment capacity, $M_n$, corresponds to the moment when extreme concrete fiber reaches $\varepsilon_c = 0.005$, $\phi_y = \frac{M_n}{E I_i} = \frac{M_n \phi_y}{M_y}$. The critical point $\phi_d$ defines the point of degradation in strength of piers associated with the maximum moment $M_{\text{max}}$. Crushing of concrete occurs at ultimate curvature $\phi_{\text{cu}}$ when the strain in the concrete reaches $\varepsilon_{\text{cu}}$, where $\varepsilon_{\text{cu}}$ accounts for the confining effect of transverse reinforcement. Assuming that the concrete in bridge pier is well confined by the transverse reinforcement in the plastic hinge zone, $\varepsilon_{\text{cu}}$ can be calculated based on the approach by Mander et al. (1988) as 0.12. t

Figure 4.15 shows moment-curvature plots generated by computer programs BIAX [Wallace (1992), Wallace and Ibrahim (1996)] and Opensees. It is observed from Figure 4.15 that the moment-curvature plots by the two computer programs match well and critical points corresponding to four limit states have been found to be almost the same. In this research, moment-curvature plots generated by OpenSees have been used. Figure
4.16 shows the value of $\phi_y$, $\phi_s$, $\phi_d$, and $\phi_u$ obtained for piers of six bridge samples. The capacity difference among samples is due to the variations in material strengths and bridge superstructure weight.

Figure 4.14 Inelastic Behavior of Bridge Piers

Figure 4.15 Comparison Between Pier Moment-Curvature Plots Using OpenSees and BIAX.
Figure 4.16 Pier Limit States Determination for Different Bridge Samples
4.3.2 Limit State of Bridges

Based on the capacity analysis of the bridge components presented above, the following limit states have been considered to develop fragility curves for critical bridge components:

- Four different levels of pier damages corresponding to four curvature thresholds $\phi_y', \phi_y, \phi_d$ and $\phi_u$.

- Unseating/Instability of elastomeric bearings and/or collapse of a pier leading to the failure of the entire bridge system. (Conservative consideration since multicolumn bent usually has redundancy and will not collapse with the failure of a pier).

4.4 Fragility Curves of Bridge Components

As discussed in Section 4.1, seismic fragility is defined as,

$$P_f = P \left[ \frac{S_D}{S_C} \geq 1 \right]$$

(4.1)

In Equation 4.1, the random nature of structural demand $S_D$ and structural capacity $S_C$ are described by the lognormal distribution. Hence, fragility $P_f$ can be expressed as a standard normal distribution,

$$P_f = \Phi \left( \ln \left( \frac{S_D}{S_C} \right) \frac{1}{\sqrt{\beta_d^2 + \beta_c^2}} \right)$$

(4.10)
where $S_c$ is the mean value of the structural capacity defined for the damage state, $\beta_c$ is the lognormal standard deviation of the structural capacity, $S_d$ is the mean value of seismic structural demand in terms of a chosen ground motion intensity parameter, for example, PGA, and $\beta_d$ is the lognormal standard deviation of the structural seismic demand.

It should be noted that there have been several methods to obtain structural demand, such as using elastic response spectral analysis, nonlinear static analysis and nonlinear time-history analysis. In this research, nonlinear time history analysis has been carried out to obtain structural seismic demands for earthquakes of different PGAs.

By performing nonlinear time-history analysis of each of the bridge-earthquake pair, maximum response quantities of different bridge components have been obtained. Ratios of structural demand and capacity ($S_d / S_c$) have been obtained by dividing peak structural demands for different components by corresponding capacities of the bridge components. Since PGA of 10 earthquake ground motions vary from 0.1g to 1.0g, ratio of $S_d / S_c$ can be plotted as a function of PGAs. As described previously and illustrated in Figure 4.17, structural demand ($S_d$) and structural capacity ($S_c$) follow lognormal distribution. Hence, it can be assumed that $\ln(S_d / S_c)$ follows a normal distribution as a function of $\ln$(PGA). The relationship between $\ln(S_d / S_c)$ and $\ln$(PGA) can be obtained through regression analysis of 60 data points obtained through nonlinear time history analysis.
Although traditionally fragility analysis is carried out by considering linear regression between $\ln\left(\frac{S_d}{S_c}\right)$ and $\ln(PGA)$, Pan(2007) has shown that quadratic regression is more representative of the relationship between $\ln\left(\frac{S_d}{S_c}\right)$ and $\ln(PGA)$. Assuming $\lambda$ representing the mean value of $\ln\left(\frac{S_d}{S_c}\right)$, the quadratic regression curve can be expressed as,

$$\lambda = a\left(\ln(PGA)\right)^2 + b\ln(PGA) + c$$

(4.11)

where $a$, $b$ and $c$ are the regression coefficients. The standard deviation for the regression curves is defined by,

$$\zeta = \sqrt{S_r / (n-2)}$$

(4.12)
where \( S_r = \sum_{i=1}^{n} (y_i - \lambda_i)^2 \), the summation of squares of the residuals with respect to (w.r.t.) the regression curve in Figure 4.11. Defining \( S_t = \sum_{i=1}^{n} (y_i - \bar{y})^2 \) as the summation of the squares of the residuals w.r.t. the mean value \( \bar{y} = \frac{1}{n} \sum_{i=1}^{n} y_i \), the coefficient of determination \( r^2 \) is calculated as,

\[
r^2 = 1 - \frac{S_r}{S_t} \tag{4.13}
\]

The coefficient of determination \( r^2 \) indicates the appropriateness of regression equation in Eq.(4.11) for representing the statistical data. A value of \( r^2 \) closer to 1 represents better fit of regression data. Plots in Figure 4.18 show the plots of regression data along with regression equation in Eq.(4.11). These plots also show linear regression equation for comparison. It is observed from Figure 4.18 that the quadratic regression equation in Eq.(4.11) is more representative of the statistical data for \( \ln\left(\frac{S_d}{S_c}\right) \) as a function of \( \ln\text{(PGA)} \). It is also observed that the values of coefficient of determination \( r^2 \) is closer to 1 for quadratic regression than for linear regression.

Once parameters \( \lambda \) and \( \zeta \) have been obtained by regression analysis, fragility curves as a function of PGA can be developed using the Equation 4.14,

\[
P_f = \phi(\frac{\lambda}{\zeta}) \tag{4.14}
\]
Figure 4.18 Linear and Quadratic Regression Analysis of Pier Ductility in MSC Bridge (a) First Yielding (b) Beginning of Plastic Hinge (c) Beginning of Degradation (d) Collapse
Figure 4.18 (a) to (d) present plots of linear and quadratic regressions of column curvature ductility as a function of PGAs for the piers for four damage levels to bridge column corresponding to exceeding the critical curvature $\phi_y$, $\phi_d$ and $\phi_u$.

Figure 4.19 shows fragility curves for piers of the MSC bridges when SSI effects have been included. Figure 4.20 shows fragility curves for piers without including SSI effects.

It is observed from Figure 4.19 and 4.20 that piers in typical MSC bridges in NYC metropolitan area are less susceptible to failure (or are less fragile) when SSI effects have been included. For the bridge piers where SSI effects have been included, median PGAs (corresponding to 50% probability of capacity exceedance) are 0.65g and 0.725g for limit states of first yielding of in column longitudinal reinforcement ($\phi_y$) and beginning of plastic hinge formation ($\phi_d$), respectively. These values are 0.6g and 0.675g, respectively, for the bridge piers when SSI effects have been ignored. The reasons for reduced fragility because of inclusion of SSI effects are: (a) earthquake energy is absorbed by soil layers when ground motion is propagating from the bedrock to the ground surface within the soil layers in the SSI model and (b) fundamental period of the bridge with SSI effects and soil model increases significantly, which results in significantly lesser seismic demand. For example, plots in Figure 4.21 show the Fourier transform of response of pier top for cases without and with SSI effects. It is observed from this figure that the period of the bridge with SSI is elongated, which results in lesser seismic demand on the bridge piers.

It is also observed from Figures 4.19 and 4.20 that the probabilities of pier strength degradation and pier collapse, even under the 2500 Yr earthquake, are very low. As
shown in Table 4.3, PGA for a 2500 yr return period earthquake on rock site B in NYC area is only 0.33g. Hence, probabilities of damage to piers under any of the four limit states ($\phi_y$, $\phi_y$, $\phi_d$, and $\phi_u$) are very low (less than 1%) during the 2500-yr return period earthquake, as observed from Figure 4.19.
Figure 4.19 Fragility Curves for Piers in MSC Bridges with SSI Effects

Figure 4.20 Fragility Curves for Piers in MSC Bridges without SSI Effects
Figures 4.22 and 4.23 show fragility curves for displacements of elastomeric bearings for bridges with and without the consideration of SSI effects, respectively. As observed for the bridge with SSI effects, elastomeric bearings EB1 have higher risk of failure than those of EB2. However, the differences between the fragility curves for these two bearings are relatively small. This is because of higher allowable seismic displacement for EB2 that that for EB1, as discussed previously. The largest probability of failure for EB1 is approximately 13% at a PGA of 1g. However, the probability of failure at 0.33g PGA, which corresponds to 2500 Yr return period earthquake at rock site B, is less than 1%.
Comparing the fragility plots in Figure 4.22 for the bridge with SSI effects to those in Figure 4.23 for the bridge without SSI effects, it is observed that the fragility of collapse by unseating decreases drastically because of inclusion of SSI effects. This happens because of combined effects of dissipation of seismic energy by the soil layers and lengthening of fundamental period because of inclusion of soil models in the bridge with SSI effects.
Figure 4.22 Fragility Curves for Bearings in MSC Bridges with SSI Effects

Figure 4.23 Fragility Curves for Bearings in MSC Bridges without SSI Effects
4.5 Fragility Curves of the Bridge System

Although fragility analysis has been carried out for critical bridge components (piers and bearings), limit states of the entire bridge system depend on the combination of limit states of all critical bridge components. Since the collapse of piers and/or the un-supporting of elastomeric bearings may result in the failure of the bridge system, the fragility curves for bridge is obtained by combining fragility curves for each of the components statistically.

Pan (2007) has discussed two approaches for combining fragility curves of bridge components to obtain fragility curves of the entire bridge system. These two approaches are: first-order reliability bounds by ignoring possible correlations between different failure modes and second-order reliability bounds considering that the failure modes may be correlated. First-order reliability bounds can be expressed as:

\[
\max_{i=1}^{m}[P(F_i)] \leq P_{sys} \leq 1 - \prod_{i=1}^{m}[1 - P(F_i)] 
\]  

(4.15)

where \(P(F_i)\) is the probability of failure in ith mode (or component). For independent failure modes, the system failure probability can be represented by the product of the mode survival probabilities. In the case where all failure modes are fully dependent, the weakest failure mode will always be the most likely failure mode for the bridge system.

Second-order reliability bounds define lower and upper probability bounds as,

\[
p^− = p(F_1) + [p(F_2) - p(F_2 \cap F_1)]^+ + [p(F_3) - p(F_3 \cap F_1) - p(F_3 \cap F_2)]^+ 
\]  

(4.16)

\[
p^+ = p(F_1) + p(F_2) + p(F_3) - [p(F_2 \cap F_1)]^+ - [p(F_3 \cap F_1), p(F_3 \cap F_2)]^+ 
\]  

(4.17)
In Equations 4.16 and 4.17, \([\bullet]\) \(\equiv\) \(\max(\bullet,0)\), where \(F_i\) denotes the event "failure of the bridge due to failure in the \(i^{th}\) mode", and \((F_i \cap F_j)\) is the event that failure occurs in both the \(i^{th}\) and \(j^{th}\) modes. Hence the joint probability between \(F_i\) and \(F_j\) is defined as,

\[
p(F_i \cap F_j) = p \left( \frac{S_{di}}{S_{ai}} \geq 1, \text{and} \frac{S_{dj}}{S_{aj}} \geq 1 \right) = p \left( \ln \left( \frac{S_{di}}{S_{ai}} \right) \geq 0, \text{and} \ln \left( \frac{S_{dj}}{S_{aj}} \right) \geq 0 \right)
\]  

(4.18)

Based the fragility curves for bridge components presented in previous subsection, it is noted that the first three failure modes of pier damage control first several damage levels of the whole bridge, and bearing unseating/un-supporting failure mode controls the ultimate damage mode of the bridge system. However, the probabilities of the ultimate damage for both pier and bearing are significantly low because of very low probabilities of failures of bridge components. Consequently, the difference between the results for lower and upper bounds using both first-order and second-order reliability bounds will be very small and will be negligible. Hence, only first-order reliability bounds have been calculated to develop fragility curves for the entire bridge system. These curves have been developed by considering the following damage states for the entire bridge system:

**Slight Damage**: Initiation yielding of longitudinal reinforcement bars in piers

**Moderate Damage**: Formation of plastic hinge in piers

**Extensive Damage**: Strength degradation occurring in piers

**Ultimate Damage**: Bearing un-supporting and/or pier collapse

Figures 4.24 and 4.25 present fragility curves for the bridge with and without consideration of SSI effect, respectively. It is observed from the Figure 4.24 that median
PGAs (corresponding to 50% probability of capacity exceedance) are 0.63 g and 0.7 g for the slight damage and moderate damage to the bridge. For extensive and ultimate damage states, both the PGA values exceed 1.0 g, which implies that the probabilities of occurrence of these two damage modes in a typical MSC bridge in the NYC metropolitan area are significantly low. By extrapolating the results, median PGAs for extensive and ultimate damages will be approximately 1.25 g and 1.4 g. It is observed from Figure 4.25 for the case of the bridge without SSI effects that the median PGAs for four damage states are 0.60 g, 0.66 g, 1.07 g and 1.19 g. Note that the PGAs for extensive and ultimate damage states have been obtained by extrapolation.
Figure 4.24 Fragility Curves for MSC Bridge System with SSI Effects

Figure 4.25 Fragility Curves for MSC Bridge System without SSI Effects
4.6 Summary

In this chapter, detailed fragility analysis of the multi-span steel bridge with and without SSI effects has been carried out. A detailed description of the process of generating fragility curves, including the modeling of the bridge with and without surrounding soil, parametric sensitivity analysis to identify random variables affecting response quantities, generation of synthetic ground motions, Latin Hypercube Sampling (LHS) approach to develop bridges with uncertainties and regression analysis to carry out fragility analysis, has been presented. Fragility curves for bridge piers and elastomeric bearings with and without the inclusion of SSI effects have been developed based on detailed nonlinear time history response analysis. Fragility curves for piers and bearings have been combined to develop first order fragility of the bridge with and without SSI effects. It has been observed that the probabilities of collapse of the bridge components and the bridge system decrease because of the inclusion of SSI effects in the analytical model of the bridge.
Chapter 5 Deterioration Model of Bridges and Its Components

5.1 Introduction

As mentioned in Chapter 3, the comprehensive three-dimensional (3D) finite element model (FEM) of SSI system is computationally expensive. Though combined consideration of SSI effects and deterioration model will be a valuable numerical experiment, this research work presented herein considers deterioration of the bridge independently.

Deterioration of components of bridges is caused because of environmental factors, such as continuous exposure to the chlorine in deicing salts, vehicular collisions which results in cracks on concrete cover, and degradation of materials. Bearings are one of the main elements of a bridge system. The primary functions of bearings are to connect and tie the superstructure and substructure of bridges and allow movement because of temperature or dynamic loads, such as seismic loading. During seismic excitations, bearings transfer forces from the superstructure, where majority of seismic force is applied, to the substructure. Hence, the behavior of the bridge and its components during seismic excitations may depend on deterioration in bearings because of material degradation. This degradation in material may lead to time-dependent fragility of bridge bearings.

Bridge piers are frequently more vulnerable to deterioration because of corrosion in the presence of chlorine from deicing salts. As observed previously, probability of damage to the piers of bridges without corrosion is low. However, the capacity of piers may be reduced significantly because of corrosion of rebars and spalling of concrete. Hence, seismic vulnerability of a bridge with deteriorated column may be significantly higher
than that without corrosion. In this dissertation, fragility analysis has been carried out for the bridge by considering degradation in the elastomeric material of bearings and corrosion of reinforced concrete columns.

5.2 Deterioration Models

5.2.1 Deterioration Effects on Elastomeric Bearings

Elastomeric bearing is one of the most commonly used bridge bearings. A typical laminated elastomeric bearing is shown in Figure 5.1. The elastomeric bearing pad, connecting the girder above and pier cap underneath, consists of a sandwich of mild steel shims and rubber moulded as one unit. The bearing pad is usually under the vertical compression and lateral shear loads, sometimes horizontal torque, simultaneously. In the structural point of view, the initial stiffness of the bearing can be calculated by Equation 5.1 (Choi, 2002)

\[ k_0 = \frac{GA}{h_r} \]

(5.1)

where \( A \) is the area of the elastomeric bearing, \( G \) is the shear modulus of the elastomeric rubber and \( h_r \) is the thickness of the elastomeric pad.
Among different degradation factors such as oxidation, ultraviolet radiation, ozone, temperature, acidity and humidity, it has been observed that thermal oxidation changes the rubber properties, such as shear modulus, more significantly than other factors (Yoshida et al., 2004). Detailed analysis has been done by Itoh et al., 2006b to evaluate the time-dependent changes in horizontal stiffness of the rubber pad.

Accelerated thermal oxidation tests have been carried out in Itoh et al. 2006b based on aforementioned research and a deterioration prediction model is developed to estimate the property profiles. In their proposed prediction model, Itoh et al. 2006b has quantified four general types of deterioration characteristics, namely 1) Critical depth, 2) Property variation of interior region, 3) Property variation at block surface and 4) Shape model of property profile. The feasibility of the deterioration prediction method is verified by comparing to their tests results.

Based on the verified deterioration model, a finite element methodology (FEM) model has been developed to evaluate the time-dependent performance of the elastomeric bridge bearings. Figure 5.2 shows the change in horizontal stiffness of rubber bearings with age, is plotted from their FEM numerical experiments.
As shown in Figure 5.2, the stiffness of the bearing can be considered as a variable with uncertainty. From the figure, one can see that the equivalent horizontal stiffness increases over the time, and it increases much faster during the earlier stage after installation in a bridge. It is also clear that the equivalent horizontal stiffness increases more significantly at a higher temperature. However, the effect of temperature is small when the temperature is below 10°C. Although temperature varies significantly during a year in NYC metropolitan area, it is assumed for simplicity that the change in stiffness of bearings follows uniform distribution during a year.

5.2.2 Deterioration Effects on Columns

Corrosion of reinforcement is considered to be the principal cause of deterioration of reinforced concrete bridge piers. It can affect the residual strength (capacity) of the column in several ways, such as reduction in reinforcement strength, loss of bond, corroded bar length, loss of concrete cover and cross-section asymmetry. Research on the evaluation of the residual strength of the reinforced concrete structures based on effect of
these various variables is still going on. In this research, only the reduction in reinforcement strength because of deterioration, which is the most significant one, is considered for developing fragility analysis.

The yield strength of corroded reinforcement at any time $t$ can be calculated by (Du et al. 2005a and b)

$$f_y(t) = (1 - 0.005m(t))f_{y0}$$

(5.2)

where $f_y(t)$ is the yield strength of corroded reinforcement at each time step, $f_{y0}$ is the yield strength of non-corroded reinforcement, $t$ is the time elapsed since corrosion initiation (year), and $m(t)$ is the percentage of steel mass loss over the time. The rate of mass loss per unit length for a time step of $\Delta t$ (sec) can be described by

$$\Delta M_{loss}(t) = k\pi D(t)i_{corr}\Delta t$$

(5.3)

where $D(t)$ is the reduced diameter of reinforced bar during the corrosion process, $k$ is the mass transport coefficient, and $i_{corr}$ is the current per unit area of the reinforced bar. The reduced diameter $D(t)$ of corroded rebar can be calculated by:

$$D(t) = \sqrt{D_0^2 - \Delta V_{loss}(t)/\pi}$$

(5.4)

where $D_0$ is the initial diameter of the rebar and $\Delta V_{loss}$ is the change in the volume of corroded steel calculated from $\Delta M_{loss}$.

Since both strength of the reinforced bars and the area of the rebar will be affected by deterioration, these two parameters should be included in the uncertainty analysis,
moment-curvature analysis, nonlinear time history analysis and the fragility analysis. The parametric and uncertainty analysis aforementioned in section 4.2.3 indicates that the yield strength of the reinforcement steel is one of the most significant factors need to be considered in the uncertainty analysis. Hence, the calculated residual steel strength based on equation (5.2) will also be a random variable similarly with the yield strength in the intact bridge models. Though the process of deterioration is definitely a random and uncertain process, the distribution of the steel strength remains lognormal distribution considering that the uncertainties in the steel materials have been counted in the probabilistic analysis. For moment-curvature analysis, the parameters in fiber cross-section, such as area of reinforced bars and strength of bars have to be updated based on deterioration model. By combining deterioration effects on both elastomeric bearings and RC columns, the fragility curves for deteriorating highway bridges can be constructed.

5.3 Fragility Curves considering Deterioration Models

The methodology and detailed procedure of fragility analysis in this chapter are the same as those in Chapter 4. The analysis presented in this chapter follows the same procedure with the focus on parameters relative to deterioration models only.

5.3.1 Fragility Curves for Bridges with Bearing Deterioration

Two sets of elastomeric bearings, namely EB1 and EB2, with different designs have been installed for retrofitting the multi-span continuous (MSC) bridge shown in Figure 5.3. Detailed information on the design of elastomeric bearings can be found in Pan (2007). Selection and modeling of typical bridges has been discussed in Chapter 5.
Figure 5.3 Retrofit Scheme for Elastomeric Bearings in MSC Bridges

Figure 5.4 and 5.5 present plan and elevation view of EB1 and EB2 types of bearings. As shown in these figures, six EB1 bearings per line are installed in the transverse direction under the bottom flanges (as wide as 16 inches) of the steel plate I-girder equally spaced at 8 ft on the abutments at the two ends of the bridge. Likewise, twelve EB2 type bearings are installed under the girder over the column bents. Dimensions and lamination details of EB1 and EB2 bearings are shown in Figures 5.4 and 5.5, respectively.

Parametric analyses of variations in shear modulus of elastomers on the response quantities of the bridge have been carried out by Pan (2007). Their results show that the displacements of bearings was sensitive to the shear modulus of elastomers. Hence, Pan (2007) have developed fragility curves for bridge without any deterioration in bearings. In this dissertation, the deterioration effects on elastomeric bearing are only considered to change the horizontal stiffness. It has also been assumed that this change in stiffness follows uniform distribution. Effect of temperature on change in horizontal stiffness of bearings is neglected.

Figure 5.6 shows the fragility curves for a 20-year deteriorated bridge for different limit states. It is observed from this figure that the probability of exceeding any damage state increases over the time due to the deterioration, but these increases are relatively small.
This may be either because of the fact the deterioration of bearings only may not have significant effect on the response. The deterioration time of 20-years also may be relatively short for the deterioration to affect the behavior of the bridge significantly. Hence, the change in stiffness of bearings may only cause minor increase in seismic vulnerability of the bridge.
Figure 5.4 Elastomeric Bearings of the Type EB1
Figure 5.5 Elastomeric Bearings of the Type EB2
5.3.2 Fragility Curves Considering Pier Deterioration

Deterioration of bridge piers could be more serious, since the corrosion of reinforcement bars may reduce the capacity of the column and potentially increase the response of the pier and the whole bridge. Using the bridge example and ground motions generated by the NYC spectra with Class B soil on Class D rock in Chapter 4, the seismic fragility curves for bridge system considering both bearing and pier deterioration can be constructed. As indicated in section 5.2.2, the corroded reinforcement steel has reduced yield strength, which result in reduced capacity of the bridge pier. The moment-curvature analysis with reduced yield steel strength parameter explicitly gives out less allowed...
curvature corresponding to different limit states. On the other hand, the seismic response of the bridge pier may potentially increase. This tendency is implicitly reflected in the structural analysis and will be explicitly shown in the final seismic fragility curves. Although the deterioration parameter, the corroded steel yield strength, is considered as a random variable following lognormal distribution, the randomness in the deterioration procedure is ignored and experimental equation (5.2) which already considered the randomness during the procedure, implies that the final state of the deterioration is determinate.

Figure 5.7 Time-dependent Fragility Curves Considering Elastomeric Bearing and Pier Deterioration
Figure 5.7 shows the fragility curves of the bridge without any deterioration and bridge after 20 years in service by considering the deterioration of the elastomeric bearing and RC column for different limit states.

It is observed from the figure that the probability of failure for any limit state increases more significantly than that because of degradation in elastomers of bearings. This increases the seismic damageability of bridge and makes it more fragile to natural hazards.

It should be noted that the consideration of the deterioration of the stirrups would be more realistic and reliable since the stirrups affect the moment capacity as well as the shear capacity of the pier. However, in this research work, the bridge piers are modeled as nonlinear beam-column displacement based elements, with the assumption that every section of the pier is uniformly and well confined, thus, the consideration of the stirrup deterioration would not be available in the currently presented model. It would be better that 3D solid elements are introduced to the pier model and even more detailed, to the rest components of the bridge, though this will dramatically increases the numerical analysis consumption in term of seismic fragility analysis.

It also should be noted that the fragility analysis carried out in this dissertation considers selected deterioration mechanism to show effects of deterioration on seismic vulnerability. In order to make a realistic estimate of effects of deterioration on seismic vulnerability, all prominent deterioration mechanism should be considered. This work will require extensive work on realistic deterioration modeling based on empirical and analytical
models of deterioration. Time-dependent fragility curves developed through such research will be a key tool in seismic hazard mitigation of aging bridge network.

5.4 Summary

This chapter investigates deterioration in elastomeric bearings and reinforced concrete column and generates fragility curves by considering these deterioration mechanisms. It is observed that while the deterioration of elastomers doesn’t affect the seismic vulnerability significantly, combined deterioration in bearings and reinforced concrete pier increases seismic vulnerability of bridge piers significantly. These results show that aging bridges with deterioration in key elements are more vulnerable to seismic excitations and their vulnerability increases with the progression of deterioration.
Chapter 6 Conclusion and Future Research

6.1 Summary and Conclusions

Little attention was paid to the seismic design and detailing of highway bridges in New York State prior to 1990 due to the underestimation of the seismic hazard. Since then, numerous research studies have been carried out to evaluate the seismic vulnerability of the typical highway bridges in New York State. As a result of these studies, various seismic retrofit strategies have been investigated, proposed and implemented. However, very little consideration has been given to the effects of deterioration and SSI effects on seismic vulnerability of bridges. The main objective of the research in this dissertation has been to generate an improved methodology to take deterioration and SSI into account when performing the seismic fragility analysis for typical MSC highway bridges in the New York City metropolitan area.

The objectives of this research have been achieved through the following two tasks: (i) development of a multi-span steel bridge with detailed model of soil, (ii) development of a model considering prominent deterioration effects. The average soil profiles in the New York City metropolitan area have been considered in this work and trial and error method has been adopted to construct the link behavior at the interface of soil and structure, specifically, soil-to-pile interface. Effects of uncertainties in soils parameters on the behavior of the bridge have been evaluated through detailed sensitivity analysis. Based on uncertainties in material, structural and soil parameters, probabilistic bridge models with consideration of SSI effects have been generated. These models have been
used to generate fragility curves for MSC bridges in the New York City metropolitan area through nonlinear time history analysis.

In order to reasonably estimate the typical bridge response to the possible seismic hazards, several synthetic ground motions have been generated using NYCDOT design spectra. Comparison of the fragility curves considering deterioration with those not considering degradation in elastomer of elastomeric bearings and corrosion of reinforced piers shows that the seismic vulnerability of a typical 20-year deteriorated multi-span continuous bridges increases by 10%~15% compared to those without deterioration. Inclusion of SSI effects in the modeling of the bridge results in lesser seismic vulnerability of bridge components. This happens because of (i) dissipation of seismic energy by the soil layers, (ii) elongation of the effective period of the bridge because of soil model. Specific unique contributions on this research are summarized in the following.

1. **Deterioration model of elastomeric bearing and reinforced concrete piers:**
   
   Fragility curves in prior studies are primarily based on the bridge components without any deterioration. However, deterioration of bridges is inevitable, considering that in-service bridges are continuously exposed to the environment and service factors, such as corrosion, temperature cycles, impact / collisions, material degradation, etc. Deteriorated components of bridges not only compromise the capacity of the bridge system, but also result in possibly more serious responses of the bridges to the same level seismic hazard. Hence, it is essential to take the deterioration model into account, when developing the fragility curves for the bridges in service. It has been observed from the results in this research that although the deterioration of the
elastomer in bearings has lesser effects on the seismic vulnerability of the bridge, the combined effect of deterioration in bearings and reinforced concrete pier because of corrosion increases the seismic vulnerability of bridge components and the bridge system significantly.

2. **Generation of synthetic ground motions:** Ground motion time histories of different PGA, representing earthquakes of return periods 500 Yr, 1500 Yr and 2500 Yr in the New York City area, have been generated to account for uncertainties in ground motions based on design spectra for the rock underneath the soil layers. One of the unique contributions of this study has been the inclusion of site specific analysis automatically in the finite element models when carrying out the structural response analyses.

3. **SSI model and Uncertainty analysis of the soil material:** Previous research work on fragility analysis have either simplified or have ignored the effects of SSI during the time history analysis of bridge response for fragility analysis. In these studies, soil models have been simplified by p-y lateral springs or by fixed boundaries. Neither of these simplifications capture the SSI behaviors well, particularly when simulating near-field motions and the fundamental period of the target structure. The SSI model developed in this dissertation not only simulates the propagation of the earthquake wave from the bedrock to the ground surface where the bridge pier seats, but also considers soil layers underneath bridge pier as a part of the entire structural model, thereby facilitating the simulation of actual structural condition. The SSI model has been validated and verified by comparing the numerical results to those of centrifuge test results. In order to develop probabilistic bridge models with
consideration of the SSI effects, parametric and uncertainty analyses of the soil model have been carried out to account for the uncertainties in the soil material. Reasonable assumption has been made based on analyses results to incorporate soil material uncertainties in the bridge behavior.

4. **Fragility curves for typical MSC bridges in NYC metropolitan area:** Fragility curves for bridge components as well as bridge system have been developed for the bridge models with and without SSI effects. Simulation results show that the seismic fragility of the highway bridge components in the NYC metropolitan area decreases by approximately 10% when SSI effects have been included in the bridge model. This is a significant reduction to the seismic vulnerability of bridges. This decrease, unlike usual increase in other regions, for example, the Central and Southeastern United States, is because of uniqueness of the bedrock motion spectra and soil profile in NYC metropolitan area.

In conclusion, the seismic fragility of the typical MSC bridges in NYC metropolitan area will increase by 10% to 15% when the bridge has been in service and has undergone deterioration for 20 years. On the other hand, seismic fragility decreases by 10% when SSI effects have been included in the bridge analysis model.

### 6.2 Recommendations for Future Research

The present study can be extended through additional research work on the following aspects. The work on these aspects is beyond the scope of this dissertation.

1. Fragility analysis has been done by considering deterioration and SSI effects separately because of excessive simulation time requirements. Since deterioration and
SSI affect the bridge response jointly, deterioration model can be included in the model of the bridge with SSI to carry out fragility analysis. These fragility curves through such analysis will be more realistic. In addition to this, longer service time could be considered to develop fragility curves for deteriorated bridges, since the design life of a typical highway bridge is more than 50 years. Since deterioration of bridges is a nonlinear process, results by considering 20 year service life cannot be directly extended to bridges with longer service life.

2. Although uncertainty analysis for the soil material has been carried out in this dissertation research, more detailed parametric and uncertainty analysis can be carried out to account by uncertainties in soil parameters in more detail. As discussed in Chapter 5, the randomness in soil parameters is very complicated. More detailed modeling of uncertainties in soil can be carried out based on large amount of experimental data available in the literature. This will have significant effect on the propagation of earthquake wave through the soil media and energy dissipation by the soil.

3. Current study considers the damage to the bridge components only. However, the failure of the soil also can cause failure of the entire bridge system. Although the bedrock in the NYC metropolitan region is mostly hard rock or stiff soil, there is still a possibility that the soil supporting the bridge structure may fail under certain conditions, for example because of liquefaction of the soil. In this case, the soil could be treated as a structural component, and limit state(s) analyses as well as the response demand could be carried out to adjust the fragility of the entire bridge system.
4. The SSI model developed in the present research considers only the transmission of the forces at the soil-structure interface. No damping model has been considered in developing the horizontal link spring at the soil-structure interface in the SSI model. It is well known that the damping plays an important role during the dynamic time-history analysis. Additional analytical model of the damping at the soil-structure interface could result in more accurate structural response of bridge components.

5. Vertical ground motion components have been ignored in the time-history analysis and vertical contact behavior at the soil-structure interface have been simplified by q-z and t-z springs. The fluctuation in axial forces are present when taking vertical motions into account during a dynamic analysis. Hence, it is possible that the seismic fragility is underestimated because of exclusion of vertical components of ground motions from the nonlinear dynamic analysis. Although vertical ground motions can be developed using vertical design spectra, modified SSI model(s) considering the transfer of vertical forces need to be considered.
References


Risk Engineering Inc. (2002). "Seismic Hazard for New York City", Final Report to New York City Department of Transportation (NYCDOT)


