Performance assessment of reinforced concrete bridges under the multi-hazard conditions and climate change effects

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Performance assessment of reinforced concrete bridges under the multi-hazard conditions and climate change effects

by

Ameh Fioklou

A dissertation submitted to the graduate faculty
in partial fulfillment of the requirements for the degree of

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The student author, whose presentation of the scholarship herein was approved by the program of study committee, is solely responsible for the content of this dissertation. The Graduate College will ensure this dissertation is globally accessible and will not permit alterations after a degree is conferred.

Iowa State University

Ames, Iowa

2019

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ABSTRACT

The National Bridge Inventory (NBI) database indicates that approximately 500,000 out of 600,000 bridges in the United States (U.S.) are located over rivers and streams. As such, their foundations are constantly exposed to the erosive action of the flowing water. In addition, the projected change in the river hydrologic regime raises concerns about bridges stability failures with adverse consequences such as a traffic disruption and significant socio-economic loss. Statistical evaluation of bridge failure causes suggests that flood-induced foundation scour is the leading cause of failure in the U.S. and around the world. Moreover, as regionally distributed infrastructure systems, bridges are exposed to natural hazards such as earthquakes. Understanding how the presence of flood-induced scour affects the dynamic performance of bridge structures will lead to their resilient design. Furthermore, evidence on the climate change suggests that transportation infrastructure will be forced to operate in conditions out of their design range as flood events exceeding current 100- and 500-year design floods will become frequent. Recent disruptions in the transportation network due to flooding events highlight the importance of considering the increased likelihood of extreme flood events in design and hazard mitigation.

The objective of this dissertation is to assess the performance of highway bridges considering extreme flood-induced scour in the context of multi-hazard scenarios, then develop cost-effective adaptation strategies to mitigate the impact of climate change on bridges. To achieve these goals, a vulnerability assessment was conducted through a probabilistic-based approach by utilizing fragility functions. Furthermore, a framework is proposed to assist transportation decision-makers face with large uncertainties imposed by climate change. This framework is based on scenario planning and provides decision makers with guidelines for economic analysis of the impact of climate change on bridges.
CHAPTER 1. GENERAL INTRODUCTION

1.1 Background

Bridges are important components of the transportation systems. During their service life, they are subjected to natural and man-made hazards such as collisions, wind, floods, waves and earthquakes which cause them to fail. Understanding how these hazards affect the performance of bridges will lead to an improved design, help develop better mitigation plans and ultimately benefit the society. Among the natural hazards, hydraulic events are the leading cause of bridge failures in the United States and around the world. Flood-induced scour is one of the major components of hydraulic-related failures. The failure of I-90 Thruway in New York State in 1987 marked the turning point for Federal and State transportation agencies to develop a better understanding of the scour mechanism and to improve underwater bridge inspection. Moreover, the need for more research to understand the scour phenomenon was emphasized. Since then, significant improvements have been made in this regard by developing mathematical models to predict bridge scour depth with limited errors. The need to consider scour-related failures in bridge design and assessment is underlined by the evidence of the increased frequency and intensity of the precipitation events. This is in addition to the exposure to other types of natural hazards such as earthquakes in seismic prone regions that highlight the importance of a multiple-hazard approach for design and assessment of the bridges over waterways.

The National Bridge Inventory (NBI) database shows that there are approximately 600,000 bridges in the United States (U.S.) among which, 500,000 have their foundations in rivers and streams. Further investigations revealed that about 26,000 of the bridges crossing rivers are classified as scour critical while 100,000 of them have unknown foundations. As vital components of the transportation network, highway bridges failure can cause traffic disruption and have adverse social and economic implications. The inventory of bridge failure causes range from
collision, flood, overload, wind, earthquake, fire, aging and deterioration processes, design error, lack of maintenance, and construction deficiency (Lee et al 2013).

Previous studies have reported that hydraulic failures represent the number one cause of bridge failures observed in the U.S. (Lee et al. 2013, Briaud 2006a, Wardhana and Hadipriono 2003). Furthermore, past earthquakes (Loma Prieta and Northridge) and recent earthquakes (Haiti, Japan and Chili) events demonstrate the vulnerability of bridges to seismic activities. A survey of the scientific literature shows that most of the studies looked at these extreme events separately. Only a few studies (Guo 2014, Alipour et al. 2013, Banerjee and Prasad 2011, and Ghosn et al. 2003) are available that address the reliability performance of bridges under this multi-hazard condition.

Moreover, the AECOM (2013) report on the impact of climate change and population growth indicated that on average the 1% annual exceedance probability (AEP) of flood in riverine and coastal areas is expected to increase by approximately 40 to 45% by the year 2100. Additionally, all climate projections in the U.S. are showing, with higher confidence, an increase in heavy precipitation and duration of extreme precipitation events with the Midwest and the Northeast forecasted to experience the largest increase over the last half century (Melillo et al., 2014; Frumhoff et al., 2007). Evidence on the climate change suggests that major floods are on the rise and will continue in the future (Khelifa et al. 2013). This increase in frequency and magnitude of the potential flood events increases the concerns regarding bridge failures due to extreme flooding.

1.2 Problem Statement
The increase in frequency and magnitude of flood events and the vulnerability of bridges to seismic activities as highlighted by recent earthquake activities raise the concerns about bridge failures due
to combined action of extreme flood and earthquakes. As can be seen in Figure 1.1, an overlapping of the natural hazard maps of the United States shows that areas with high risk of earthquake are also prone to high flood events. The northeast regions are at risk of moderate earthquake, hurricane and tornado. The center of the country is exposed to the risk of tornado and floods with an area in the New Madrid failure zone with high exposure to earthquakes and flood. Additionally, a large population of bridges in the United States are in moderate to high seismic and flood prone regions.

Although the probability of concurrence of these two events is very low, its occurrence can be detrimental. The state of Washington has experienced this multi-hazard effect recently where a major flood was followed by an earthquake of magnitude 4.5. The erosion of riverbed material induced by flowing water, known as scour, caused the change in bridge foundation boundary condition, thus, its stiffness. Adverse implication may be expected on the bridge performance under the expected earthquake or the probable earthquake.

The present research evaluates the combined action of floods and earthquakes on bridge performance. The study is unique, as it will consider both the substructure and superstructure responses while accounting for the shifting of demand from one component to the other components of the bridge as the scour is taking place in the foundation. The proposed probabilistic approach accounts for the effect of flooding on the seismic performance of the bridges.

Furthermore, with the effect of climate change on the rise, the frequency and intensity of heavy precipitations are projected to increase. This will change the hydrologic regime of rivers and streams, increasing the risk of scouring on the bridges that are spanning over these waterways. Because bridges are expected to remain in service even after their service life for decades, their vulnerabilities to climate change need to be evaluated so that timely actions may be developed.
The aim of this research is to evaluate the seismic performance of Reinforced Concrete Bridges using extreme flood-induced scour at the pier foundations. Scouring will result in the alteration of the boundary conditions of the bridge and can affect its seismic performance. The vulnerability of the bridge to multiple hazard events is evaluated through analytical fragility functions. Fragility curves are developed to quantify the failure probabilities of the bridges under the effects of scour and earthquakes. Furthermore, a system level performance evaluation as well as components level performance assessment were conducted to determine the controlling component under multi-hazard scenarios.

Recent disruptions in the transportation network due to flooding events highlights what will become frequent in the future. As such, it is important to develop adaption measures for transportation infrastructure to alleviate the impact of climate change on bridges. Furthermore, due to the interconnectivity of land transportation modes, disturbance and broken links may have impacts that extend beyond the transportation system. As such, this dissertation contributes to the assessment and development of a resilient method of evaluation of transportation networks prone to similar hazard conditions and provides a framework for adapting transportation infrastructures to threats posed by climate change.

1.3 Organization of the Study

This document is divided into two sections. The first section presents the performance of a three-span reinforced concrete highway bridge under multi-hazard scenarios. The second section discusses various approaches of dealing with the impacts of climate change on the transportation sector and introduces a framework for coping with the level of uncertainty in climate change.

Chapter 1 presents the background to this research, the motivation, and defines the problem statement.
Chapter 2 recognizes that the mathematical formula to estimate scour has uncertainties that are inherent to the model and parameters involved in the scour equations. A probabilistic approach is used to estimate the scour depth at bridge foundations. Nonlinear analyses are performed to investigate the performance of a bridge under multi-hazard conditions. The damageability of the bridge under a ground motion intensity was evaluated through fragility analyses at component and system levels.

Chapter 3 presents a sensitivity analysis to evaluate the unintended consequences of assuming uniform scour depths at the upstream and downstream pier foundations.

Chapter 4 introduces the impact of climate change on the hydrologic regime of rivers and streams and the transportation sector. An adaptation framework is presented based on the Predict-Then-Act approach of dealing with uncertainty in climate change.

Chapter 5 presents a robust approach for dealing with uncertainty in climate change to help transportation stakeholders with the financial constraints they are faced with in deciding to adapt their infrastructure to the more severe stressors.

Chapter 6 summarizes the present study and conclusions are drawn based on the findings. The research significance and recommendations for further studies on the combined effects of flood-induced scour and earthquakes design of bridges are presented in conjunction with the adaptation strategy of transportation infrastructure to climate change.
Figure 1.1: USA natural hazards map (NOAA.ORG)
CHAPTER 2. PROBABILITY OF FAILURE ESTIMATION FOR HIGHWAY BRIDGES UNDER COMBINED EFFECTS OF UNCORRELATED EXTREME EVENTS

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Ameh FIOKLOU and Alice ALIPOUR

Abstract

The majority of the highly populated regions in the United States are susceptible to multiple natural hazards. In such regions, the design and construction of structures under multiple hazards are critical to achieve the appropriate structural performance. Multi-hazard reliability analysis of structural systems evaluates the system response under multiple random loads, some of which may occur simultaneously, or the effect of one may weaken the system before the occurrence of the succeeding event. The current paper studies the combined effects of scouring and earthquakes, as two uncorrelated extreme events, on the performance of reinforced concrete highway bridges. In a continuous effort to support future improvement in understanding the impact of multi-hazard loading scenario on bridges and to develop mitigation actions, this paper assesses the seismic vulnerability of a reinforced concrete highway bridge experiencing the effect of erosion due to the increase in frequency of flood events. The analytical fragility approach uses a three-dimensional nonlinear finite element model of the bridge cases with various levels of scouring. Because a bridge is system of components, a component level fragility curve is used to track the response of the components for a given ground motion intensity. The system fragility curves are developed to consider the vulnerability of critical components to assess the probability of bridge damage. The results indicate that under multi-hazard scenarios, the component governing the fragility of the bridge system varies depending on the level of scour sustained by the structure.
2.1 Introduction

AASHTO Load and Resistance Factor Design (LRFD) specification is the basis for the design of bridges in the United States with a typical design service life of 75 years. In such designs, the load and the resistance are assumed to follow a normal distribution and a lognormal distribution, respectively. A target reliability index of 3.5 is considered to calibrate the safety of the bridges at Strength I limit state. Although reliability-based approaches are used for calibration at the strength limit states, the load and resistance factors for other load combinations such as those involving extreme event combinations are based on engineering judgement. This makes it difficult for proper consideration of extreme events in a consistent fashion. Most of the previous research have been focused on studying the risk and reliability of bridges under individual extreme events such as earthquakes, wind effects, floods and scour effects, storm surges, and collisions (Alipour et al. 2013, Gomez and Alipour 2014). Recent events have shown that bridges undergo series of extreme events during their life cycle. These events may be correlated or uncorrelated in space and time. Post-disaster investigations of buildings, bridges, and other infrastructures damaged during the most recent natural hazards indicate that the majority of losses have occurred as a response to more than one extreme event. Significant storm surges following wind effects in Hurricane Katrina (2005) and Hurricane Sandy (2012) or tsunami effects after seismic excitations in the Indian Ocean Earthquake (2004) and Tohoku Earthquake (2011) are examples of correlated extreme events spaced in time that resulted in significant human and financial losses. The level of structural damage observed depends on whether the structures are still recovering from the first event when the second or third event occur, with short delays in time. Although rare in nature, these historical events highlight the importance of a multi-hazard approach for designing structures.
Considering the fact that in many built environments of the U.S. are exposed to more than one source of hazards there is an immediate need to reassess the design criteria that have been traditionally developed based on a single-hazard approach. The concept of multi-hazard design has received much attention in recent years (e.g., Alipour et al. 2010, Crosti et al. 2011, Li et al. 2012, Zaghi et al. 2016, Gidaris et al. 2017, and Bruneau et al. 2017). The Federal Highway Administration (FHWA 2011) has emphasized the importance of incorporating multi-hazard resistance into future design of infrastructure systems. However, the challenge facing engineers is to evaluate the impact a multi-hazard event will have on infrastructures and to design them accordingly (Arumala 2012 and Alipour 2016).

More than 600,000 bridges exist in the U.S. and about 1,000 of them failed between 1980 and 2008 with flood and scour accounting for 47% of all failures (Lee et al. 2013). An estimated 60,000 miles of coastal highways are exposed to periodic flooding as results of climate change. Additionally, in regions prone to seismic activities, structures are expected to undergo seismic events during their service life. Current bridge design codes recommend the evaluation of each extreme event independently. However, bridges that have experienced changes in the boundary conditions due to scour would have their dynamic characteristic altered. Consequently, their seismic performance could deviate from that of the original structure. The topic of multiple-hazard assessment and design has been the focus of several recent studies (Gidaris et al. 2017, Bruneau et al. 2017, Echeverria et al. 2016). A notable number of studies have focused on the performance of the columns as representatives of the bridge performance under multiple hazards (Billah and Alam 2014, Akbari 2012, Gardoni and Rosowsky 2011). However, other studies have shown that a system-level approach that accounts for the performance of the bridge as a structural system is more suitable. For instance, Alipour et al. (2012) developed the fragility surfaces under scour and
earthquakes for a suite of eighteen case study bridges. Their framework led to the calibration of scour load-modification factors for the design of bridges located in high seismic areas. Wang et al. (2014a and b) developed a risk-based design approach to study the impact of earthquakes and scour on bridges. Liang and Lee (2013) and Lee et al. (2015) used the concept of capacity reduction for scour and developed an equivalent load effect on bridges to represent scour effects. Banerjee and Prasad (2013) used region specific flood and seismicity to assess the vulnerability of bridges to flood events. Their numerical analyses indicate higher seismic damageability due the presence of scour at the foundation. Guo et al. (2016) developed time dependent fragility estimates of a case study bridge with scour to assess the lifetime probability of failure under scour and earthquakes. Yilmaz et al. (2017) used a sensitivity analysis to find the most important parameters that impact bridge vulnerability to scour and earthquakes.

In this study, a reliability-based approach is presented to estimate the likelihood of the occurrence of multiple events, the associated probability of failure, and the extent of losses to the structures. There are limited survey data of structures damaged due to correlated extreme events in the literature (Zuccaro et al. 2008) and even sparse reports on structural damages caused by the interaction of several natural events with different origins. In this study, the procedure for estimating the losses of structures under multiple events with distinct sources will be presented. Since the likelihood of occurrence of two or more simultaneous events of independent sources (such as earthquake and scour or earthquake and heavy live loads) is very rare, the process of damage generated from these different sources could be considered independent. This is a reasonable approach for large, infrequent events which have a very low probability of occurrence, but still applicable for frequent events with short durations.
The proposed methodology is applied to case study bridges that are located over a major river in a seismically prone region. To analyze and design for the impact of multiple hazards, the extreme events are characterized by identifying their sources. In this case, a fault rupture can cause an earthquake and hydrological processes can result in floods. Depending on the location of an earthquake epicenter, the soil type, and the type of fault rupture, a seismic event could affect a large region. Similarly, flood events can also affect a large region and the extent of their damages is highly correlated to the proximity of the river, the coastline, and the land topography. In this study, the focus will be on riverine floods. The exceedance probability of flood events will be measured through statistical analysis of the historical data. The probability of occurrence of earthquake events will be estimated using the available earthquake hazard maps through USGS. A quantitative intensity measure that correlates with the possible impact on bridges will be used to represent the seismic event intensity.

The vulnerability of bridges to individual extreme events will be measured through the statistical analysis of the bridge components. For this purpose, fragility functions will be generated to analyze the probability of failure of the bridge under different intensity levels of earthquake and scour. Here, one major point to consider is that floods affect the bridge by changing its boundary conditions (Wang et al. 2016). The reason is that when flood events occur, they can result in the erosion of soil around the foundation of the bridge, which in turn will change the dynamic characteristics of the structure (Fioklou and Alipour 2015). The analysis results suggest that a comparative evaluation of components and system-level performance is needed to properly quantify the reliability of highway bridge infrastructures under the combined effects of scour and earthquakes.
2.2 Multiple Hazard Analysis

High storm surges in the gulf coast regions and possible tsunami, landslides, or liquefaction in seismic regions are correlated in time and space with the main natural hazards such as hurricanes and earthquakes. The effect of these natural hazards can be additive in term of structural response that result in demands exceeding the design values. While some natural hazards are correlated in space and time others are not. Uncorrelated natural hazards include loading scenarios on structures such as the combinations of earthquake, scour, wind, collision, and many more. Although, scarce in nature such occurrence could be detrimental to the performance of a transportation network.

Equation 2.1 provides a basis for estimating the probability of a structural failure (or exceeding a specific state of damage) under a series of extreme events. This formulation depends on the mean occurrence rates of individual extreme events, the probability of failure of a structure under x extreme events at time \( t \). This will be an important contribution to the current design and assessment procedures, which are mostly deterministic and based on single-hazard approaches.

\[
P_f(t) = 1 - \exp\left(-\left[\sum_i \kappa_i p_i + \sum_i \sum_j \kappa_{ij} p_{ij} + \sum_i \sum_j \sum_k \kappa_{ijk} p_{ijk}\right]t\right)
\]

in which:

\[
\kappa_i = \lambda_i - \lambda_{ij} - \lambda_{ik} + \lambda_{ijk}
\]

\[
\kappa_{ij} = \lambda_{ij} - \lambda_{ijk}
\]

\[
\lambda_{ij} = \lambda_i \lambda_j (\mu_{d_i} + \mu_{d_j})
\]

\[
\kappa_{ijk} = \lambda_{ijk} = \lambda_i \lambda_j \lambda_k (\mu_{d_i} \mu_{d_j} + \mu_{d_j} \mu_{d_k} + \mu_{d_i} \mu_{d_k})
\]

where \( p_i \), \( p_{ij} \), and \( p_{ijk} \) are the conditional probability of failure given the structure is subjected to the \( i \)-th extreme event only, \( i \)-th and \( j \)-th extreme events, and \( i \)-th, \( j \)-th, and \( k \)-th extreme events,
respectively. In Equations 2.1-2.5, \( \lambda_i \) is the mean occurrence rate of the \( i \)-th extreme event; \( \lambda_{ij} \) is the mean coincidence rate of \( i \)-th and \( j \)-th extreme events; and \( \lambda_{ijk} \) is the mean coincidence rate of \( i \)-th, \( j \)-th, and \( k \)-th extreme events. Furthermore, \( \mu_{di} \), \( \mu_{dj} \), and \( \mu_{dk} \) represent the mean duration of each of the \( i \)-th, \( j \)-th, and \( k \)-th extreme events, respectively.

This procedure can be employed for one structure or a series of representative structures to determine the vulnerability of a group of similar structural systems. However, prior to implementing Equation 2.1, two major tasks must be accomplished: i) characterization of the involved extreme events, in terms of expected intensity, duration, and occurrence rate, and ii) assessment of the structural response under single and multiple extreme events. In this study, the focus will be on assessing the probability of failure given the structure is subjected to two events, scour and earthquakes. The assessment of the structural response was based on a component-level model that is capable of re-distribution of the demands. Given the wealth of knowledge on the stochastic characterization of extreme events, temporal and spatial variability of the major influential parameters are incorporated into the structural simulations, in addition to the uncertainties associated with structural properties.

2.3 Hazard Characterization

Earthquakes and flood-induced scour are two uncorrelated extreme events. Each of these events will have a different impact on the bridge structure and therefore, are considered in this paper for hazard characterization. There are well-accepted probabilistic methods in the literature to model both earthquake and flood events, their occurrence frequency, and intensity. The following sections provide a brief summary for characterizing each event as it relates to the probabilistic approach for performance assessment of bridges under multiple hazards.
2.3.1 Seismic hazard

A relationship between the peak demands in the structure and the ground motion intensity measure, IM, needs to be established when a probabilistic method is sought to assess the seismic performance of bridges. Different intensity measures such as peak ground acceleration (PGA), peak ground velocity (PGV), spectral acceleration ($S_a$), spectral displacement ($S_d$), and spectral intensity (SI) have been extensively used in the literature as metrics to describe ground motions. Shinozuka et al. (2001) related the damage data observed in the 1994 Northridge and 1995 Kobe earthquakes to PGA. Padgett et al. (2008) examined ten different IMs for seismic demand evaluation to conclude that PGA is the best IM for bridge portfolios. Nielson and DesRoches (2006) studied the effect of using PGA versus $S_a$ for seismic demand of highway bridges to conclude that PGA is the best IM when dealing with different bridge classes. Zelaschi et al. (2014) conducted a comparative study on twenty-three commonly used IMs in the literature and selected PGA, PGV, and $S_a$ as optimal IMs. However, they concluded that the final choice of appropriate IM for analysis would depend on personal judgment and availability of data.

As the review of the literature suggested, there is not a definitive IM measure for bridge seismic analysis. The available ground motion models or intensity-attenuation relationships estimate with high confidence PGA and is considered as IM in this study. A suite of 60 ground motions generated by FEMA-SAC07 project are used and are grouped into three sets of equal size each having a return period of 72, 475, and 2500 years, corresponding to an exceedance probability of 50%, 10%, and 2% in 50 years, respectively. In addition to covering a large range of the PGA ($0.109 \text{ m/s}^2$ to $1.304 \text{ m/s}^2$), the frequency content of the selected ground motions is such that they are capable of exciting different modes of the bridge structure.
2.3.2 Flood hazard

The case study bridge is located over Sacramento River, which carries 31% of the total surface runoff of the state of California. The annual discharges were collected from USGS database for the gauge station situated under the Bend Bridge near Red Bluff, CA. The peak annual discharge records for the period of 1950 to 2014 was extracted from the USGS website. Figure 2.1 shows the time history of the annual peak discharge. The flow data were used to find the best-fit probability distribution at this site. Because the sample size is less than 100 data points, Kolmogorov-Smirnov (K-S) sample test was selected to perform the goodness-of-fit test on each probability distribution function to verify its validity. This test evaluates the maximum absolute difference between the cumulative distribution of the peak streamflow data and the probability distribution function considered in the null hypothesis. The K-S value is compared with a critical value, $\alpha$, which is a function of the level of significance chosen and the size of the sample. For this analysis, 5% level of significance or 95% confidence level is assumed, which is an acceptable value for civil engineering applications. The comparative results of the K-S tests indicate that Pearson Type III distribution is the best fit for the discharge.

2.3.3 Scour process and probabilistic estimation of scour depth

Bridge scour is the erosion of soil sediment particles around pier or abutment foundation due to flowing water. The extent of scour depth depends on the characteristics of the streambed soil material, flood depth, and the velocity of flow, with the latter two parameters related to the discharge (flood hazard). Three major types of scour have been identified: contraction scour, general scour, and local scour. Local scour at bridge piers and abutments results from the formation of horseshoe vortices and the acceleration of flow as it bends around these obstructions. Local scour is known to have the most adverse effects on the performance of the bridges. Substantial efforts have been made to quantify the maximum attainable scour depth through numerical
modeling and laboratory-based experiments. In the U.S., the most commonly used equation for local scour depth evaluation in cohesionless sediments was developed at the Colorado State University (Equation 2.6). Recently, Briaud et al. (2011) have developed an equation for scour depth evaluation in cohesive soil (Equation 2.7). These equations are recommended by the FHWA (HEC-18 2012) for local scour depth estimation.

\[
\frac{y_s}{a} = 2.0k_1k_2k_3k_4 \left(\frac{y_1}{a}\right)^{0.35} Fr_1^{0.43} \tag{2.6}
\]

\[
y_s = 2.2k_1k_2a^{0.65} \left(\frac{2.6V_1-V_c}{\sqrt{g}}\right)^{0.7} \tag{2.7}
\]

where \(y_s\) is the scour depth (m), \(a\) is the width of the column (m), \(Fr_1\) is the Froude number directly upstream of the pier, \(V_1\) is the mean velocity directly upstream of the pier, \(V_c\) is the critical velocity at which the erosion of cohesive material is initiated, \(g\) is the acceleration due to gravity, \(k_1, k_2, k_3, \) and \(k_4\) are correction factors for pier nose shape, angle of attack of the flow, dune factor, and correction factor for size of bed material, respectively. Because the above equations 2.6 and 2.7 were developed for a single pile, when used for pile group an effective pile width, \(a_{eff}\) (Equation 2.8), is used instead of the pile diameter (HEC-18, 2012).

\[
a_{eff} = a_{proj}k_sk_n \tag{2.8}
\]

where \(a_{proj}\) is the sum of the non-overlapping projected width of the piles onto a plane normal to the flow, and \(k_s\) and \(k_n\) are coefficients accounting for pile spacing and the number of aligned pile rows, respectively.

In cohesive streambeds, the rate of sediment removal increases slowly once the critical shear stress is reached. Note that the critical shear stress is the stress at which the removal of soil particles is initiated. As such, the maximum attainable scour depth (Equations 2.7) may not be
reached over the lifetime of the bridge. Thus, Briaud et al. (2008) suggested a time-dependent formulation of the scour depth evolution as follows:

\[
y_s(t) = \frac{t}{\frac{1}{z_i} + y_s} \tag{2.9}
\]

where \( z_i \) is the initial erosion rate in m/hr, \( t \) is duration of the flood in hour, and \( y_s \) is the scour depth calculated using Equation 2.7. For this study, a flood duration of 48 hours was assumed. Using the computed shear stress and assuming a medium erodibility of the material, it was determined that the average initial erosion rate is 0.1 m/hr.

According to AASHTO LRFD requirements, bridges should be designed and checked for a 100 and 500-year flood events, respectively. For these levels of floods, the flow depth at the upstream, \( y_1 \), in Equation 2.6 is unknown a priori. Therefore, they are estimated by solving Manning’s equation for open channel flow as follows:

\[
Q = \frac{A}{n} R_h^{2/3} S^{1/2} \tag{2.10}
\]

\[
R_h = \frac{A}{P} \tag{2.11}
\]

where \( A \) is the cross-sectional area of the flowing fluid; \( n \) is Manning’s roughness number; \( S \) is the slope energy; and is assumed uniform across the river, \( P \) is the wetted perimeter.

To account for the possibility of variation in Manning’s roughness number across the riverbank and the irregularity of the section, the channel flow at the bridge crossing can be evaluated by discretizing the flow area. Adopting this approach, Equation 2.10 is updated as follows:

\[
Q = \frac{A_1}{n_1} R_{h1}^{2/3} S^{1/2} + \frac{A_2}{n_2} R_{h2}^{2/3} S^{1/2} + \frac{A_3}{n_3} R_{h3}^{2/3} S^{1/2} + \cdots + \frac{A_n}{n_n} R_{hn}^{2/3} S^{1/2} \tag{2.12}
\]
The major factor affecting the scour depth is the peak annual discharge rate, $Q$. Considering the uncertainty associated with the different parameters of the design scour depth, a probabilistic approach has been adopted in this study. A Latin hypercube sampling with $10^6$ samples was performed to simulate the flow depth and the scour depth at the location of the bridge. Furthermore, the cross-sectional effects on the scour depth were investigated by considering three channel shapes: rectangular, triangular and trapezoidal. The flow is considered uniform at the upstream and Manning’s roughness number, $n$, is assumed identical for all sections. Two-to-one slope (2H:1V) is adopted for the triangular and the trapezoidal sections.

The scour risk curves for the three channel shapes in cohesive and cohesionless soils are shown in Figure 2.2. The effect of the river channel cross section shape is clearly shown in this figure. For the same discharge, the probability of occurrence of 5.0 m scour depth in cohesionless soil is 25% for the rectangular shape and 50% for the trapezoidal and triangular shapes. Considering the natural topography at the bridge site, a trapezoidal channel shape scour depth analyses results are used in the bridge vulnerability assessment.

2.4 Bridge Vulnerability Analysis

The occurrence of an earthquake can compromise the functionality or even cause bridge failures. A fragility function, which expresses the relation between the intensity of an event and the quantitative measure of its probable damage to the structure, can be used to assess the seismic vulnerability of the bridge structure. Shinozuka et al. (2000) identified four methods for developing fragility functions: professional judgment; quasi-static and design code consistent analysis; use of damage data from past earthquakes; and numerical simulation of seismic response of structures based on dynamic analysis. This paper will generate fragility functions by conducting nonlinear
time history analyses and relating different engineering response parameters to the pre-defined damage states.

Equation 2.13 is used to define the fragility function and a maximum likelihood approach was adopted to estimate the parameters. Two methods could be used to form the likelihood functions, as shown in Equation 2.14. The difference between the two methods lies in the estimation of the log-standard deviation for the damage states. In Method 1, the log-standard deviation for each damage state is estimated independently, whereas Method 2 assumes the same log-standard deviation for all damage states. For the two methods, the best parameters are obtained by solving the Jacobian of the natural log of the likelihood function, \( L \), (Equation 2.15).

\[
F_j(IM, S_c) = \Phi \left[ \frac{\ln \left( \frac{IM}{\zeta_j} \right)}{\zeta_j} \right] \quad (2.13)
\]

Method 1: \( L = \prod_{i=1}^{k} \left[ F(IM)^{x_i} [1 - F(IM)]^{1-x_i} \right] \quad (2.14) \)

Method 2: \( L = \prod_{i=1}^{k} \prod_{j=1}^{n} \left[ P_j(IM_i; E_k)^{x_i} \right] \)

\[
\frac{\partial \ln L}{\partial v_j} = 0 \quad (2.15)
\]

where \( F_j \) is the fragility function for the \( j^{th} \) damage state as a function of the ground motion intensity measure, \( IM \), and the scour depth, \( S_c \); \( \zeta_j \) and \( \zeta_j \) are the median and log-standard deviation of the fragility function; \( \Phi(*) \) is the cumulative normal distribution function; \( E_j \) is the damage state; \( P_j \), is the probability that a bridge will suffer a damage \( j \) under the ground motion \( i \); \( v_j \) represents the variables \( c_j \) and \( \zeta_j \) and \( x_i = 1 \) if bridge sustains damage state \( j \) under a ground motion \( i \) and \( x_i = 0 \) otherwise. Method 2 is used in this paper, and the damage states are assumed not independent of each other for a specified PGA.
Previous research on seismic performance of bridge system were based on the column fragility. The added flexibility in the system, brought by the scouring of soil sediments around the bridge foundation, has led some to believe that scour improves bridges seismic performance. However, the presence of scour hole at the foundation results in the change in the dynamic characteristics of the system and the seismic demands in the critical sections of the column decrease (Klinga and Alipour 2015, Fioklou and Alipour 2014). As such, assuming the column as critical component in the presence of scour will result in overestimating the seismic performance of the bridges. For a bridge with scoured foundation, the fragility functions need to be developed for any updated state of the system after flood-induced scour events. Furthermore, a review of the literature indicates that columns, piles, bearings, and abutments have the largest impact on the seismic performance of the bridges. This paper provides a procedure for developing a multi-component fragility function that considers the performance of the bridge as the collective performance of its individual structural components.

### 2.5 Damage States for Bridge Components

The impacts of bridge failure or damage on a transportation network have direct and indirect societal costs associated with it. To generate the fragility functions, a clear definition of damage states is needed. Fragility functions have been developed for four different levels of damage: “at least minor”, “at least moderate”, “at least major damage” and “collapse”. Accordingly, four different damage states will be considered for each component. For each damage state, the limits are specified for each component considered in the analysis. These limits generally follow from the observations made in laboratory experiments, field data, and engineering judgement. Table 2.1 lists the summary of the limit states in the literature and those adopted in this
study. The following subsections define the background of the different damage states for the various components of the bridge system.

### 2.5.1 Columns

Different engineering demand parameters such as the displacement ductility ratio, curvature ductility ratio, section rotation, and drift ratio among others have been used to characterize the failure of reinforced concrete columns. Many researchers have assessed the seismic performance of bridges using the column section properties and column top displacement (Deepu et al. 2014, Torbol et al. 2012, Banerjee and Shinozuka 2007, Nielson et al. 2007, Choi et al. 2004). However, the aforementioned metrics only measure the capacity of the cross section and do not take into account the overall response of the column or the structure. In no-scour condition, the use of displacement ductility ratio could be justifiable. However, the erosion of the soil results in a large portion of the displacements occurring in the piles due to increase in the foundation flexibility (Kling and Alipour 2015). As such, using the displacement ductility ratio as performance measure will overestimate the failure probability of the columns on scoured foundations. Consequently, the drift ratio is an appropriate measure for the damage state of the columns because i) it provides a holistic overview of the response of the structure, and ii) by estimating the drift, the response of the column is isolated from that of the piles, which is affected by the presence of scour. A yielding drift ratio of 0.005 suggested by Priestley et al. (1996) is used to define the “at least minor” damage state of the column. The other three damage states correspond to those suggested by Dutta and Mander (1998).

### 2.5.2 Abutments

The reconnaissance reports on abutment failures indicate that abutment backfill material plays a significant role in resisting seismic loads. The inertia force required to mobilize the backfill
is generally larger than a practical sized backwall can be designed to resist (Caltrans Memo 1992). Consequently, to protect the abutment foundation from the inelastic action that might develop during a strong motion, the abutment backwalls and shear keys are designed as fuse elements (SDC 2013 and Seible and Silva 2001). The drawback of this type of design is that excessive displacement of the deck will result in the unseating of the deck, thus the collapse of the end span.

The most common abutment damage during an earthquake is due to the unseating of the superstructure. Unseating of the deck at the abutments may result from earthquake-induced displacement greater than the seat width. Kwon and Elnashai (2010) evaluated the abutment vulnerability using three limit states: serviceability, damage control, and collapse prevention. Ledezma and Bray (2008) proposed a simplified approach for evaluating the consequences of the seismically induced lateral displacement of the abutment by identifying five damage states including: negligible, small, moderate, large, and collapse.

The case study bridges in this paper have seat-type abutments and the design of the shear keys are assumed adequate for the expected seismic events. Therefore, failure in the transverse direction is not expected to occur. The longitudinal movement of the deck away from or toward the backwall is used to define the abutment damage states. The minor damage state corresponds to 70.0 mm displacement of the deck toward the abutment backwall. The collapse damage state corresponds to the unseating of the deck at either abutment. This corresponds to the deck relative displacement at the abutment level that is greater than the seat width. Caltrans SDC (2013) recommends a minimum of 760 mm (30 in) for seat width and this value is used to define the abutment collapse damage state. Closure of the gap will not affect the functionality of the bridge but will require some cleaning. From the gap closure until the full passive pressure of the backfill is reached, spalling and possible damage of backwall might occur and will require repair. As such,
moderate damage state is defined for 300 mm displacement of the backwall. From this point, any residual displacement will result in a permanent deformation of the backwall or the embankment. The relative displacement resulting from the backfill reaching its capacity to the onset of the unseating of the deck is used to define the major damage state.

2.5.3 Bearings

Elastomeric bearings are used to resist the lateral loads and to accommodate for deformations. Their ability to undergo reversible deformation makes them attractive for seismic isolation in bridges compared to other traditional bearings. However, their failure modes are often the result of large shear strains caused mainly by cyclic axial forces (Stanton et al., 2008). During a seismic event, bearings are subjected to simultaneous vertical and lateral displacement. Furthermore, the level of seismic excitation may induce large shear strain in the bearings that can result in instability failure. Caltrans-SDC (2013) recommends a maximum shear strain of +/- 150% in the bearings as a limit for instability failure. Sanchez et al. (2013) and Han et al. (2013) experimental tests results show that instability failure of a bearing may occur for shear strain exceeding 150%. In this study, the bearings’ four damage states limits followed those defined in Zhang and Huo 2009.

2.5.4 Pile foundations

Statistical evaluation of bridge failure data indicates that compared to the superstructure, there are few instances where damages to the pile foundations were reported. This could be related to the general design approach that ensure elastic response from the piles. In cases where the demands exceeded the elastic response range of the piles, damages were found to be located at the connection between piles and the cap; along the piles at some depth below ground surface; at the
bottom of the liquefiable soil layer; and at the interface of two soil layers with significant relative stiffness ratios (Tang and Ling 2014, Abdoun and Dobry 2002).

During a seismic event, a pile foundation will be subjected to lateral load and its performance will depend on the pile characteristics as well as the soil-structure-interaction. The lateral displacement of the pile and soil may result in the formation of plastic hinges in the pile. The location of the plastic hinge is found to vary with the lateral soil stiffness and the above ground pile height in the case of drilled shafts (Bucdek et al. 2000). The experimental study conducted by Tang and Ling (2014) indicates that failure mechanism can develop at locations where the seismically induced bending moment is greater than the pile plastic bending moment. Liu et al. (2001) conducted an experimental study in which collapse damage state was defined as the horizontal displacement equal to 2% of the column clear height. Broms (1964) proposed a methodology for estimating the allowable horizontal displacement for a pile embedded in a uniform soil. Moulton et al. (1985) surveyed 314 bridges for tolerable bridge movement to conclude that, in general, a horizontal movement of 1.5 and 3.5 inches of the pile head are considered tolerable and intolerable, respectively. Ledezma and Bray (2008) defined pile failure based on the section plastic rotation. Blandon (2007) used section analysis to evaluate the formation of pile failure mechanism. Song and Chai (2006) related the number of failure mechanisms developed in the pile to the horizontal displacement imposed on the pile head. As can be inferred from the previous studies, there is no definitive quantitative measure in the literature for defining piles damage limit states. In this study, the damage state considered is the formation of plastic hinges following recent work by Hung and Yao (2014, 2017). For this purpose, four damage states are defined for the piles and correspond to the formation of 0, 1, 2, and more than two plastic hinges along the length of the pile. It should be noted that considering the SPT values
for the soil and the depth of the sandy soil the effects of liquefaction have not been considered in this study.

2.6 Case Study Bridge

2.6.1 Description of the bridge

The bridge cases considered in this study are three three-span reinforced concrete box girders located in California and designed following the Caltrans Seismic Design Criteria. The seismic hazard curve at the location of the bridge is generated from the USGS hazard maps (Figure 2.3). The bridges have two-column bents with a circular cross section (diameter of 1.6 m), and a longitudinal reinforcement ratio of 2.5%. The difference in the bridge cases is based on the column heights: 7.5, 10.0, and 12.5 m. Each column is supported on a pile cap founded on a pile group foundation. Each pile has a cross-sectional diameter of 0.4 m and a longitudinal reinforcement ratio of 1.6%. The bridges have equal approach spans of 30.0 m and the middle span is 36.0 m. The deck has a width of 23.0 m and consists of a four-cell box girder and seat-type abutments. The soil profile at the bridge site is made of layers of cohesive and cohesionless materials and follows the case study that was introduced in Klinga and Alipour (2015). Figure 2.4 shows the elevation of the bridge and cross section at one of the bents with the details of the soil profile that supports the deep foundation.

2.6.2 Modeling procedure

The numerical analyses of the case study bridges are conducted in the OpenSees software platform. The deck and cap beam are modeled as elastic elements since they are designed to remain elastic during a seismic event. The column and pile cross sections are discretized into confined, unconfined, and steel elements. Inelastic behavior is expected in the columns; therefore, they were modeled using force-based elements. This element allows for not only the input of the plastic hinge length but also the spreading of plasticity beyond the plastic hinge regions. Pile element size is
selected to coincide with the soil discretization element size (0.5 m) and the pile elements are modeled using displacement-based beam-column elements. Detailed modeling of the columns and piles and their constitutive behavior can be found in Fioklou and Alipour (2015).

The abutments and deck of bridges located in seismically active regions are prone to damage caused by impact forces also known as pounding forces. These impact forces are the result of out-of-phase movement between various components of the bridge system. In the analytical models, pounding effect resulting from the seismic excitation is implemented by utilizing two approaches: contact elements and stereo-mechanical elements. Figure 2.5 shows the model used in the present study, which was proposed by Muthukumar (2003). It is a bilinear truss element with gap and is based on Hertz damp model. This model is capable of capturing pounding effects in the bridge and accounts for the energy dissipation due to impact. Spring elements are used to monitor the pounding force and the stiffness of the spring elements are estimated using the following equations suggested by Muthukumar (2003):

\[
K_{\text{eff}} = K_h \sqrt{\delta_m} \quad (2.16)
\]

\[
K_{t1} = K_{\text{eff}} + \frac{\Delta E}{\alpha \delta_m^2} \quad (2.17)
\]

\[
K_{t2} = K_{\text{eff}} - \frac{\Delta E}{(1-a)\delta_m^2} \quad (2.18)
\]

\[
\Delta E = \frac{K_h \delta_m^{a+1}}{u+1} (1 - e^2) \quad (2.19)
\]

where \(K_h\) is the hertz stiffness (=275x10^6 kN-m^{3/2}); \(\Delta E\) is the energy loss during impact; \(u\) is the Hertz coefficient and is taken as 1.5; \(e\) is the coefficient of restitution (=0.7); the yield displacement parameter \((a)\) is assumed to be 10% of the maximum displacement \((\delta_m)\).
The bearing devices in bridges are used to allow for the transfer of loads from superstructure to substructure. The loads transferred by the bearings include the superstructure gravity load and horizontal loads. There are several types of bearings and the selection process for a particular application depends on their load resistance and movement capabilities. Elastomeric reinforced bearings are widely used and offer the advantage to be less susceptible to corrosion and require less maintenance (Fasheyi 2012). Other advantages of elastomeric bearings are that they allow movement in all directions by elastic deformation and rotation around every direction, thereby allowing for the transfer of forces from one component to another. A typical elastomeric reinforced bearing pad consists of a number of rubber layers bonded to intermediate steel shims to produce a vertically stiff and horizontally flexible element. In the analytical model, the bearing pads are modeled using Steel01 material in the longitudinal direction. In the vertical direction, the spring are only allowed to take compressive force. The stiffness’s of the bearing pad in the longitudinal ($K_H$) and vertical ($K_V$) directions were calculated as follows (AASHTO 2012):

$$K_H = \frac{G A}{H_r}$$

(2.20)

$$K_V = \frac{E A}{H_b}$$

(2.21)

where $A$ is the elastomer gross plan area, $H_b$ is bearing height, $H_r$ is the bearing total thickness, $G$ and $E$ are the shear and Young’s moduli of the bearing, respectively.

The hyperbolic nonlinear model proposed by Shamsabadi (2007) is employed to capture the nonlinear behavior of the abutment-embankment system in the longitudinal direction. Zero-length spring elements are used to represent the load deformation of the abutment system. Each element includes a 5.0 cm gap element, which represents the expansion joint between the superstructure and the abutment. Additionally, the bearing pad spring elements are placed in parallel with the backwall backfill elements. The behavior of the abutment is that of a gap element
in series with the backfill compressive element, both in parallel with the elastic bearing pad element. In the transverse direction, a bilinear model is used to capture the behavior of the backfill. The backfill spring elements are placed in series with a gap element. The stiffness and strength of the springs were calculated using equations suggested by Caltrans SDC (2010).

The soil-pile-structure interaction is implemented in the analyses by three nonlinear springs \((p-y, t-z, q-z)\) simulating the nonlinear behavior in the longitudinal, transverse and vertical directions. Each soil layer is discretized into 0.5 m as element sizes. Detailed approach for evaluating soil spring parameters (ultimate capacity and depth at which 50% of the capacity is mobilized) for cohesionless material can be found in Fioklou and Alipour (2014). For soft and stiff clay materials, the ultimate capacity of the longitudinal and transverse springs are calculated using Equations 2.22 and 2.23, respectively (Reese and Van Impe 2011). The friction along the pile is evaluated using Equation 2.24, as suggested by Kolk and Van der Velde (1996). The depth at which 50% of the ultimate strengths are mobilized on the \(p-y\) curve, \(y_{50}\), of soft and stiff clays are estimated using Equations 2.25 and 2.26, respectively. Chen and Kulhawy (2002) showed that at a depth corresponding to 0.4% of the pile diameter, 50% of the ultimate frictional strength is mobilized in cohesive materials.

\[
P_{\text{ult}} = \min \left\{ \frac{3dS_u + \gamma'zd + JS_ud}{9dS_u} \right\}
\]

\[
P_{\text{ult}} = \min \left\{ \frac{2dS_u + \gamma'zd + 2.83S_ud}{11dS_u} \right\}
\]

\[
T_{\text{ult}} = S_u \times \begin{cases} 
0 & \text{for } 0 \leq z \leq 1.524 \\
0.55 & \text{for } \frac{S_u}{P_a} \leq 1.5 \\
0.7 - 0.1 \frac{S_u}{P_a} & \text{for } 1.5 \leq \frac{S_u}{P_a} \leq 2.5
\end{cases}
\]
\[ y_{50} = 2.5\varepsilon_{50}d \]  \hspace{1cm} (2.25)

\[ z_{50} = \varepsilon_{50}d \]  \hspace{1cm} (2.26)

where \( d \) is the pile diameter, \( S_u \) corresponds to the undrained shear strength of the soil, \( \gamma' \) is the effective soil unit weight, \( z \) is the depth at which the soil response is being evaluated, \( J \) is a factor used to represent the ultimate soil resistance near ground surface (=0.5), \( P_a \) is a factor relating the unit side resistance to the undrained shear strength of the soil, and \( \varepsilon_{50} \) is the strain at which 50% of the ultimate strength is mobilized. Detailed approach on the modeling procedure for the soil-pile structure interaction is provided in Klinga and Alipour (2015). The erosion of top soil layers around the piles due to scouring would result in loss of lateral support and the friction. To account for this phenomenon in the analytical model, the \( p-y \) and \( t-z \) springs are removed to a depth corresponding to the calculated scour depth and the soil spring stiffness’s were updated in the analysis.

### 2.7 Implementation of Multiple Hazard Analysis on the Case Study Bridge

Modal analysis was performed to determine the vibration modes of the bridge. Table 2.2 shows the first two natural periods for all three bridges conditions; while Figure 2.7 displays the first two vibration modes of the intact bridge case. It can be observed that the first fundamental vibration mode occurs in the longitudinal direction. The increase in period observed is due to the reduction in the system stiffness as a result of soil erosion around the foundations. For dynamic evaluation, the ground motions are applied in the direction that coincides with that of the first fundamental mode of vibration of the bridge: the longitudinal direction.

The suite of ground motions is used to conduct the nonlinear time history analysis on the intact bridges and bridges with different levels of scour associated with the 100- and 500-year
floods. The time histories of the various parameters used to define the relevant limit states of the critical components are recorded during the analysis and then used to generate the component-level fragility functions. These parameters include the column drift ratio, the horizontal displacement of the abutment, the shear strain in the bearings, and the history of bending moment along the length of the pile that would help estimate the number of plastic hinges forming in the piles. These data are then used to establish the relationship between the performance of the bridge components, the peak ground acceleration, and the scour depth. The following discussion pertains to the medium pier height (10.0 m) bridge case.

Figures 2.7 through 2.10 display the fragility analysis results for the column, abutment, bearing, and the pile, respectively. Figure 2.7a shows the fragility curves of the column for the bridge in its intact condition with no scour. In the intact condition, the bridge experienced only minor and moderate damages. With the development of the 100-year flood-induced scour at the pier foundation, the exceedance probability of the minor and moderate damages decreases (Figure 2.7b) to a point that with another increase in scour depth caused by 500-year flood event no damage is observed in the columns (Figure 2.7c). This observation corroborates the claim that scour improves the seismic performance of the bridge as suggested in previous studies.

Figure 2.8 depicts the fragility curves of the pile foundation, which is based on the number of failure mechanisms that develop in pile. To estimate the formation of plastic hinges and the number of its occurrence, the bending moment demand along the length of the piles (at 0.5 m increments) is captured over time and compared to the plastic moment capacity of the pile section. Whenever the moment demand exceeds the plastic moment capacity of the section marks the onset of a plastic hinge formation. The number of instances that this exceedance took place over the duration of each ground motion is used to establish the number of plastic hinges. As can be seen
from Figure 2.8a, the piles only sustain minor damage in the intact condition. Following the development of scour hole, pile head displacement increases significantly. As such, plastic hinges start to form along the pile length leading to the increase in the probability of having moderate, major, and collapse damage states in the piles.

The abutment fragility curves are shown in Figure 2.9. As can be seen from these graphs, contrary to the column fragility curves, increase in the scour depth results in the increase in the abutment exceedance probability of failure for the damage states considered under the same intensity measures. This increase in failure probability can be attributed to a redistribution of forces through the bridge elements in order to compensate for the inability of the column to dissipate more of the energy induced in the system. In addition, the number of abutment failures observed at the 500-year hazard level for minor damage is the same as that of the 100-year hazard. In the intact condition and with the 100-year flood-induced scour, collapse damage state was not observed. However, with the increase in scour depth, the added flexibility of the bridges and the larger deck level displacements resulted in the collapse damage state due to the unseating of deck at abutments.

Figure 2.10 presents the evolution of fragility curves for the bearings in intact and scoured conditions of the bridge. As can be seen, the bearings could sustain all four damage states in the intact and scoured conditions, with a possibility of a slight increase in probability of failure as the scour depth increases. The negligible increase in the probability of failure could be justified by the fact that the majority of displacements in the bridge are due to those occurring at the piles, hence decreasing the relative displacement that could cause higher strains in the bearings.

Figures 2.11 through 2.14 compare the fragility curves of all components in all damage states for the intact and scoured bridge conditions. For instance, Figure 2.11 shows that for minor
damage state case (Fig. 2.11a), the abutment and column represent the weak component of the bridge with no scour. In moderate and collapse damage states, the only component failing is the bearings. This behavior is consistent with the performance-based design philosophy where no collapse is accepted in the columns (to ensure life safety). As scour depth increases to the 500-year flood hazard condition (Figure 2.12), there is a shift in the component demand. The seismic performance of the bridge is now deviating from the standard form where the controlling component is the pier. This comparative analysis is showing that pile and abutment are competing to control the minor damage state, pile and bearing controlling major damage state, while pile is controlling the moderate and collapse damage states. As the scour depth is increasing, Figure 2.13 shows that the piles control the performance of the bridge in all damage states. This means that in such conditions, the pile is the first component of the bridge to fail.

With a component-level analysis, we can see that although the column experiences less damage, the redistribution of forces in the system results in more damage in the other elements. This was confirmed by the review of the abutments, bearings, and piles performances. The analyses further show that there is a shift in demand from one component of the bridge to another due to the change in the dynamic characteristics of the bridge caused by scour. The problem with this shifting in demand is that less demand is imposed on a component that was originally designed as critical to a component that was unforeseen to bear such demands. This component-level fragility analysis approach helps identify the weakest link in the system and pinpoint the component(s) that could create issues for the bridge that is otherwise designed to sustain targeted damage levels. Here the bridge is going to perform as good as its weakest component. Hence, the system-level fragility curves for the bridge could be estimated using the fragility curve of the controlling component in each of the damage states.
Figure 2.14 displays the fragility curve for the bridge system. To determine the parameters of the system fragility curves, the binary value for each earthquake was generated assuming that all the components worked as a system. As it was the case for the components fragility curves, the exceedance probability of failure of the system decreased as the damage intensity increased. Furthermore, as scour depth is increasing the exceedance probability is also increasing for each damage state. At a scour depth corresponding to that of the 100-year return flood, the bridge system has reached the states of minor and moderate damage conditions for nearly all ground motions used in the analysis. The same observations were valid when the bridge experienced the erosive action of the 500-year return flood. Comparing these figures with those generated for the column, one can notice the importance of multi-component fragility analysis in conditions where the dynamic characteristics of the bridge changed from its initial design. Using only the column as the basis of vulnerability analysis could result in underestimating the severity of damage to bridges with scoured foundations.

The Fragility curves in the present study were developed on the assumption that the damages follow a lognormal distribution function. As such when plotted on a lognormal probability scale, the slope of the fitted line corresponds to the inverse of log-standard deviation. Consequently, this value controls the shape of the curve. The median values determine the shifting in the curves as the damage level increases. As the median value increased, the corresponding exceedance probability of failure decreased. Figure 2.15 shows the estimated median values for all of the components and systems of the case study bridges (all three-span with different column heights) for all three hazard levels. Note that NF line in the figure indicates no failure observed. As can be seen from this figure, the system fragility curves correspond to that of the weakest link. This observation is independent of the pier height. For all bridge cases, the piers do not experience
any collapse for the suite of ground motion used and only minor and moderate damages were observed. In the intact condition, the system with short and medium pier heights did not reach a collapse damage state whereas the tall pier height bridge-system did. The system fragility curves for minor, moderate, and major damage states coincide with that of the pile for 100 and 500-years flood induced scour levels.

2.8 Conclusions

The objective of this paper is to study the vulnerability of bridges under multi-hazard conditions, which includes the effect of flood-induced scour and earthquakes. For this purpose, three bridge cases with varying pier height founded on deep pile group foundation were considered. The analytical models generated for the performance evaluation consider the soil-structure-interaction at the foundation. The failure probability expressed in the form of fragility curves was evaluated at the component and system levels. Four damage states (minor, moderate, major, and collapse) were used to relate the damageability of the bridge to the ground motion intensity, PGA.

As scour results in the change of the bridge’s dynamic characteristics, its seismic performance will differ from that of the bridge, which did not experience the erosive action of the flowing water. The analysis showed that while a particular damage state might not be of concern for one component, other components might be vulnerable under the same ground motion intensity. The observations in this paper suggest that considering only specific components might mislead the performance evaluation of the bridge system and that system level performance should be adopted under multi-hazard scenario. The evaluation of the fragility curves, under the combined action of scour and earthquakes condition indicates that contrary to what has been common practice, the column failure is not the controlling component when assessing bridge performance.
at moderate, major and collapse damage states. Since a bridge is a system of components, the overall performance of the system depends on the participation of all components. The comparative result of the system’s fragility curves and the components fragility curves for all damage states indicates that in some cases the system performs better than the individual component.

The sensitivity analysis based on the column height indicates that in its intact condition, the exceedance probability of failure of the bridge increases for all damage states with the increase in pier height at the system level. This sensitivity analysis is also reflective at the component level when the flood hazards are factored into the analysis. Some of the components have identical exceedance probability of failure under some damage states. Therefore, conducting comprehensive analyses considering all levels of damage for the various pier heights, will lead to the identification of the component(s) for which the performance is influenced by the pier height in addition to the critical component.

References


Caltrans, Bridge Memo to Designers Manual, California Department of Transportation, 1992


IABMAS. (2012). Bridge maintenance, safety, management, resilience and sustainability. Proceeding of 6th International IABMAS Conference. Stress, Italy.


Table 2.1. Components damage limit states

<table>
<thead>
<tr>
<th>Component</th>
<th>Parameter</th>
<th>Minor</th>
<th>Moderate</th>
<th>Major</th>
<th>Collapse</th>
<th>Reference</th>
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<tr>
<td></td>
<td>Curvature ductility</td>
<td>1.29</td>
<td>2.1</td>
<td>3.52</td>
<td>5.24</td>
<td>Neilson and DesRoches (2007)</td>
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<tr>
<td>Column</td>
<td>Rotational ductility</td>
<td>3.39</td>
<td>4.75</td>
<td>8.43</td>
<td>9.16</td>
<td>Torbol et al. (2013)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.58</td>
<td>3.33</td>
<td>6.24</td>
<td>9.16</td>
<td>Banerjee and Shinozuka (2011)</td>
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<tr>
<td></td>
<td>Displacement ductility ratio</td>
<td>1</td>
<td>2.25</td>
<td>2</td>
<td>4</td>
<td>7</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>2.9</td>
<td>4.6</td>
<td>7</td>
<td>5</td>
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<td>Drift ratio</td>
<td>Yield</td>
<td>0.025*</td>
<td>0.05*</td>
<td>0.075*</td>
<td>Dutta and Mander (1998)</td>
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<tr>
<td></td>
<td></td>
<td>0.005*</td>
<td></td>
<td></td>
<td></td>
<td>Priestly et al. (1996)</td>
</tr>
<tr>
<td></td>
<td>Displacement (mm)</td>
<td>100</td>
<td>7</td>
<td>500</td>
<td>2000</td>
<td>&gt;2000</td>
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<tr>
<td>Abutment</td>
<td></td>
<td>37</td>
<td>15</td>
<td>146</td>
<td>30</td>
<td>60</td>
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<tr>
<td></td>
<td></td>
<td>25</td>
<td>47</td>
<td>47</td>
<td>N/A</td>
<td>N/A</td>
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<tr>
<td></td>
<td>Residual displacement (ratio)</td>
<td>0.75</td>
<td>1</td>
<td>3</td>
<td>5</td>
<td>Bignell and LaFave (2010)</td>
</tr>
<tr>
<td></td>
<td>Unseating (mm)</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>Aygün et al. (2011)</td>
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<tr>
<td></td>
<td></td>
<td>70</td>
<td>900</td>
<td>950</td>
<td>1000</td>
<td>Bignell et al. (2010)</td>
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<tr>
<td></td>
<td></td>
<td>70*</td>
<td>300*</td>
<td>570*</td>
<td>760*</td>
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<tr>
<td></td>
<td>Bearing</td>
<td>Displacement (mm)</td>
<td>50</td>
<td>100</td>
<td>150</td>
<td>225</td>
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<td></td>
<td>Shear strain (%)</td>
<td>100</td>
<td>150</td>
<td>200</td>
<td>250</td>
<td>Zhang et al. (2009)</td>
</tr>
<tr>
<td></td>
<td>Pile foundation</td>
<td>Displacement (mm)</td>
<td>28</td>
<td>42</td>
<td>86</td>
<td>115</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Number of plastic hinges</td>
<td>0*</td>
<td>1*</td>
<td>2*</td>
<td>&gt;2*</td>
</tr>
</tbody>
</table>

*Value used in this study

Table 2.2. Period of the bridge cases for the first two vibrational modes for various scour states

<table>
<thead>
<tr>
<th>Scour depth</th>
<th>Mode 1 (sec)</th>
<th>Mode 2 (sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Column height</td>
<td></td>
</tr>
<tr>
<td>0 m</td>
<td>7.5 m 0.593</td>
<td>10.0 m 0.628</td>
</tr>
<tr>
<td>4 m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>8 m</td>
<td></td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>Scour depth</th>
<th>Mode 1 (sec)</th>
<th>Mode 2 (sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 m</td>
<td>1.117</td>
<td>1.200</td>
</tr>
<tr>
<td>4 m</td>
<td>1.704</td>
<td>1.813</td>
</tr>
<tr>
<td>8 m</td>
<td></td>
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</tr>
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</table>
Figure 2.1. Time history of the annual peak flow discharge for Sacramento River

Figure 2.2. Scour depth risk curves for rectangular, triangular, and trapezoidal river channel sections in cohesionless (Top) and cohesive (bottom) soils
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Figure 2.4. Bridge elevation (Top) and section A-A (Bottom)
**Figure 2.5.** Bilinear impact model (Muthukumar 2003)

**Figure 2.6.** First 2 modal vibrations of the intact bridge
Figure 2.7. Column fragility curves for the bridge with pier height of 10m: a) Intact, b) 100-year flood-induced scour, and c) 500-year flood-induced scour

Figure 2.8. Pile fragility curves for the bridge with pier height of 10m: a) Intact, b) 100-year flood-induced scour, and c) 500-year flood-induced scour
Figure 2.9. Abutment fragility curves for the bridge with pier height of 10.0 m: a) Intact, b) 100-year flood-induced scour, and c) 500-year flood-induced scour

Figure 2.10. Elastomeric fragility curves for the bridge with pier height of 10.0 m: a) Intact, b) 100-year flood-induced scour, and c) 500-year flood-induced scour
Figure 2.11. Comparative fragility curves for components of the bridge with pier height of 10m under the intact condition for: a) Minor, b) Moderate, c) Major, and d) Collapse
Figure 2.12. Comparative fragility curves for components of the bridge with pier height of 10.0 m under the 100-years flood hazard condition for: a) Minor, b) Moderate, c) Major, and d) Collapse
Figure 2.13. Comparative fragility curves for components of the bridge with pier height of 10m under the 500-years flood hazard condition for: a) Minor, b) Moderate, c) Major, and d) Collapse
Figure 2.14. System level fragility curves for the bridge with pier height of 10.0 m for: a) Intact, b) 100-year flood-induced scour, and c) 500-year flood-induced scour.
Note: Green: Intact; Blue: 100-year flood-induced scour; Red: 500-year flood-induced scour

Figure 2.15. Median values for components and systems fragility curves for various pier height: short (top), medium (middle), and tall (bottom)
CHAPTER 3. SIGNIFICANCE OF NON-UNIFORM SCOUR ON SEISMIC PERFORMANCE OF BRIDGES

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Ameh FIOKLOU and Alice ALIPOUR

Abstract

Multiple hazard such as flood-induced scour followed by earthquakes can adversely impact the performance of bridges spanning over water ways. Scour is the erosion of soil around the foundation of bridges, which can result in loss of the structure’s lateral support and possible failure. The impact is specifically amplified when a flood event resulting in the development of a scour hole is followed by an earthquake event. Recent inspections have shown a nonuniformity in the scour depths at bridge foundations with multiple piers. More specifically, the upstream and downstream pier foundations are exposed to notably different scour depths. However, it is common practice to assume a uniform scour depth at all bridge foundations in the water stream when evaluating the lateral stability of the structure. This paper uses a comparative case to identify the unintended consequences of this assumption of uniform local scour depth for bridges with multiple columns in the direction of flow. This analysis considers bridges with both uniform and nonuniform scour depth at the support foundations. The effects of variable scour depths on the behavior of bridge components is evaluated by tracking the demand redistribution as the scour depth changes. The results of this investigation are applied to understanding the performance of bridges crossing waterways when they are subjected to multi-hazard effects from flood-induced scour and earthquakes.

KEYWORDS: variable scour, earthquake, multiple hazards, dynamic behavior.
3.1 Introduction

The successive occurrence of multiple hazards may cause severe impacts on a structure beyond those considered in the initial design. Multiple hazards could occur with extreme events that are correlated and causative, such as an earthquake and tsunami, hurricane and storm surge, or flood and scour, or they can occur with uncorrelated and independent events, such as flood and earthquake (Ghosn et al. 2003; Alampalli & Ettouney, 2008; Alipour, 2016). Therefore, it is important to consider such multi-hazard scenarios in designing new bridges and rehabilitating existing bridges in regions that are expected to have one or more correlated or uncorrelated hazard events (Alipour and Shafei 2015 and 2016).

Among uncorrelated extreme events, flood-induced scour followed by an earthquake has received attention as a multi-hazard scenario (e.g., Alipour et al., 2013; Chandrasekaran & Banerjee, 2016; Guo & Chen, 2016; Prasad & Banerjee, 2013; Shang et al., 2017; Wang et al., 2015; Wang et al., 2014; Yilmaz et al., 2014). Flood-induced scour is defined as the removal of material from the riverbed and banks as a result of the erosive action of flowing water. Scour accounts for 58% of bridge failures in the United States (Briaud, 2006), and occurrences are exacerbated by the increase in frequency and magnitude of flood events across the contiguous United States (Mellilo et al., 2014; Douglas et al., 2017). While flood events by themselves can create multi-hazard conditions for bridges (such as flood-induced scour, hydraulic pressure, debris impact), the impact of these conditions can be magnified by the seismicity level in the region. As such, the occurrence of an earthquake in the presence of undetected scour represents a different level of multi-hazard scenario caused by uncorrelated events (earthquake and flood) that are not necessarily anticipated. Therefore, it is important to consider such multi-hazard scenarios in the design of new bridges and the rehabilitation of existing bridges in susceptible regions.
A study by Klinga and Alipour (2015) showed that the lateral performance of bridges is affected by the presence of scour. To minimize this adverse effect, the Federal Highway Administration (FHWA) recommends an underwater inspection every two years of all bridges with a submerged substructure. Furthermore, to ensure the stability of bridges against scour, AASHTO-LRFD (2015) requires the design of bridge foundations for scour resulting from a 100-year flood and stability checks based on a 500-year flood.

Research on the development of scour at pile group foundations indicates that pile arrangements, numbers, and spacing influence the extent of scour depth (Chang et al., 2013; Castiblanco, 2016; Moreno et al., 2016). Furthermore, the bathymetries following bridge failures show that bridge piers do not all experience the same scour depth at their foundations (e.g., Tubaldi et al., 2018; Song et al., 2015; Khan & Amanat, 2014). The nonuniformity in scour depth observed at the foundations may be attributed to variations in soil profiles or the difference in the velocity profiles between the downstream and upstream piers. Khan and Amanat (2014) reported that, for a multiple-pier bent, the upstream pier (facing the flow directly) shields the downstream pier, thus reducing the flow velocity at the downstream pier. In addition, sediments transported from the upstream pier could be deposited in the scour hole formed at the downstream pier.

Despite the evidence of nonuniformity in scour depth around bridge piers, studies are lacking that account for such variation in the multi-hazard performance assessment of bridges. The objective of this study is to investigate the effects of nonuniform soil erosion on the dynamic characteristics and seismic performance of bridges. This paper presents a detailed overview of the parameters that affect the development of a scour hole. Due to the uncertainty in these parameters, a probabilistic approach combined with the Latin hypercube sampling technique is used to estimate the scour depth resulting from flood events with 100- and 500-year return periods. The influence
of scour on the seismic demand of bridge components, such as the superstructure, piers, abutments, embankments, bearing pads, piles, and soil-pile-structure interaction, is considered through the change in scour depth. The results are presented by studying the effects of nonuniform scour depth for a number of scenarios that assume a different level of scour depth at a bent with multiple piers. The proposed evaluation approach will improve the understanding of the performance of bridges experiencing both uniform and nonuniform flood-induced scour—conditions that alter their dynamic characteristics—with earthquakes as the dynamic loading on the structures.

3.2 Scour Process and Probabilistic Estimation of Scour Depth

Multiple research groups have studied the scour process and estimated the maximum attainable scour depth around piers (e.g., Kothyari & Kumar, 2012; Lu et al., 2011; Melville & Coleman, 2000). These studies indicate that the three-dimensional vortex flow field around the bridge pier is the primary contributing factor to the development of the scour hole at the bridge pier foundations. The horseshoe vortex that develops at the front of the obstruction and the subsequent acceleration of the flow around the pier nose are responsible for the removal of soil sediment (Kothyari et al., 2014). The erosion of soil at the bridge pier depends on various parameters, with the most influential being the pier geometry, flow characteristic, and riverbed material (Kothyari et al., 2014; Briaud, 2015).

Three types of scour are expected at the location of bridges crossing waterways, depending on the riverbed characteristics: general scour, contraction scour, and local scour. Local scour is the most critical for structural stability. A study conducted by Lagasse et al. (2013) on the uncertainties in the evaluation of scour depths indicated that the analytical equations for estimating the scour levels, which are based on flume tests, are conservative. Shepperd et al. (2014) reviewed the literature for the most commonly used local scour depth equations for non-cohesive materials and
concluded that, while some equations overestimate the local scour depth, more recently proposed equations better predict the local scour depths measured in the field. AASHTO LRFD (2012) suggests using the HEC-18 (2012) equation to compute the design scour depth for a single-pile foundation in a non-cohesive material as follows:

$$\frac{y_s}{a} = 2k_1k_2k_3k_4 \left(\frac{y_1}{a}\right)^{0.35} Fr^{0.43}$$

(3.1)

where $y_s$ is the scour depth (m); $y_1$ is the flow depth directly upstream of the pier, $a$ is the pile diameter (m); $Fr$ is the Froude number directly upstream of the pier; and $k_1$, $k_2$, $k_3$, and $k_4$ are correction factors for the pier nose shape, angle of attack of the flow, size of dune, and size of bed material, respectively.

The flow depth at the upstream pier, $y_1$, is estimated for the given flow conditions by solving Manning’s equation for open channel flow:

$$Q = \frac{A}{n} R_h^{2/3} S^{1/2}$$

(3.2)

$$R_h = \frac{A}{P}$$

(3.3)

$$A = y_1 (b + y_1 \tan \theta)$$

(3.4)

$$P = \frac{b \sin \theta + 2y_1}{\sin \theta}$$

(3.5)

where $Q$ is the discharge flow rate, $A$ is the cross-sectional area of the flowing fluid, $n$ is Manning’s roughness number, $R_h$ is the hydraulic radius, $S$ is the slope energy, $b$ is the width of the channel, $\theta$ is the side slope, and $P$ is the wetted perimeter. Assuming a trapezoidal section with an aspect ratio of 2H:1V and substituting Equations 3.3 to 3.5 into Equation 3.1, $Q$ could be estimated as

$$Q = \frac{y_1(b+0.5y_1)}{n}, \left[\frac{y_1(b+0.5y_1)}{b+2\sqrt{3}y_1}\right]^{2/3} S^{1/2}$$

(3.6)
where a recursive approach is used to estimate $y_1$ based on $Q$ and $b$. Also, for a pile group foundation, the spacing, the number of piles in the group, and other parameters influence the geometry of the scour hole and must be considered when estimating the scour depth.

The major factor affecting the scour depth is the peak annual discharge rate, $Q$. This rate can be evaluated using historical flow data for all major streams in the U.S., available through the USGS Water Resources program. Figure 1 shows the time history of the annual peak discharge rate for the river flowing around the case study bridge. Considering the uncertainty associated with the different parameters of design scour depth, a probabilistic-based approach was adopted in this paper. To evaluate the best distribution describing the discharge flow rate, four different probability distribution functions (generalized extreme value, Weibull, extreme event, and Pearson Type III) were checked. Based on the Kolmogorov-Smirnov (K-S) test, the Pearson Type III distribution was selected as the best fit for the site discharge flow rate. The probabilistic distributions suggested by Alipour et al. (2013) for the other parameters in Equation 1 were also adopted in this study. A full review of the sources for the assumptions related to the distribution of the parameters can be found in Alipour et al. (2013). According to the FHWA, new bridge foundations should be designed to sustain the adverse effects resulting from a 100-year return flood, and existing foundations should be checked with a 500-year return flood. The local scour depths at the bridge foundation for flows with 100-year and 500-year return periods were obtained using a Latin hypercube sampling technique. The analysis resulted in average scour depths of 4.0 m and 8.0 m for flood events with 100- and 500-year return periods, respectively.

### 3.3 Modeling the Case Study Bridge in OpenSees

This case study uses OpenSees (Mazzoni et al., 2006) to model a two-span box girder bridge with a bent consisting of two piers founded on pile groups and supported at its ends by seat-
type abutments. The bridge pier has a circular cross section (diameter of 1.6 m) with a longitudinal reinforcement ratio of 2.5%. Each pier is supported on a pile cap founded on a group of 23 piles, where each pile has a cross-sectional diameter of 0.4 m and a longitudinal reinforcement ratio of 1.6%. Figure 2 shows the critical element behaviors and the elevation details at the bent along with the geometrical properties of the bridge.

The effects of differential scour depth were evaluated considering the following: (i) bridge with no scour (intact), hereafter called Case 1; (ii) bridges with a scour depth of 4.0 m and 8.0 m resulting from 100-year and 500-year return floods: Case 2 and Case 3, respectively; (iii) non-uniform scour cases associated with Case 2: Case 2-1 shows 25% more scour depth development in the upstream \( S_{2-1}^{up} = 5.0 \) m, Case 2-2 shows 50% more scour depth in the upstream \( S_{2-2}^{up} = 6.0 \) m, and the downstream scour is considered at 4.0 m \( S_{2-1}^{down} = S_{2-2}^{down} = 4.0 \) m; and (iv) non-uniform scour cases associated with Case 3: Case 3-1 shows 25% more scour depth development in the upstream \( S_{3-1}^{up} = 10.0 \) m, Case 3-2 shows 50% more scour depth in the upstream \( S_{3-2}^{up} = 12.0 \) m, and the downstream scour is considered at 8.0 m \( S_{3-1}^{down} = S_{3-2}^{down} = 8.0 \) m. To clarify the terminology used to refer to the piers, note that the plane of the bent is parallel to the flow direction; thus, the front pier is referred to as the upstream pier and the back pier, which is shielded by the upstream pier, is referred to as the downstream pier. The values for the 25% and 50% difference represent a range that has been observed in either the field or experiments or simulated using fluid-structure interaction analyses (e.g., Chang et al., 2013; Castiblanco, 2016; Moreno et al., 2016). The no-scour case (Case 1) is used to benchmark the impact of uniform and nonuniform scour on bridge system performance. Each scour state is simulated by removing the soil springs corresponding to the estimated scour depth and updating the characteristics of the soil springs based on the new elevation at the pier. The finite element model is composed of a linear elastic superstructure,
nonlinear fiber section columns, and seat-type abutments, and it integrates passive and active embankment responses, material nonlinearity, and the soil-pile-structure interaction.

### 3.3.1 Superstructure

The bridge deck is 23.0 m wide with a cross-sectional area of 12.0 m$^2$ and a back-to-back abutment length of 60.0 m. Each span is discretized into six elements of equal length to correctly capture the response of the superstructure (Aviram et al., 2008), and they are modeled using elastic beam-column elements. The beam cap has a cross-sectional area of 4.0 m$^2$ and is 11.0 m long. It is discretized into four equal elements and modeled using linear elastic beam-column elements with the cross-sectional properties of the beam assigned to these elements. Considering that the beam cap is restrained in torsion due to its connection to the deck, the torsional stiffness of the element is enhanced by applying an amplification factor to the beam cap torsional moment of inertia. Rotational masses are added to the superstructure (Equation 3.7) and the column (Equation 3.8) nodes:

\[
M_{XX} = \frac{m_{l\text{sup}} L_d d_w^2}{12} \tag{3.7}
\]

\[
M_{ZZ} = \frac{m_{l\text{col}} L_{col} D_{col}^2}{8} \tag{3.8}
\]

where $m_{l\text{sup}}$ and $m_{l\text{col}}$ are the linear masses of the superstructure and column, respectively; $L_d$ and $L_{col}$ are the tributary lengths associated with the deck node and column node, respectively; $d_w$ is the superstructure width, and $D_{col}$ is the column diameter.

### 3.3.2 Piers

The column is 10.0 m high with a diameter of 1.6 m. Each column is modeled using force-based beam-column elements. These elements have the advantages of being computationally less expensive, as they use fewer elements per column compared to displacement-based elements, and
being able to capture the large curvature that could develop in the plastic hinge region (Scott et al., 2006; Filipov et al., 2013). The use of force-based beam-column elements also allows for the distribution of plasticity along the length of the column rather than confined to specific user-defined lengths, as is the case with beam-with-hinge elements commonly used in the literature.

### 3.3.3 Soil-pile-structure interaction

The bridge pier foundation consists of a group of 23 18-meter-long Class-140 precast prestressed reinforced concrete piles designed following California Department of Transportation Seismic Design Criteria (SDC, 2013) recommendations. The global response of the bridge structure to the static and dynamic loadings depends on the soil-pile-structure interaction (SPSI). The SPSI effects are caused by the inertia of the soil medium and the supported structure, the relative stiffness of the foundation structures to the surrounding soil, the unbounded nature of the soil (Shamsabadi et al., 2013), the flexibility of the piles and the interaction between the pile and surrounding soil, and the transfer of deformations to the structure.

Pile foundations have long been designed as capacity-protected elements. However, studies have shown that, even under this design criterion, seismic failure can occur in the pile, particularly at its connection with the pile cap where bending moments tend to be the largest. Tang and Liang’s (2014) experiments indicate that failure mechanisms can develop in the pile foundation at the location where the seismically induced bending moment is greater than the pile’s plastic moment capacity. Fioklou and Alipour (2015) showed that the presence of scour at the bridge foundation causes the location of the maximum bending moment to shift downward as the scouring progresses. To account for the possibility of developing failure mechanisms in the subgrade, the piles in this study are modeled using displacement-based beam-column elements. Such elements can capture the formation of plastic hinges along the piles as the scour depth increases.
The soil at the location of the bridge piers is sand with an assumed unit weight of 19.64 kN/m$^3$, a Poisson’s ratio of 0.3, and an average corrected blow count ($N_{60}$) of 20. To model the SPSI, the soil column is discretized into layers having a thickness corresponding to the pile element length (0.5 m). Each soil layer is represented by three nonlinear soil spring elements attached to each pile node. These soil springs are oriented in the longitudinal ($p-y$), transverse ($p-y$), and vertical ($t-z$, $q-z$) directions and are readily implemented in OpenSees. Note that a $q-z$ spring is only assigned to the pile tip, and the soil properties are assumed identical in all directions. The ultimate lateral resistances of the soil in the longitudinal and transverse directions, ($P_{ult}$), are calculated as suggested by API (1987) and Boulanger et al. (2003); the ultimate pile tip resistance, $q_{ult}$, is estimated following Meyerhoff (1976); and the ultimate friction capacity between the soil and pile, $t_{ult}$, is calculated based on Mosher (1984). In the estimation of the $p-y$ and $t-z$ curves, the overburden pressure is considered following the guidelines provided by Hannigan et al. (2006). For this purpose, the pile axial capacity is computed from the post-scour ground level. The influence of reduced overburden pressure due to only general and contraction scours is included in estimates of the pile axial capacity, while that due to local scour is ignored.

One important factor to consider is that the performance of a single pile differs from the performance of a pile in a group. Experimental studies on pile groups indicate that the reduction in the lateral resistance of a pile in a group is due to group interaction effects and that the lateral resistance is a function of the row location. These studies further show that pile spacing is the dominant factor affecting the pile group response. The $p$-multiplier method (Brown et al., 1988) is used here to account for the group effect. In this method, the soil resistance from $p-y$ curves is reduced based on location with respect to the pile group. Because of the alternating ground motions, the front row changes as well. For this purpose, an average $p$-multiplier is estimated for
the first, second, and third (central) row of piles as 0.3, 0.5, and 0.85. These multipliers are then applied to each row of piles to achieve the global response of the bridge (Curras et al., 2001).

### 3.3.4 Abutment system modeling

The seat-type abutment of the case study bridge is modeled following the procedure outlined by CalTrans (SDC, 2013). The approach uses rigid elements to represent the stem wall. Soil springs in the vertical, transverse, and longitudinal directions are attached to the ends of the wall to represent the bearing pads, the wing wall, and the passive action of the abutment backwall fill, respectively. Only longitudinal and vertical springs are used at the abutment interior nodes. A comparative study of four different embankments suggests that the bridge embankment model should comprise the abutment, superstructure, substructure, and a sufficient length of the embankment known as the critical length (Kwon & Elneshai, 2007). For the seat abutment under study, the stem wall is supported on a sand medium with a Poisson’s ratio of 0.35, a density of 1760 kN/m$^3$, and a shear wave velocity of 150 m/s. The height of the embankment, $H$, is 4.5 m with a crest width, $B_c$, of 23 m and a side slope, $S$, of 0.5. The critical length, $L_c$, can be approximated using Equation 10 (Zhang & Markis 2002a). Using the shear beam approximation (Wilson, 1988; Zhang & Markis, 2002b), the stiffness of a unit width embankment can be estimated using Equation 9 and then multiplied by $L_c$ to obtain the stiffness of the entire soil embankment:

$$k_z = \frac{4(1+\nu_s)G_s}{S \ln(1+\frac{2H}{SB_c})}$$  \hspace{1cm} (3.9)

$$L_c = 0.7 \sqrt{SB_cH}$$  \hspace{1cm} (3.10)

where $G_s$ and $\nu_s$ are the shear modulus and the Poisson’s ratio of the embankment soil, respectively.
The stiffness of the bearing pads in the vertical and longitudinal directions were calculated following AASHTO (2012) recommendations. Two springs representing the bearing and the embankment soil are placed in parallel and attached to the abutment nodes in the vertical direction:

\[ K_H = \frac{G A}{H_r} \]  

(3.11)

\[ K_V = \frac{E A}{H_b} \]  

(3.12)

where \( A \) is the elastomer gross plane area, \( H_b \) is bearing height, \( H_r \) is the bearing total thickness, \( G \) and \( E \) are the shear and Young’s moduli of the bearing, respectively.

The hyperbolic nonlinear model proposed by Shamsabadi (2007) is employed to capture the nonlinear behavior of the abutment-embankment system in the longitudinal direction. Zero-length spring elements are used to represent the load deformation of the abutment system. This element includes a 5.0 cm gap element, which represents the expansion joint between the superstructure and the abutment. Additionally, the bearing pad spring elements are placed in parallel with the backwall backfill elements. The behavior of the abutment is that of a gap element in series with the backfill compressive element, and both are in parallel with the elastic bearing pad element. In the transverse direction, a bilinear model is used to capture the behavior of the backfill. The backfill spring elements are placed in series with a gap element. The stiffness and strength of the springs were calculated using equations suggested by Scott and Fenves (2006) and modified according to Maroney and Chai (1994) for wall effectiveness \( (C_L = 2/3) \) and participation coefficients \( (C_w = 4/3) \).
3.3.5 Impact element modeling

Bridges located in seismically active regions are prone to damage caused by pounding impact forces. These impact forces are the result of out-of-phase movement between various components of the bridge system. Pounding is known to cause localized damage to the deck and failure of the bearing, shear keys, and abutment.

The elements used to model the impact are only activated if the gap between the sections is exhausted. Both approaches have their limitations. This paper uses the model proposed by Muthukumar (2003), which is a bilinear truss element with gap (Figure 3.4). The model is based on the Hertz-damp approach, which is capable of capturing the effect of pounding in bridges and accounting for the energy dissipation due to impact. The stiffness parameters are estimated using Equations 13–16:

\[ \Delta E = \frac{K_h \delta_{m+1}^2}{n+1} (1 - e^2) \]  \hspace{1cm} (3.13)

\[ K_{\text{eff}} = K_h \sqrt{\delta_m} \]  \hspace{1cm} (3.14)

\[ K_{t1} = K_{\text{eff}} + \frac{\Delta E}{a \delta_m^2} \]  \hspace{1cm} (3.15)

\[ K_{t2} = K_{\text{eff}} - \frac{\Delta E}{(1-a)\delta_m^2} \]  \hspace{1cm} (3.16)

where \( K_h \) is the Hertz stiffness (\( \approx 25,000 \text{ kip-in}^{-3/2} \)); \( \Delta E \) is the energy loss during impact; \( n \) is the Hertz coefficient and is taken as 1.5; \( e \) is the coefficient of restitution (\( =0.7 \)); and \( a \), the yield displacement parameter, is assumed to be 10% of the maximum displacement, \( \delta_m \). Table 3.1 summarizes all the stiffness values for the columns, pile, bearing, and abutment.
3.4 Analysis

To evaluate the performance of the bridge with different scour scenario cases, modal and nonlinear time history analyses (NTHA) were performed. The modal analysis of the bridge in its intact state is conducted first, followed by the four scour states introduced in Figure 3.3. It is important to evaluate the dynamic characteristics of the structure, as scour may shift the natural frequency, resulting in a dynamic performance that is not anticipated in the original design. An eigenvector analysis was employed to determine the dynamic characteristics of the bridges for the different scour states.

The modal analyses were followed by NTHA to determine the seismic demands of each bridge case in the longitudinal and transverse directions. A suite of 60 ground motions representing the 2%, 10%, and 50% exceedance probability in 50 years was employed. These ground motions were originally developed by the Federal Emergency Management Agency (FEMA) for Los Angeles, California. Table 3.2 shows the characteristics of the ground motions. These ground motions were applied in the direction of interest (longitudinal or transverse) during the analysis. The transverse direction results, although different in value from the longitudinal direction results, have the lowest magnitude but follow similar trends. Therefore, only the results in the longitudinal direction are discussed in the next section.

3.5 Discussion of Results

The scour resulting from 100-year (4.0 m) and 500-year (8.0 m) return floods are equivalent to approximately 22% and 44% of the pile length, respectively. Figure 3.5 shows the modal analysis results for the bridge in its intact and scoured conditions for Cases 3, 3-1, and 3-2. The mode shapes for Cases 2, 2-1, and 2-2 are not shown, as they are similar to those of the intact bridge case but with different values, as shown in the table embedded in Figure 3.5.
As can be seen in Figure 3.5, the first two modes of the intact bridge (Case 1) correspond to longitudinal and transverse vibrations. These two modes involve components that are directly affected by soil erosion. The third mode corresponds to the in-plane vibration of the deck and could be justified by the lower participation of the foundation stiffness in this mode.

The first mode is identical among Cases 3, 3-1, and 3-2 and occurs in the longitudinal direction. The second mode of vibration is in the transverse direction for all cases except for Case 3-2, which is a torsional mode. In the third mode, each of the cases represents different modes of vibration, such as a combination of in-plane rotation and torsion in Cases 3 and 3-1 and pure transverse in Case 3-2. The noticeable difference in the third mode of vibration between the intact case and the bridge cases with higher scour depths is indicative of a larger contribution of the foundation in this mode. Furthermore, the extent of scouring not only affects the period of the structure but also influences higher mode shapes.

A comparative analysis of identical modes of the bridge cases shows that the natural periods increase with scour depth. The lengthening of the natural periods is substantial in the first mode of vibration of the bridge. The first fundamental periods of Cases 2, 2-1, and 2-2 increase by 90%, 96%, and 107% when compared to Case 1, respectively (table in Figure 3.5). These increases in the first fundamental periods are larger for the higher scour depths associated with Cases 3, 3-1, and 3-2. The effect of nonuniformity in scour depth between the downstream and upstream pier foundations on the modal characteristic of the bridge cases is assessed by comparing the period of identical modes. Cases 2-1 and 2-2 show 3% and 9% increases in the first fundamental period, respectively, when compared to Case 2; whereas 3% and 6% increases are observed in the first fundamental period for Cases 3-1 and 3-2, respectively, when compared to Case 3. This indicates a change in the stiffness of the structure beyond what was considered in the initial design. The
third and higher modes display little influence of scour on the bridge frequencies of vibration, which could be attributed to the lower participation of the foundation stiffness in the higher modes of vibrations.

To understand the effect of scouring on the seismic behavior of the bridge, different performance measures, such as (i) the deck center of mass displacement, (ii) moments and curvatures in the column plastic hinge region, (iii) abutment forces and displacements, (iv) column base shear, (v) impact forces in the gap elements, and (vi) the forces in the pile group foundations, were evaluated during the NTHA under a suite of 60 ground motions. The responses of the different bridge cases are first discussed in detail for ground motions with the highest peak ground acceleration (PGA) in each hazard category (2%, 10%, and 50% exceedance probabilities in 50 years). First, the fundamental periods of the bridges with different scour scenarios are compared with the spectral acceleration of the three ground motions representing different hazard levels. As observed, the shift in natural period moves the structure to the portion of the seismic spectrum that has less intensity. This traditionally has misled some previous studies to conclude that the presence of scour in fact helps with bridge seismic performance. A more rigorous study that considers the bridge as a structural system, as presented later in this paper, questions these earlier discussions.

Figure 3.7 shows the time histories of the three ground motions considered together with the deck center of mass displacement time history responses for the intact bridge and for bridges with a uniform scour of 4.0 and 8.0 m. At a peak intensity of 1.33 g for the third ground motion (GM #3 in Figure 3.7), the erosion of 4.0 m of soil around the foundation (44% increase in the unsupported length of the pile) results in a 50% increase in the deck center of mass displacement. This provides evidence of change in the flexibility of the structure due to scour. In addition, the increase in the period of the response, seen clearly in the free vibration portion of the response,
underlines the substantial change in the dynamic characteristics of the structure. Comparing the results for the three scour cases (Cases 2, 2-1, and 2-2) in Figure 3.7, one can observe the similarity in the peak displacements.

Figure 3.8a shows the maximum displacements for Cases 1, 2, and 3 under the suite of ground motions. As the scour depth increases, the maximum displacement also increases. To show the effect of differential scour on the maximum displacement, the results from bridges with nonuniform scour depth cases were normalized by results from respective bridges with uniform scour depths (e.g., the results from Case 2-1 were normalized with Case 2). Figures 3.8b and c show the normalized maximum deck center of mass responses. A ratio greater than unity indicates that the bridge case with nonuniform scour results in larger demand, whereas a ratio smaller than one implies that nonuniform scour results in less demand on the structure. A 25% difference in the scour depth between the upstream and downstream piers results in a -12% to +25% change in the maximum displacement for Case 2-1, while Case 3-1 displays a change that ranges from -17% to +33% in the maximum displacement. When the difference in scour depth is increased to 50%, Cases 2-2 and 3-2 display changes that range from -13% to +30% and -19% to +56% in maximum displacement, respectively. These results indicate that using a uniform distribution of scour depth to assess the seismic displacement demand of bridges will underestimate the demand that the bridge is expected to sustain.

Figures 3.9 displays the moment-curvature hysteresis loops in the plastic hinge region at the bottom of the pier. Figure 3.9a shows the intact bridge (Case 1) as well as the bridges with uniform scour depths of 4.0 m (Case 2) and 8.0 m (Case 3). Figure 3.9b presents the response at the upstream pier for Case 2 and the nonuniform Case 2-1 ($S_{2-1}^{up}=5.0$ m) and Case 2-2 ($S_{2-2}^{up}=6.0$ m) at the upstream pier, and Figure 3.8c displays the response at the downstream pier for the same
bridges for Cases 2-1 and 2-2 ($S_{dwn}^{2-1}=S_{dwn}^{2-2}=4.0$ m) under the three selected levels of ground motion. The comparison of the results of the intact bridge and the bridges with uniform scour (Cases 2 and 3) shows that, at lower seismic excitations, the response of the columns with or without scour remains in the elastic range (Figure 3.9a, left). As the intensity of the earthquake increases, the intact bridge absorbs more of the seismically induced energy through the development of plastic hinges. However, the bridges with scour remain in the elastic range. The response to the ground motion with the highest PGA (GM #3) displays a noticeable hysteresis loop in the plastic hinge region for both the intact bridge and the bridge with a 4.0 m scour depth, while the bridge with higher scour depth (8.0 m) remains elastic (Figure 3.9a, right). This caused by two phenomena: (1) with higher scour depths and a shift in the natural period of the structure, the bridge absorbs less energy from earthquakes (see Figure 3.6); and (2) the extra flexibility at the bridge support results in larger displacements at the deck level, which results in shifting the demand to other parts of the structure. Figures 9b and c show the respective performance at the base of the upstream and downstream piers for bridges with the scour depths in Cases 2-1 and 2-1. Comparing the results of the high-intensity ground motion (GM #3) it is evident that the upstream pier absorbs less energy compared to the downstream pier. This is due to the decrease in local stiffness at the foundation of the upstream pier considering the deeper scour depth, which results in less energy being absorbed at the base of the pier. For the downstream piers, the moment curvature plots show that the maximum curvature demand increases for a bridge with nonuniform scour depth compared to a bridge with uniform scour depth. This is due to the fact that the local reduced stiffness at the bridge downstream pier results in a larger portion of the seismic energy being absorbed at this pier. The largest energy dissipation is observed in the nonuniform scour cases. The moment-curvature responses show that the curvature demand at the base of the piers
varies with the level of scour at the foundation. These results indicate that the condition at the downstream pier influences the response at the upstream pier and vice versa.

To follow the redistribution of the forces in the bridge system, the force-displacement response of the abutment for uniform and nonuniform scour cases is show in Figure 3.10. In compressive loading, the resistance to the deck movement is provided only by the bearing pads prior to the gap closure, after which the whole system (back wall, bearing pads, and embankment soil) is engaged in resisting the deck movement. The unloading of the abutment system occurs elastically through the bearing pads as the abutment and embankment soil are modeled to act only in compression. The reloading of the embankment is initiated only after prior deformation in the soil is exhausted by the pads. Similar behaviors in the abutment system are observed at the other ends of the bridge deck. Although the erosion of soil resulted in less rotational demand at the base of the piers, the energy dissipated at the abutments increases with an increase in the scour depth. For the same ground motion, Figure 3.10a shows that more demand is imposed on the abutment due to scour. In addition, the nonuniformity in the erosive action of the flowing water between the upstream and downstream piers results in an additional displacement of the abutment (Figure 3.10b). This increase in displacement compensates for the inability of the piers to absorb the energy induced in the system. The implication of these observations is that, as the scour depth increases, there is a shift in demand from one component of the bridge system to another. Additionally, the excessive displacement may result in the unseating of the deck at the abutment level and thus the collapse of the bridge. Furthermore, as the scour depth and ground motion intensity are increasing, the impact forces recorded on the abutment back wall are also increasing (Figure 3.10c). If these forces exceed the compressive strength of the abutment concrete wall, cracks might ensue in the wall, which will result in poor performance of the abutment under repeated loading.
The shear force demands were recorded at the base of the downstream and upstream piers for all hazard levels. The maximum base shear (at the first integration point of the plastic hinge region) under the suite of ground motions for the uniform scour cases are shown in Figure 3.11. As the scour depth increases, the maximum base shear force demand at the base of the pier decreases under the same ground motion (Figure 3.11, top). This is partially due to the decrease in seismic demand due to the shift in the natural period of the structure, as well as the fact that, due to the extra flexibility induced at the base of the column, the demand is distributed to other components of the system. To show the effect of differential scour on the shear force response, the results of the nonuniform scour depth cases are normalized to their respective uniform cases (for instance, the base shear at the upstream pier of the bridge with nonuniform scour is divided by the base shear at the upstream pier of the bridge with uniform scour). Figure 3.11 (middle and bottom) displays the maximum responses at the piers normalized with those from the uniform scour cases at scour depths of 4.0 m and 8.0 m, respectively. As shown in this figure, there is variability in the maximum shear force absorbed at the base of the piers, particularly at the downstream pier, for Cases 2-1 and 2-2 compared to the uniform cases. As the degree of nonuniformity increases, on average, a 20% reduction in maximum shear force is recorded at the base of the upstream pier foundation experiencing 4.0 m and 8.0 m scour. As much as a 40% increase in shear force is observed at the downstream pier base for Case 3-2. The inertia forces induced at the base of the piers are related to the structure by its mass and period. Since the structure is becoming more flexible due to loss of support around the piles, it is absorbing less energy from the ground motion. However, within the structure, the stiffness of the upstream foundation decreases more than that of the downstream foundation, which results in less contribution of the upstream pier to the overall seismic performance of the bridge.
Figure 3.12 demonstrates the effect of soil erosion around the foundation on the curvature demand in the plastic hinge region of the pier. As can be seen in this figure, the curvature demands change with changes in the scour depth. Figure 3.12 (top) shows ±10% variation in the curvature demand between uniform and nonuniform scour, with a few instances where the ratio is greater than 20%. By isolating the downstream pier, the observed decrease in curvature demand with increasing scour depth is consistent with previous observations. In general, a 50% difference in scour depth at the foundations results in a 20% increase in the curvature demand for Cases 2-2 and 3-2. At the same flood hazard level, even though the downstream pier foundation experiences the same erosive action as in the uniform case, the curvature demands are different. Under a 100-year return flood, the curvature demand at the downstream pier varies between ±20% for a 50% differential scour depth, and, in a few cases, as much as a 40% decrease and 60% increase are observed. The same variation can be seen under a 500-year flood hazard. It can be concluded that the curvature demand in the plastic hinge region of the piers is affected by the boundary conditions at the foundations. Because scour is a random phenomenon, using uniform scour at a multi-column foundation will lead to an underestimate in the seismic rotational demand at the base of bridge piers with scoured foundations.

Figure 3.13 shows the response of the substructure components (soil and pile) under the abovementioned three ground motions for the uniform scour cases at one of the piles in the third row. The soil response is recorded by measuring the forces generated in the $p-y$ springs, while the shear and bending moments are recorded at the pile elements. To generate these graphs, the time associated with the first occurrence of the maximum soil response during the ground shaking is retained, and the corresponding values of the shear and bending demands at the same time instance are extracted along the length of the pile. The resistance of the soil to the pile deflection is
concentrated in the top 4.0 m of the soil column, with the largest demand recorded at a depth ranging from 2D to 7D below the soil surface, where $D$ is the pile diameter. This is most obvious when the scour depth reaches 8.0 m (Case 3), when barely any forces are imposed on the remaining soil springs. For this case, most of the lateral displacement occurs in the exposed pile elements. In Case 2, with the increase in the intensity of the ground motion, the forces resisted by the soil also increase.

The distribution of shear force along the length of the pile (Figure 3.13, middle column) shows that, with the increase in scour depth, the shear force at the pile cap level decreases. This is in agreement with the decrease in the base shear force observed at the pier as the scour depth increases. However, similar trends are not apparent along the length of the pile. In all cases, the largest shear and moment demand are recorded with 4.0 m scour depth. At this depth, there is still a fair amount of resistance provided by the substructure. Although the total base shear decreases due to the shift in the frequency of the structure and energy absorbed, the foundation still contributes significantly to the resistance. By increasing the scour depth to 8.0 m, the total shear forces induced by the ground motion decrease considerably. Additionally, the stiffness of the substructure decreases which means less of the energy induced in the system is absorbed by the substructure. The moment distribution along the length of the piles shows that, with the increase in scour depth, the peak moments observed increase and then slightly decrease for the 8.0 m scour depth. The maximum bending moment in the pile is shifted toward the soil interface or grade level as the scour depth increases.

### 3.6 Conclusions

Field observations indicate that the action of flowing water at bridge pier locations is random and that the maximum attainable local scour depth at the time of failure is unequal between
downstream and upstream pile foundations. However, assumption of uniform flood-induced erosion of sediments around downstream and upstream pier foundations has been the traditional approach in the design community. This work investigates the seismic performance consequences of the unrealistic assumption of identical scour depths at different pier locations in highway bridges. For this purpose, a case study bridge is subjected to the erosive action resulting from flood hazards with 100 and 500-year return periods. For each flood hazard level, a differential scour depth of 25% and 50% between the upstream and downstream piers is considered. Finally, for all bridge cases, including the intact bridge, modal analyses are performed followed by a dynamic excitation using a suite of 60 ground motions representing three different levels of hazards. The performance of the bridge is evaluated at the component level in order to identify the component sensitivity to both uniform and nonuniform scour depth in particular.

The modal analysis shows that scour at the bridge foundation leads to lengthening of the natural period. The shift in natural periods in this case results in the structural dynamics move away from the highest frequency content under considered earthquake motions, hence decreasing the amount of seismic energy experienced by the structure. However, due to the excessive flexibility in the foundation support, more displacement demand is exposed to other components, such as abutments. Furthermore, the comparative results of the case studies indicate that identical scour depth for the two piers does not always produce the maximum responses for design. The performance metrics investigated in this study show that, for the downstream pier foundation, which experienced lower levels of scouring in uniform and nonuniform scour cases, the shear force, moment, and curvature demands are not identical, indicating the local shift in demand between piers with different levels of scour. The curvature demand was found to increase with the degree of nonuniformity at the downstream pier. The maximum shear force and moment in the
piles shift downward as the scour action progresses, and these maxima are located approximately between 2D and 7D below the soil surface. This study highlights the importance of considering appropriate characteristics of possible flood-induced scour in the seismic performance assessment of bridges. It should be noted that the results are limited by the assumption inherent to the formulation of the soil stiffness through the $p$-$y$ curves when accounting for overburden pressure in scoured piles. A viable future work for the current study is to account for such effects in the response of bridges.

References


Yao, C. (2013). LRFD Calibration of bridge foundations subjected to scour and risk analysis (Doctoral dissertation). Texas A&M University, College Station, TX.


**Table 3.1:** Stiffness of the different components of the bridge considered in the OpenSees model.

<table>
<thead>
<tr>
<th>Component</th>
<th>$K_z$ (kN/m)</th>
<th>$K_x$ (kN/m)</th>
<th>$K_v$ (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column</td>
<td>96,008</td>
<td>96,008</td>
<td>-</td>
</tr>
<tr>
<td>Pile</td>
<td>18</td>
<td>18</td>
<td>-</td>
</tr>
<tr>
<td>Bearing</td>
<td>1073.6</td>
<td>1073.6</td>
<td>56,507</td>
</tr>
<tr>
<td>Abutment</td>
<td>218,595</td>
<td>615,868</td>
<td>19,869,391</td>
</tr>
</tbody>
</table>

**Table 3.2:** Characteristics of the three earthquake motions representative of each hazard level, including the peak acceleration ($a_{max}$), peak velocity ($v_{max}$), peak displacement ($d_{max}$), and Arias intensity ($I_a$).

<table>
<thead>
<tr>
<th>Ground Motion</th>
<th>$a_{max}$ (g)</th>
<th>$v_{max}$ (m/sec)</th>
<th>$d_{max}$ (m)</th>
<th>$I_a$ (m/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>GM#1</td>
<td>0.112</td>
<td>0.226</td>
<td>0.143</td>
<td>0.225</td>
</tr>
<tr>
<td>GM#2</td>
<td>0.569</td>
<td>0.802</td>
<td>0.173</td>
<td>2.404</td>
</tr>
<tr>
<td>GM#3</td>
<td>1.323</td>
<td>1.936</td>
<td>0.438</td>
<td>13.28</td>
</tr>
</tbody>
</table>
**Figure 3.1.** Annual peak flow rate discharge of the case study river
Figure 3.2. Schematics of the bridge under study.
**Scour scenarios**

<table>
<thead>
<tr>
<th>Upstream pier</th>
<th>Case 2-1</th>
<th>Case 2-2</th>
<th>Case 3-1</th>
<th>Case 3-2</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.0 m</td>
<td>6.0 m</td>
<td>10.0 m</td>
<td>12.0 m</td>
<td></td>
</tr>
<tr>
<td>Downstream pier</td>
<td>4.0 m</td>
<td>4.0 m</td>
<td>8.0 m</td>
<td>8.0 m</td>
</tr>
</tbody>
</table>

**Figure 3.3.** Definition of the upstream and downstream piers and the considered nonuniform scour scenarios.

**Figure 3.4.** Analytical model of pounding between bridge elements.
Figure 3.5. Mode shapes for the intact bridge and different bridge cases under uniform and nonuniform scour associated with Case 3 and the frequencies of first three modes for all presented cases.
Figure 3.6. Spectral acceleration for the selected ground motions and the fundamental periods of bridges with all scour scenarios considered.
Figure 3.7. (a) The three ground motions representative of the three hazard levels of 2%, 10%, and 50% in 50 years; (b) the deck center of mass displacement time history of the intact bridge and bridges with uniform scour of 4.0 and 8.0 m; and (c) the deck center of mass displacement time history for bridges with nonuniform Case 2-1 ($S_{2-1}^{up}=5.0$ m) and Case 2-2 ($S_{2-2}^{up}=6.0$ m).
Figure 3.8. (a) Maximum deck center of mass displacement for intact bridge and bridge with uniform scour of 4.0 and 8.0 m, (b) normalized maximum displacement for nonuniform scour Case 2-1 ($S_{2-1}^{up}=5.0$ m) and Case 2-2 ($S_{2-2}^{up}= 6.0$ m), and (c) normalized maximum displacement for nonuniform scour cases associated with Case 3-1 ($S_{3-1}^{up}=10.0$ m) and Case 3-2 ($S_{3-2}^{up}=12.0$ m).
Figure 3.9. Moment curvature demand at the plastic hinge located at the bottom of the pier in (a) intact bridge and bridge with uniform scour of 4.0 and 8.0 m, (b) upstream pier of the bridge with nonuniform scour Case 2-1 ($S_{2-1}^{up}$=5.0 m) and Case 2-2 ($S_{2-2}^{up}$=6.0 m), and (c) downstream pier for Case 2-1 and Case 2-2 ($S_{2-1}^{down}$=$S_{2-2}^{down}$=4.0 m).
Figure 3.10. Abutment force displacement response for (a) intact bridge and bridge with uniform scour of 4.0 and 8.0 m, (b) the bridge with uniform scour of 4.0 m and the bridge with nonuniform scour Case 2-1 ($S_{2-1}^{up}=5.0$ m) and Case 2-2 ($S_{2-2}^{up}=6.0$ m), and (c) the impact force displacement response for intact bridge and bridges under uniform scour 4.0 and 8.0 m.
Figure 3.11. Maximum base shear for intact bridge and bridges with uniform scour of 4.0 and 8.0 m (top); maximum shear demand ratio for nonuniform scour of Case 2-1 ($S_{2-1}^{up}=5.0$ m) (middle left) and Case 2-2 ($S_{2-2}^{up}=6.0$ m) (middle right); maximum shear demand ratio for nonuniform scour of Case 3-1 ($S_{3-1}^{up}=10.0$ m) (bottom left) and Case 3-2 ($S_{3-2}^{up}=12.0$ m) (bottom right).
Figure 3.12. Maximum base curvature for intact bridge and bridges with uniform scour of 4.0 and 8.0 m (top); maximum curvature demand ratio for nonuniform scour of Case 2-1 ($S_{2-1} = 5.0$ m) (middle left) and Case 2-2 ($S_{2-2} = 6.0$ m) (middle right); maximum curvature demand ratio for nonuniform scour of Case 3-1 ($S_{3-1} = 10.0$ m) (bottom left) and Case 3-2 ($S_{3-2} = 12.0$ m) (bottom right).
Figure 3.13. Soil response and shear and bending moments generated along the length of the piles under the three ground motions for three cases of intact bridge with Case 2 (uniform scour =4.0 m) and Case 3 (uniform scour = 8.0 m).
CHAPTER 4. INTEGRATION OF CLIMATE CHANGE DATA INTO DESIGN OF COUNTERMEASURES OF BRIDGES UNDER FLOOD EVENTS

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Ameh FIOKLOU and Alice ALIPOUR

Abstract

The available research on the impact of climate change suggest the increase in frequency, duration and intensity of heavy precipitation. This change in precipitation trends will affect the runoff that must be handled by rivers causing floods and accelerated erosion. The formation of scour hole around the bridge piers and abutments in waterways is affected by the frequency and intensity of discharge rates at the bridge location. Climate change could increase scour related bridge closures and failures by the effects it has on discharge rate at bridge location. To enhance the resiliency of transportation infrastructure, there is a need for integration of climate change in engineering design. However, very little effort has been made to assess how climate change increase the risk of scour failure and how to adapt bridges to this emerging environmental stressor. This paper integrates the climate and hydrologic data developed under a recently completed FHWA pilot project for climate change resiliency and provides a framework to assess vulnerability and design adaptation strategies for bridges. The paper highlights the use of multiple countermeasure designs and uses a benefit-cost approach to guide the choice of an optimal countermeasure. The selection of the appropriate countermeasure to reduce the accelerated soil erosion around the piers was based on the economic loss associated with the bridge failure. While it is unequivocal that adaptation of bridges to climate change will improve their performance to future environmental conditions, the results of analyses help identify the most important parameters that influence the magnitude of the infrastructures vulnerability and the design of the countermeasure to alleviate the impact of climate change.
4.1 Introduction

Climate drivers (e.g., rainfall, temperature, sea level rise, precipitation, and hurricanes) are key parameters that adversely affect the resilience of the transportation infrastructure, specifically by detrimental effects on bridges and roadways. For example, 2015 and 2016 flooding events in Texas, Oklahoma, Iowa and Florida have resulted in failure and closure of multiple bridges (Erdman 2016, Danner and Fuler 2015, Fechter 2015). These events have highlighted the fact that the existing infrastructure designs are inadequate to handle stressors that are beyond the historically recorded frequencies and intensities. In fact, over the past decade many simulations of future climate predictions have confirmed that the climate is no longer stationary (Milly et al. 2008, Walsh et al. 2014, Kilgore et al. 2016). The increase in temperature, intense precipitation events, intense hurricanes, and rising sea levels would affect the transportation sectors primarily through increases in several types of weather and climate extremes (MacArthur et al. 2012, Burbank et al. 2012, Kilgore et al. 2016).

Many studies have been directed toward the evaluation of the effects of climate change on river flows and their hydrologic regimes (e.g. Groisman et al. 2001, Tebaldi et al. 2006). All of these studies indicate, with high confidence, an increase in the frequency and the duration of extreme precipitation events across the U.S. due to climate change. In the United States, highway bridges have been designed to withstand floods with a recurring period of 100 years, while the stability of existing bridge foundations is based on flood events with a 500-year return period. These design floods are based on historical annual discharge rate data collected at the gauge stations near the bridge sites. These design estimates are based on the assumption that climate is stationary over time. However, current observations suggest that the past is no longer a reliable indicator of the conditions under which infrastructure will have to perform in the future (Walsh et al. 2014). Evidence of the change in weather conditions show that flood events exceeding current
design floods will become frequent in most regions. This finding raises the concern about the potential of damage to the bridges in the United States that are located over waterways (approximately over 500,000). A survey of 503 bridge failure cases between 1989 and 2000 concluded that hydraulic failures represent the number one cause of bridge failure in the U.S. with scour accounting for 53% of the failures (Wardhana and Hadipriono 2003, Flint et al. 2017). This underlines the importance of assessing the bridge vulnerability to future weather conditions so that proper adaptation strategies are put in place by transportation agencies.

There has been numerous research on evaluation of performance of bridges under flood-induced scour; this includes approaches to estimate scour at bridges (Arneson et al. 2012, and Briaud 2014), effects of scour on bridge lateral performance (Klinga and Alipour 2015, Lin et al. 2014), scour in multi-hazard conditions (Alipour et al. 2013; Yilmaz et al. 2016; Gehl and D’Ayala 2016; Zhu and Frangopol 2016). However, all of the mentioned studies have used the historical data to assess the risk and vulnerability of bridges under flood events.

Using the results of global climate models (GCM) and downscaling techniques to derive regional-scale data can help with achieving flood hazards considering the effects of climate change. Global climate models are generated using a general circulation model to simulate atmospheric conditions. GCMs can also be constructed for different scenarios of future greenhouse gas (GHG) emission. To use these models for localized conditions such as those pertaining to design and decision making of the infrastructure components, downscaled or high resolution climate models should be generated. This can be achieved by using statistical downscaling methods or running a higher resolution GCM, using boundary conditions of surrounding global climate model. There has been numerous studies that produce downscaled models from GCMs (e.g., Anderson et al. 2003; Xu 1999) and use them for applications related to the water
management and update to intensity-duration-frequency (IDF) curves (i.e., Cheng and AghaKouchak 2014; Zhu 2013) or depth-duration-frequency (DDF) (Cook et al. 2017). There are, however, very limited studies that incorporate the regional models to assess the vulnerability of bridges to scour. A few studies have accounted for the uncertainties associated with climate change on scour risk of transportation infrastructure through development of qualitative tools with Hyrisk (Khelifa et al. 2013), use of hypothetical increase scenarios for flood (Kallias and Imam 2016), by characterizing the effects of uncertainty of bridge uncertainty on vulnerability to climate change (Dikanski et al. 2018), or by development of adaptation tools for different assets of the transportation system (Oswald and McNeil 2013). As Douglas et al. (2014) mentioned while the transportation agencies’ knowledge base is considerably advancing, the critical challenge is to cast this knowledge in a manner that allows the designers, asset managers, and transportation officials to modify planning, operation, and design guidelines to consider future climates. While there are a number of sources that provide the guidelines to integrate the climate data to design of adaptation strategies (e.g., Moss et al. 2013, Olsen 2015, Espinet et al. 2017), most are qualitative in nature.

This paper aims to address this gap by providing an adaptation design approach that takes into account the effects of climate change on the increased flooding of the rivers and consequent scouring at bridge piers. Because discharge rate at the bridge crossing directly influences the development of scour holes around bridge piers and abutments, the focus is in characterizing the flood-induced scour depth to evaluate the vulnerability of bridges and to provide a process for the design of adaptation measures. It should be mentioned that the goal of this paper is not to evaluate the process for generating the downscaled climate data or addressing the uncertainties associated with the emission scenarios, natural variability, or modeling assumptions as there has been much advancement within the climate science community to address these aspects (e.g., Mote et al. 2011,
The assumption here is that the downscaled data for the climate stressor of interest has been generated (here the climate stressor of interest is stream flow) and the stream flow regimes have been predicted based on the selected basin models. While this paper focuses on the detailed vulnerability analysis and countermeasure design of a bridge, it’s expected that that the approach could be used universally by the bridge designers and asset managers provided the availability of the climate change-induced stream flow data. The next sections of the paper review the available climate predictions and the impact on river flow estimation; the correlation between increased river flow and the vulnerability due to bridge scour; and alternative countermeasures and the design process, risk assessment due to scour, the cost and benefit analysis of countermeasures and its selection, and overview on conclusions, limitations, and future work.

4.2 Climate Change Impact on River Flow

In an effort to help the nation’s transportation agencies prepare for the continuing change in the frequency of occurrence of extreme weather conditions Federal Highway Administration (FHWA) launched the Climate Resilience Pilot Program in 2010 (FHWA 2016) with 19 pilots. In these projects FHWA partnered with state Departments of Transportation (DOT) and metropolitan planning organizations (MPOs) to identify the vulnerability of transportation assets to climate change and extreme weather events. As part of this effort, climate change predictions were generated as part of the pilot project for the state of Iowa where detailed analyses were conducted for two basins with short and moderate drainage times: Cedar River Basin and South Skunk River Basin (Anderson et al. 2015). The study by Iowa considered nine global climate models in conjunction with three greenhouse gas emissions scenarios (A1B, A1FI, and A2) to assess the increased frequency and intensity of precipitation. Fourteen (14) different scenarios were
generated which have formed the basis for the analyses in this paper. This paper utilizes the scenarios and the subsequent stream flow data generated by Anderson et al. (2015) to conduct the vulnerability assessment and the subsequent countermeasure design. The readers are encouraged to refer to the report by Anderson et al. (2015) for more details on the calibration of the models and the accuracy of the predictions.

The fourteen emission scenario-climate model pairs $E_iC_j$ \((i=1\ to\ 2, \ j=1\ to\ 7)\) are presented in Table 4.1. $E_i$ represents the emission scenario (A1B and A2 hereafter referred to as $E_1$ and $E_2$, respectively) and $C_j$ presents the climate model. This study performs the proposed analyses on a bridge that is part of the commercial industrial network, a designation system for primary highways that connect the regional growth areas and carries a significant amount of the commercial traffic, in state of Iowa. The bridge is located on one of the two basins that were the object of the climate change resilience project. The bridge has three spans with the middle section spanning over 38.10 m and outer two spans having a length of 29.72 m each. Two concrete wall piers support the superstructure. The seat type abutment and piers are skewed 20 degrees and are supported on battered wooden pile group foundations. Figure 4.1 shows the case study bridge.

According to the original bridge plans, at the time of its construction in 1964, the 100-year design discharge rate was 396 m$^3$/s and the overtopping discharge rate corresponding to 25-yr flood. Since 1964, the bridge has experienced four major flood events that exceeded either the 100-year or the 500-year design discharge rate. The historical peak annual discharge rate data were extracted from the USGS 5471000 gage station that is closest to the case study bridge from 1961 to 2015 except for the records for 1980-1990 when the gauge was not operational. Figure 4.2 displays the time history along with the historical flood events that have occurred in last four decades and the 100-year design discharge rate at the time of construction of the bridge. The
recorded discharge rate at the USGS 5471000 gage station during the 2010 flood event was 1025 m$^3$/s, which exceeded the 100-year design discharge rate by approximately 250%. While it’s likely that other exogenous factors such as land use and demand patterns have contributed to the increased demand over the life of the structure (Kilgore et al. 2016), the study by Anderson et al. (2015) did not account for these factors and limited the study to scenario pairs considered.

The $EiCj$ scenarios suggest future increase in the 100-year design (Table 4.2). Under such a high increase flood risk, bridge stakeholders need to design scour adaption measures considering the effect of climate on the design flow. The approach taken here involves i) identifying the impact of projected climate scenarios for the study area, ii) evaluating the effect of each model on the bridge scour and designing different countermeasures, iv) determining the costs for each adaption strategy and the losses avoided for cost-benefit analysis, and v) compiling and summarizing the results.

### 4.3 Scour Depth Estimation

Scour is a natural phenomenon caused by the erosion of sediment particles in the river and its banks. It is affected by various factors including the flow conditions, floodplain characteristics of the riverbed. The presence of obstruction in the streamflow also exacerbate the phenomenon at local level. According to Klinga and Alipour (2015), scour adversely affects the performance of bridges by decreasing the lateral capacity of the structure. Depending on the riverbed characteristics, three types of scour—aggradation and degradation, contraction scour, and local scour—can occur at the location of bridges crossing waterways. In this study, only contraction and local scour depths were estimated for the various peak discharge rates simulated.
Contraction scour can occur in live-bed and clear water conditions. For the site under consideration, a live-bed condition exists and the contraction scour depth, $Y_c$, is estimated using Equation 4.1 (Anerson et al. 2012).

$$Y_c = y_0 \left[ \left( \frac{W_1}{W_2} \right)^k - 1 \right]$$  \hspace{1cm} (4.1)

where $y_0$ is the existing water depth in the contracted section before scour (m); $W_1$ is the bottom width of the upstream main channel (m); $W_2$ is the bottom width of the upstream main channel in the contracted section less pier width (m); and $k$ is a factor which depends on the shear velocity and the fall velocity of $D_{50}$ materials (diameter of the smallest non-transportable particle in the bed material, m).

Based on the soil profile at the location of the case study bridge, Equation 4.2 suggested by Anerson et al. (2012) for cohesionless material was used to estimate flood-induced local scour depth, $Y_t$, for the projected annual peak flow discharge rates (EiCj scenarios).

$$\frac{Y_t}{a_{eff}} = 2.0k_1k_2k_3 \left( \frac{y_1}{a_{eff}} \right)^{0.35} Fr_1^{0.43}$$  \hspace{1cm} (4.2)

where $a_{eff}$ is the effective pile group foundation width; $y_1$ is flow depth upstream of the pier; $Fr_1$ is the Froude number directly upstream of the pier; $k_1$, $k_2$, and $k_3$ are correction factors for pier nose shape, angle of attack of the flow, and bed condition, respectively.

Flow depth, $y_1$, and mean flow velocity, $V$, can be calculated using basic hydraulic equations given the flow discharge rate, $Q$, and the shape of the channel at the location of the piers. Mean flow velocity is usually defined as the total flow discharge rate divided by the cross-sectional area of the channel. The relationship between the flow discharge rate and flow depth is given in Equation 4.3 for a channel with a rectangular cross-section.
where $b$ is the river width, $n$ is the Manning roughness coefficient, and $S$ is the energy slope.

Figure 4.3 presents the estimated scour depth for all of the 14 scenarios considered in this study with different return period values. The scour depth based on the 100-year design that was reported on the bridge plans and using equations 4.1-4.3 will be equal to 5.3 m. All climate scenarios in future, even with lower return periods, will yield larger scour depths at the bridge location.

Previous studies have proposed using an event-based adaptation model. The concept in that approach is that the adaptation research is being conducted but no action is taken for the bridge until the stressors reaches a critical threshold that could result in damage to the infrastructure. A 20% change in the stressor is recommended as the threshold (Larsen et al. 2008, Wright et al. 2012). The base line is the scour depth that the bridge was designed for at the time of construction which results in a 5.3 m scour depth (threshold of 6.36 m). One can observe that all scenarios with return periods of 200 and 500-year period will result in exceeded thresholds. For the 100-year flood that was the basis for the design for scour, all scenarios except for E1C5 and E2C5 result in higher scour depths than the threshold. Table 4.2 shows the 100-year flood discharge rate and associated scour depth at the bridge location.

4.4 Possible Scour Countermeasures

Different scour countermeasures are available and are used by the transportation agencies to alleviate sediment erosion at bridge foundations. The choice of the countermeasure depends largely on the extent of scouring, the location of the bridge, and availability of material and equipment. Scour protection methods are generally grouped into two categories subjected to the

\[ Q = \frac{by_1}{n} \left( \frac{by_1}{b+2y_1} \right)^{0.67} S^{0.5} \]
nature of protection: i) flow altering devices such as sacrificial piles (Melville and Hadfield 1999, Singh et al. 1995) and ii) armoring devices such as riprap, grouted riprap, cable-tied blocks, tetrapods, tetrahedrons, gabion mattresses (Lagasse 2007, Lagasse et al. 2006). It has been shown that flow-altering devices are not very effective when used alone as scour countermeasures (Parker et al. 1998). After review of the commonly used approaches by different transportation agencies (Alipour 2016), the present study evaluates three of the mostly used armoring devices as potential candidate for scour countermeasure design: i) gabion mattress, ii) grout-filled/cement bag, and iii) partially grouted riprap. In addition, the possibility of using geocontainers as scour countermeasure around bridge piers is explored. More details are presented in the following sections.

4.4.1 Gabion mattresses

Gabion mattresses are wire mesh containers filled with loose rocks, where units are interconnected to form a continuous layer. Compared to other armoring countermeasures, gabion mattresses can be considered as self-centered units in the sense that when instability occurs the units will mold themselves to the new geometry in the subgrade and restore stability. This added quality is due to the flexibility provided by the wire mesh. The performance of gabion mattresses depends on the sizing and filling of the basket quality and integrity of the wire, diaphragms, lids and lacing wire. The minimum thickness of the basket is 6 inches (Parker et al. 1998) and the minimum volume is computed as follows:

\[ V = \frac{0.069U^6K^6}{(S_s-1)^3g^3} \]  

(4.4)

where \( U \) is the approach design flow velocity, \( K \) is the coefficient for pier shape (1.5 for round-nose pier and 1.7 for rectangular), \( S_s \) is the specific gravity of the stone, and \( g \) is the gravitational acceleration.
The stone size, $d_{n50}$, is estimated using Equation (4.4) proposed by (Heibaum 2000). The selection of the stone size is an iterative process. It is also recommended that $d_{n50}$ be greater than or equal to 1.25 times the maximum spacing between wires and not to exceed 2/3 times the thickness of the basket.

$$d_{n50} = \frac{\mu}{(1-p)(S_s-1)} \cdot \frac{0.035 K_T K_Y V^2}{\Psi_{CR} K_s V^2 g}$$

(4.5)

where $\mu$ is the stability correction factor, $p$ is the porosity of the stone, $S_s$ is the relative density, $\psi_{CR}$ is the stability factor, $K_T$ is the turbulence factor, $K_Y$ is the depth factor, and $K_S$ is the slope factor, $V$ is the approach design flow velocity and $g$ is the gravitational acceleration. For gabion countermeasure, a loose fill is assumed and the quantity of stone was increased by 20%, following the recommendation by Lagasse et al. (2001).

### 4.4.2 Grout-filled bags

Grout filled bags are individual permeable synthetic sacks filled with concrete grout. They have gained popularity as bridge pier scour armor devices in the recent years due to their rapid deployment in addition to direct installation in erosion susceptible area around bridges (Garcia 2008). The effectiveness of grout-filled bags to protect bridge pier foundation depends on the size, placement, use of filter fabric or geotextile, tightness of the seal to the pier face and the lateral extent (Lagasse et al. 2007). Equation (4.6) suggested by Isbash (1936) is used to size the height of the bags.

Because grout-filled bags can be dragged away under flow field, engineering judgment is needed when selecting the adequate size for scour protection, while remaining stable under on coming flow (Lagasse et al. 2001). Research conducted by the Maryland State Highway Administration (Thornton 1996) recommended grout-filled bags size of 0.9 to 1.2 m wide, 1.2 m
long and approximately 0.3 m thick. Similar to other scour countermeasures, it is recommended that grout-filled bags be extended 1.5 to 2 times pier width and geotextile fabric is recommended as underlying material for installation to prevent the sinking of the bags in the bed material.

\[
D_{50} = \frac{V^2}{2g(S_g-1)E^2}
\]  

(4.6)

where \( V \) is the average approach flow velocity, \( S_g \) is the specific gravity of the stone, \( E \) is the Isbash’s coefficient and is 0.86 for bags that will not move at all and 1.2 for those that are able to roll slightly once installed.

### 4.4.3 Partially grouted riprap

Partially grouted riprap is an alternative for loose riprap where the required minimum stone size is not available. Grouting allows for the use of smaller rock sizes compared to standard riprap, as it creates a conglomerate, which improves the stability against particle erosion. The partially grouted riprap countermeasure consists of stone underlaid by geotextile fabric or granular filter. The median stone sizing follows that of loose riprap design. A coverage area that extends 1.5 times the pier width in all directions is recommended for optimum performance (Lagasse 2007). The minimum thickness suggested is twice \( d_{50} \). However, in the presence of contraction scour, this thickness is the largest between \( 2d_{50} \) and the contraction scour depth.

### 4.4.4 Geotextile containers

A cost effective scour countermeasure proposed by Pilarczk et al. (2000) consists of granular material and geotextile filter combined to form geocontainers. These geocontainers provide a twofold filter barrier when the geotextile cover is designed as a filter and the container is used to fill the scour hole. Nonwoven geotextile is recommended because it combines a good filtration capacity, a high angle of friction, and the capability to withstand impact forces (Pilarczk
et al. 2000). Because they act as filter, there should not be a gap between the individual containers. Lagasse et al. (2001) present the design and installation guidelines for geocontainers.

In addition to the conventional methods discussed above, an alternate design is presented that utilized geocontainer in conjunction with the gabion mattress, grout-filled/cement bags, and partially grouted riprap and thereafter refers as Alt 1, Alt 2, and Alt 3, respectively. Rather than filling the entire scour hole with one type of material, we propose to fill up to the local scour depth with the geocontainers (Figure 4.3). The geocontainers used in the present study are filled to 70% of their theoretical volume, following the recommendation from the Red Eye Crossing Project (Parker et al. 1998). A Class II stone was selected for partially grouted riprap and gabion mattress countermeasures with a safety factor of 1.5.

4.5 Conventional and Alternative Countermeasure Cost Estimates

Itemized costs are obtained from numerous sources including past bid from state DOTs. The unit price for the riprap stone is $35.5/ton (Apaydin 2010), the cost of excavation of soil around the foundation and cement grout were $9.353/m³, $9.027/m³, respectively (Iowa DOT). The gabion size considered is 3×1×1 m³ with a unit price of $109.61. The grout-filled/cement bag is 0.9×1.2×0.3 (width, length, and thickness) and costs $4.634 each, while the geocontainers fill is $21.46/m³ per unit (Lagasse et al. 2001). According to Abboud and Kaiser (2012), gabion mattresses and partially grouted riprap require maintenance every 10 years for the remaining useful life of the bridge. During these maintenance cycles, 5% of the geotextile filter and grout will be replaced for the partially grouted riprap and 15% of the gabion mattresses will be replaced. The maintenance cost is estimated as follows (Abboud and Kaiser 2012):

\[
Cost_{Maint.} = \frac{Y_0}{(1+\frac{i}{100})^N}
\] (4.7)
where $Y_0$ is the cost of material needed for the first inspection, $i$ is the discount rate (=1.25), and $N$ is the recurring maintenance period. When estimating the cost of the countermeasures, a return period of 30 years was assumed for the replacement of the countermeasures. Figure 4.4 presents the cost estimates for all of the conventional and alternative countermeasures for the period of 2016-2070 across the 14 climate projection scenarios and for different return periods.

As can be seen from this graph, the economic analysis of conventional method versus the alternative method shows that higher financial gain can be achieved using the alternative approach. Grout/filled bags is the most cost-effective mitigation strategy when traditional approach of using scour countermeasure is considered. Furthermore, when these conventional measures were combined with geocontainers, the cost of the scour measures further reduces making the alternative countermeasures more attractive mitigation strategy. When compared to the cost of the traditional method, Alt 2 resulted in a cost reduction of two third the cost of the scour countermeasure, while a reduction Alt 1 and Alt 3 cost is 2.5 time smaller than that of the conventional approach. Figure 4.6 shows the estimated cost per countermeasure, averaged across the spectrum 14 considered climate projection scenarios, with associated standard deviations.

### 4.6 Risk Evaluation and Benefit Estimation

To analyze the true benefits of implementation of any countermeasures one should consider the losses avoided due to bridge damage or failure and disruption to the functionality of the transportation system. To account for the impact of multiple bridge failures on network-level performance measures, Alipour and Shafei (2015 and 2016) have used flow-based network-level analysis with dynamic traffic assignment modeling. Considering that only one bridge is used this study the proposed framework, the loss assessment was based on the methodology provided by HYRISK, a model developed by the FHWA in the late 1990s and modified in 2006 (Khelifa et al. [2019]).
2013) has been adopted. This estimate includes the economic loss model of rebuilding the bridge and road user costs. The risk of keeping the bridge in service without taking any measures to decrease its failure, expressed as the annual monetary value, is estimated using Equation 4.9 (Stein and Sedmera 2006).

\[
C_{total} = (C_0 + C_1)eWL + \left[ C_2 \left( 1 - \frac{T}{100} \right) + C_3 \frac{T}{100} \right]DAd + \left[ C_4 O \left( 1 - \frac{T}{100} \right) + C_5 \frac{T}{100} \right] \frac{DAd}{S} \tag{4.8}
\]

\[
Risk = K_1 K_2 P_A C_{total} \tag{4.9}
\]

where the first statement represents the bridge restoration/rebuilding costs and in it \( C_{total} \) is the total cost of failure; \( C_0 \) is the removal cost (=\$7/ft\(^2\)); \( C_1 \) unit building cost(=\$130/ft\(^2\)); \( e \) is the cost multiplier for early replacement based on ADT (=1.5); \( W \) is the bridge width; \( L \) is the bridge length. The second term represents the vehicle running costs, in which: \( C_2 \) is cost of running automobile (=0.1485/mile); \( T \) is the average daily truck traffic expressed as fraction of ADT (7.12); \( D \) is the detour length (=8 miles for the bridge in the study); \( d \) is duration of the detour based on ADT (=90 days); and \( C_3 \) is the cost of running truck (=\$0.429/mile). The last term is the time loss costs or opportunity losses, in which: \( C_4 \) is the value of time per adult in passenger car (=\$6.31/hr); \( O \) is the average occupancy (=1.63 usually); \( C_5 \) is the value of time for truck (=\$22.01/hr); \( S \) is the average detour speed (=35 mph); \( K_1 \) is an adjustment factor for the bridge span length; \( K_2 \) is the risk adjustment factor for the foundation type; \( P_A \) is the annual probability of failure that is calculated as \(4.33 \times 10^{-2}\) for the case study bridge. The values of \( K_1 \) and \( K_2 \) are taken as 0.67 and 0.8, respectively (Stein and Sedmera 2006).

In this study, it is assumed that the loss of one pier results in the collapse of the approach span and the middle spans. This is an acceptable assumption following the findings from a series of surveys conducted from the US DOTs by Alipour (2016). The reconstruction cost of the bridge,
based on bridge length \((L=217.5 \text{ ft})\), is $1,340,888. The estimated total cost of the bridge failure due to scour is $6,688,606 with an annual risk of $155,286/year. The annual risk is high compared to the rebuilding cost of the bridge. Therefore, the implementation of a countermeasure is warranted. The selection and design of a countermeasure depends on the level of risk a bridge owner is willing to take. This risk is often represented as the cost of failure of the bridge and is compared with the benefit of implementing the countermeasure for the remaining life of the bridge. Another approach of selecting a countermeasure is by comparing the benefit/cost ratios or net benefit analysis.

The case study bridge was built in 1964 and has a remaining useful life of 23 years, assuming a design life of 75 years. For this remaining life, it is important to select a countermeasure that will yield a high return over time. The minimum return period of the countermeasure is the reciprocal of the annual probability of failure of the unprotected bridge. For the case study bridge, the minimum return period is 23 years. Any countermeasure with a return period greater than or equal to the remaining useful life of the bridge will help decrease the annual probability of failure of the bridge. The probability of failure over the expected life of the protected bridge, \(P'_L\), is estimated as follow:

\[
P'_L = 1 - \left(1 - \frac{1}{RT}\right)^L
\]

(4.10)

where \(RT\) is the protection return period, and \(L\) the remaining life of the bridge.

It was assumed that following the implementation of a scour countermeasure, the failure probability of the bridge is improved at least for the design life of the countermeasure. The lifetime probability of failure, \(P_L\), is estimated as follows (Stein and Sedmera 2006):

\[
P_L = 1 - (1 - P_A)^L
\]

(4.11)
The present benefit value can be estimated as follows (Stein and Sedmera 2006):

\[ B = (P_L - P_L')\text{Cost} \]  \hspace{1cm} (4.12)

Table 4.3 presents the comparative analysis of the protection level of the bridge and its benefit. As seen from this table, designing the countermeasure with returning period equal to the remaining life of the bridge (23 years) does not yield any return. As the protection level is increasing the failure probability of the bridge is decreasing while the benefit is increasing over the protected life of the bridge.

The analysis presented up to this point will assist bridge owners to decide the course of action to take in preparing for the uncertainty about future climate conditions by selecting adaptation strategies to reduce the adverse impacts of projected increase in the frequency and intensity of precipitation on the bridges. When evaluating all available options for adapting to climate change, a decision must be made on whether to protect the bridge for its remaining life or to reduce the risk associated with failure. In addition, due to the financial constraints, transportation stockholders need to: i) minimize the design return period to balance the cost of the countermeasure with the risks of keeping the bridge in service; ii) investigate the countermeasure design that will result in the highest net benefit, and iii) identify the return period that will yield the maximum benefit/cost ratio.

Depending on the bridge classification, they are designed to withstand the adverse effect of a flood having a return period of 25, 50, 100, 200, or 500. As noted earlier, the increase in precipitation observed over the past decades may result in the decrease of the design return period. For example, a previous 100-year design flood may now occur once every 50 years (Douglas et al. 2017). The design of many bridges in service to date was based on hydrologic conditions that
differ from the current condition. As such, in order to maintain the safety of the public they must be protected against potential scour failure. Furthermore, because design discharge rate affects the estimated scour depth, a sensitivity analysis is conducted to determine the design flood that bridge owners can expect a return on investment. This analysis is presented to evaluate the level of protection by considering flood events with return period 25, 50, 100, and 200 years for the period of 2016 to 2070. For this time frame, the estimated design floods and the associated total scour depths are shown in Table 4.5 through 4.8. The vulnerability analysis shows that the estimated design discharge rates has exceeded the aforementioned vulnerability threshold for all four AEPs (4%, 2%, 1%, and 0.5%) for E1C6 emission pair (Figure 4.5).

The benefit/cost ratios for the alternative countermeasures are displayed in Figure 4.6 along with the estimated net benefit values for design discharge rates with return period of 25-, 50-, 100-, and 200-year. In this ratio the cost is associated with the countermeasure not that of the bridge failure. As can be seen from this figure, the difference in the net benefit is not clearly visible between the countermeasures. However, this difference is depicted in the benefit/cost ratio. For each design flood, the Alt 2 has the highest ratio and the flood with 4% AEP yield the maximum benefit/cost ratio.

4.7 Conclusions

The increase in frequency of intense precipitation events and the resulting floods underscores the danger posed by climate change to many transportation assets including bridges. With more than 70% of the nation’s bridges built at a time were intense precipitations are uncommon, the consequences of inaction could result in significant economic losses. Despite the major recent advancement in projecting the climate change impacts on the future precipitation regimes, there is a gap in the procedural approaches to account for them while designing new
structure or mitigating the risk in the existing infrastructure. This paper presents an approach to integrate climate change in the bridge maintenance program. Due to the uncertainty in the future, a predict-then-act approach is presented that is based on the evaluation various economic developments and their impacts on the global water circulation. Particular attention was given to the effect of climate change on the annual peak discharge rate. By relating the discharge rate to the scour depth, a vulnerability assessment is performed by using estimates based on future observations. The study used the readily available climate projection scenarios from the FHWA climate change resilience pilot project for the state of Iowa.

The probability of failure of bridges increases as scour hole forms around bridge piers over. It is expected that the implementation of scour countermeasure will decrease the failure probability over the extended life of the bridge until replacement is scheduled. The cost of such implementation of the countermeasure outweighs the economic losses associated with the bridge failure. Furthermore, when the remaining life of the bridge is taken into consideration during the planning process, a decision has to be made on the design return period of the countermeasure and the return period of the design discharge rate. Considering the increasing financial constraints on state DOTs, thus must prioritize projects, an economically attractive alternative to the conventional countermeasures was introduced which utilized the conventional measure in conjunction with geocontainers. Through a risk assessment approach the true benefit of the countermeasure was derived aimed at identifying the return period of the countermeasure and a benefit/cost ratio was used in selecting the design discharge rate.

With many regions in the U.S. projected to experience an increase in frequency and duration of heavy rain, the results of the analysis indicate that there is an increasing risk of flood-induced failure of bridges and the following economic losses. There is an unyielding need of
providing a decision making approach that would result in cost effective mitigation strategies to offset the effect of climate change on bridges crossing over waterways. These mitigation strategies include providing flood and scour countermeasures at bridge where failure would result in economic loss and loss of human life. Although the study focuses on a case study bridge, the procedure for risk assessment, countermeasure design, and benefit/cost analysis can be adopted for other bridges. The data requirements for application of the study to other bridges in other regions would be the availability of the climate projections for precipitation and the river basin analysis for the evaluation of the discharge rate, the characteristics of the bridge and riverbed, which should be available through plans or the latest inspection data. It should be mentioned that the selection of particular scour countermeasure depends on the local preferences, and its design and implementation depends on the extent to which bridge owners are willing to invest for the protection against scour-induced failure and the safety of the public. A more realistic and holistic approach for assessment of the indirect costs (due to the downtime in the functionality of the system) could be achieved through development of network models of all the bridges within a specific basin or regions exposed to the increased precipitation.

Acknowledgements

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References


### Table 4.1. Climate change scenarios and the assigned nametags in this study

<table>
<thead>
<tr>
<th>Climate Model</th>
<th>A1B (E1)</th>
<th>A2 (E2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CGCM3_T47 (C1)</td>
<td>E1C1</td>
<td>E2C1</td>
</tr>
<tr>
<td>CGCM2_T63 (C2)</td>
<td>E1C2</td>
<td>E2C2</td>
</tr>
<tr>
<td>CNRM (C3)</td>
<td>E1C3</td>
<td>E2C3</td>
</tr>
<tr>
<td>ECHAM5 (C4)</td>
<td>E1C4</td>
<td>E2C4</td>
</tr>
<tr>
<td>ECHO (C5)</td>
<td>E1C5</td>
<td>E2C5</td>
</tr>
<tr>
<td>HADCM3 (C6)</td>
<td>E1C6</td>
<td>E2C6</td>
</tr>
<tr>
<td>HADGEM (C7)</td>
<td>E1C7</td>
<td>E2C7</td>
</tr>
</tbody>
</table>

*Italic shows the nomenclatures in this paper. For instance, E2C1 is representative of A1FI-CCSM
CGCM3-T47: Coupled General Circular Model third generation version T47
CGCM2-T63: Coupled General Circular Model second generation version T63
CNRM: Centre National de Recherches Météorologiques
HADEM3: Hadley Center coupled Model version 3
HADGEM: Hadley Center Global Environmental Model
ECHO-5 is a model developed by Meteorological Institute of the University of Bonn (Germany), Institute of KMA (Korea), and Model and Data Group

### Table 4.2. Total scour depth for the period of 2016 to 2070

<table>
<thead>
<tr>
<th>Scenario</th>
<th>1% AEP flood (m³/s)</th>
<th>Total Scour (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Baseline</td>
<td>1197</td>
<td>7.96</td>
</tr>
<tr>
<td>E1C1</td>
<td>1330</td>
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<td>E1C2</td>
<td>724</td>
<td>6.51</td>
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<tr>
<td>E1C3</td>
<td>1323</td>
<td>8.32</td>
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<tr>
<td>E1C4</td>
<td>814</td>
<td>6.80</td>
</tr>
<tr>
<td>E1C5</td>
<td>633</td>
<td>6.20</td>
</tr>
<tr>
<td>E1C6</td>
<td>1716</td>
<td>9.38</td>
</tr>
<tr>
<td>E1C7</td>
<td>1463</td>
<td>8.70</td>
</tr>
<tr>
<td>E2C1</td>
<td>823</td>
<td>6.83</td>
</tr>
<tr>
<td>E2C2</td>
<td>943</td>
<td>7.20</td>
</tr>
<tr>
<td>E2C3</td>
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</tr>
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<td>6.37</td>
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<td>E2C6</td>
<td>638</td>
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<tr>
<td>E2C7</td>
<td>1463</td>
<td>8.71</td>
</tr>
</tbody>
</table>
Table 4.3. Analysis of benefits for the scour countermeasure return period

<table>
<thead>
<tr>
<th>Protection Return period (years)</th>
<th>Probability of Failure, P’l. (%)</th>
<th>Protection Benefit</th>
</tr>
</thead>
<tbody>
<tr>
<td>23</td>
<td>64</td>
<td>$-9,497</td>
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<tr>
<td>25</td>
<td>61</td>
<td>$200,000</td>
</tr>
<tr>
<td>30</td>
<td>54</td>
<td>$651,292</td>
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Table 4.4. Economic analysis of proposed countermeasures with 30 years return period

<table>
<thead>
<tr>
<th>Design flood return period</th>
<th>Cost (×$1000)</th>
<th>Net Benefit (×$1000)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Alt1</td>
<td>Alt2</td>
</tr>
<tr>
<td>25-year</td>
<td>49</td>
<td>21</td>
</tr>
<tr>
<td>50-year</td>
<td>58</td>
<td>24</td>
</tr>
<tr>
<td>100-year</td>
<td>68</td>
<td>28</td>
</tr>
<tr>
<td>200-year</td>
<td>32</td>
<td>37</td>
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</table>
Table 4.5. Discharge and scour depth for flood with 25-year return period

<table>
<thead>
<tr>
<th>Scenario</th>
<th>0.04% AEP flood (m³/s)</th>
<th>Total Scour (m)</th>
</tr>
</thead>
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<tr>
<td>E1C1</td>
<td>811</td>
<td>6.79</td>
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<td>E1C2</td>
<td>531</td>
<td>5.83</td>
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<tr>
<td>E1C3</td>
<td>861</td>
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<td>E1C5</td>
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<td>631</td>
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<td>E2C4</td>
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<tr>
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<td>E2C6</td>
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<tr>
<td>E2C7</td>
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<td>6.75</td>
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</table>

Table 4.6. Discharge and scour depth for flood with 50-year return period

<table>
<thead>
<tr>
<th>Scenario</th>
<th>0.02% AEP flood (m³/s)</th>
<th>Total Scour (m)</th>
</tr>
</thead>
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<tr>
<td>E1C1</td>
<td>1056</td>
<td>7.55</td>
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<td>E1C2</td>
<td>627</td>
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<tr>
<td>E2C7</td>
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</table>
### Table 4.7. Discharge and scour depth for flood with 200-year return period

<table>
<thead>
<tr>
<th>Scenario</th>
<th>0.005% AEP flood (m³/s)</th>
<th>Total Scour (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>E1C1</td>
<td>1633</td>
<td>9.16</td>
</tr>
<tr>
<td>E1C2</td>
<td>824</td>
<td>6.83</td>
</tr>
<tr>
<td>E1C3</td>
<td>1582</td>
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<td>981</td>
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<td>754</td>
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<tr>
<td>E1C6</td>
<td>2146</td>
<td>10.43</td>
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<td>E2C7</td>
<td>1881</td>
<td>9.79</td>
</tr>
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</table>

### Table 4.8. Discharge and scour depth for flood with 500-year return period

<table>
<thead>
<tr>
<th>Scenario</th>
<th>0.002% AEP flood (m³/s)</th>
<th>Total Scour (m)</th>
</tr>
</thead>
<tbody>
<tr>
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<td>2080</td>
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<td>7.92</td>
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<td>E2C7</td>
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Figure 4.1. US-30 Bridge over South Skunk River (Courtesy of Lu 2008).

Figure 4.2. Peak annual discharge rate time history
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CHAPTER 5. ROBUST CLIMATE ADAPTATION MEASURE SELECTION FOR HIGHWAY BRIDGES BASED ON SCENARIO PLANNINGS

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Abstract

The projected change in the hydrologic regime of rivers raises concerns about the stability failure of bridges with adverse consequences such as a disruption in traffic and significant socio-economic loss. Designing or adapting transportation infrastructures to the current and future conditions will require a collaborative effort between transportation assets managers, engineers, and climate scientists. While mitigation strategies have been the course of action in the transportation system in the presence of uncertainty, the magnitude of future climate change impacts calls for exploring new avenues for coping with the ever changing climate. Furthermore, face with budget constraint and the uncertainty in future climate, transportation decision-makers need resilient tools that will help them make economically sound choices in adapting the infrastructures to the new environmental conditions. This paper develops a new methodology for selecting and designing adaptive measure based on low regret and robust decision making through a scenario planning approach. The proposed methodology is demonstrated on a case study bridge.
5.1 Introduction

The design of highway bridge foundation is based on the criterion that the foundation should stand the adverse effect of rare flood events. Such system is designed to accommodate for moderate to small change in the conditions. However, significant changes in the design floods can undermine their stability and safety of the users. Additionally, due to the interconnectivity of the transportation modes, disturbance and broken links may have impacts that extend beyond the transportation system. Recent disruptions in the transportation network due flooding events highlights what will become frequent in the future. As such preparing infrastructure to cope with future climate will not only enhance their resiliency but also reduce future repair costs.

In the United States (U.S.), a 100-year return flood is used for highway bridge foundation design. Furthermore, the foundation is expected to maintain its stability under a flood event with a return period of 500-year. Meeting these performance conditions requires an accurate estimation of the design floods, which have an implication on the structural elements selection. For example, the length of a pile foundation is sized to account for the projected scour depth, which itself is dependent on the design discharge rate. These estimates are often based on historical data so that future damage resulting from events of equal intensity or size observed in past could be prevented. However, flood events with such long return periods require the availability of sufficient recorded data that may not be available at the time of design. In addition, current design flood estimations are based on probabilistic formulations that do not account for changing climate. With the changes in precipitation observed in the past few years, scientists have argued that climate change is altering the global water circulation and that stationary assumption on the climate is no longer a valid assumption (Villarini and Smith 2010, Vogel et al 2011).
Climate variability and change influence transportation primarily through changes in water-related extreme events. Many studies have been devoted to quantify the manifestations of climate change. These studies suggested an increase in the atmospheric pressure, changes in the precipitation trends, intensity and frequency; change in soil moisture; increase in the average global temperatures; sea level rise; and early spring snow melting. According to the Intergovernmental Panel on Climate Change (IPCC), the frequency of heavy precipitation is expected to increase in the future (IPCC 2014). These changes will result in the modification of river discharges and the frequency of extreme events (Brekke et al. 2009, Kundzewicz et al. 2008). Therefore, it is important to reassess the design conditions as more data becomes available.

Existing infrastructure design standards may not be adequate for handling possible future precipitation intensity, duration, and frequency (Moglen and Rios Vidal 2014, Mailhot and Duchesne 2010). As such, climate change will drive infrastructure to operate in conditions that they were not originally designed for (Douglas et al. 2017). Infrastructure that is not designed to sustain future climate conditions will require implementation of adaptive measures, which in many cases can result in a substantial additional cost. Considering future climate conditions during the design process or during maintenance scheduling could limit the impacts of climate change and also reduce the costs of upgrade substantially (EEA 2014). Transportation policymakers at local, regional, and state level have begun reevaluating current design, maintenance, and operational guidelines in order to assess the vulnerability of the transportation system considering the dynamic nature of the climate.

The challenge in creating and implementing adaptation measures is the high level of uncertainties associated with climate. One approach in dealing with such level of uncertainty is to develop and implement robust solutions that would be effective and efficient under a range of
climate change scenarios (EEA 2014). The objective of this study is to provide transportation decision makers with adaptation strategies in dealing with uncertainties in the climate. With all the evidence showing that previous historical floods may become frequent, highway bridges will be subjected to hydraulic loading conditions that exceed their design criterion. The exceedance of these design floods may result in temporary service interruption on these bridges or in worst-case scenario their failure.

The magnitude of climate change and associated direct and indirect socio-economic impact suggests potential benefits from exploring innovative options (EEA 2014). The aim of the present study is to propose a framework that alleviates pressures on transportation decision makers faced with the uncertainties in the climate change. This framework starts by identifying the threats posed by climate change to an individual project, then evaluates all possible adaptation measures. This project could be a new design, an upgrade, or a replacement of existing infrastructure. Since bridges replacement scheduling occur toward the end of their useful life, thus, it is important to include the remaining life of the structure in the adaptation planning process. Because it may take decades for change in the climate to be observable, the decision to adapt bridge infrastructures based on current observable data or future prediction of the climate will be based on its remaining life. In the present study, 10-year is used as reference for using future climate predictions. In this regard, if the remaining life is equal to or less than 10 years, we proposed using current observations to design the adaptive measure. However, if the remaining life extends beyond a decade, then adaptation measures design should be based on future climate scenarios. Furthermore, the selection of the optimum measure should be based on the measure that performed well under current and plausible future conditions. The proposed framework is summarized in Figure 5.1 and demonstrated using US 30 bridge located over Skunk River in Ames, Iowa.
5.2 Significance

Change in the climate is unevenly distributed through the world and its impacts will depend on the geographic location and local climate. It is projected to affect the economic, social, political, and environmental sectors in every region of the world (IPCC 2014a). The impacts of climate change are intertwined with socio-economic trends like demographic change, changes in production pattern and lifestyles leading to altered transport demand (EEA 2014).

Climate change manifests through various ways in different locations due the complex nature of climatologic interactions that contribute to regional climates (Brekkle et al. 2009). Many climate models suggest an increase in the global temperatures with an average increase between 1 and 6°C by 2100. The Intergovernmental Panel on Climate Change (IPCC 2007) attributed these changes to the increase in greenhouse gases (GHG) emissions in the atmosphere. As such, surface transportation represents one of the main contributors to this increase and transportation infrastructures are expected to be significantly impacted by these changes. The amount of scientific evidence and understanding of climate change clearly substantiate taking steps in reducing the amount of greenhouse gas emissions in the atmosphere. In the fourth assessment report, IPCC recommended that by the year 2050 the emission of GHG in the atmosphere should corresponds to 50-80% of the year 2000 emission level to avoid serious consequences (Pachauri and Reisinger 2008).

Future anthropogenic emission of GHG depends on future social and economic development, land use, population growth and change in technological innovations. However, these factors are difficult to predict and highly uncertain (EEA 2014). The source of these uncertainties are aleatory and/or epistemic and there are several ways to account for these uncertainties. For climate change, these uncertainties reside in the climate models and their basic
assumptions. Designing and planning of infrastructures are accomplished at regional and local levels, however, uncertainty is much higher at these levels. Consequently, there is a resistance for including the effects of climate change in the maintenance, planning, and designing of transportation infrastructures. This resistance is also due to the gap of understanding the effects of climate change on the precipitation and frequency of exceedances between climatologists and civil engineers (Bonnin et al. 2011).

The impacts of climate change on the transport system in the future face several uncertainties. Learning from recent extreme events in addition to expanding the knowledge base and improving the availability of information can help reduce some of these uncertainties. EEA (2014) suggests that increasing collaboration among transportation and climate experts can help develop improve climate models and increase the use of their outputs in connection with transportation infrastructure design and operations. To bridge the gap between climatologists and civil engineers, a collaborative effort is necessary for characterizing and quantifying uncertainty in future climate (ASCE 2015). The results of such alliances is the awareness of the unattended consequences of climate in planning and designing of infrastructures.

Transportation sector is one of the main contributors to GHG emissions in the atmosphere and so far mitigation strategies have been the course of actions taken in dealing with climate change (Susskind 2010; Pielke 2010). However, even if global emissions were to be stopped completely today, the effects of anthropogenic emission of GHG will continue to affect the climate system for decades. New extreme events will continue to occur as a result of the already existing variables in the climate (Bhatkoti et al. 2015). As highlighted by Milly et al. (2008) and Susskind (2010), even aggressive mitigation could not reverse the trend of the impact of climate change.
Thus, adaptation to the current climate conditions as well as to plausible future climate scenarios is, therefore, consequential.

5.3 Planning for Future Climate Scenario

The USGS has gage stations across the U.S. to monitor the discharge rate and the water depth in rivers. The design of new infrastructures uses the historical records at these gage stations to estimate the expected design floods with specific return periods, following the guidelines put forth by the hydrology subcommittee of the Interagency Advisory Committee on Water Data (IACWD 1982). This guideline uses a probabilistic approach with a stationary assumption on the climate to estimate future extreme recurring flood events. However, damage to infrastructures observed in the past decades suggests that current extreme might be higher than projected due to the climate change.

With all the evidence on the increase in frequency of extreme events, transportation assets managers are faced with making hard decisions. They can decide to do nothing or take action to reduce the expected negative impacts by implementing countermeasures. The implementation of these measures should start with assessing the risks posed by climate change. However, changes that might occur in the future are deeply uncertain. Lampert and Collins (2007) defined deep uncertainty as conditions were decision makers do not know or cannot agree upon the system model relating actions to consequences or the prior probabilities of key parameters of the system model. Many areas from water resource management to counterterrorism are faced with deep uncertainty and new tools and methods have been developed, including scenario planning.

According to Van Der Heijden (2005), scenario planning is an alternative to traditional modelling and forecasting capable of considering multiple conditional assumptions. The method originated from the Royal Dutch Shell in 1970 and has since been utilized in many area including
business’ face with unpredictable future (Ringland 2002). Rather than focusing on a single future prediction, the method explores all plausible future alternatives with a particular attention given to uncertain drivers. Because each scenario represents a realization of a plausible future, a carefully constructed set of scenarios can emphasize future risk and opportunities, thus, providing managers with the information needed to assess the effectiveness of alternative strategies (Susskind 2010).

The first step in scenario planning is to identify the drivers. The anthropogenic GHG emission have been identified as one of the drivers of the climate change. The IPCC Special Report on Emission Scenario defined four emission scenarios (A1, A2, B1, and B2) that explore alternative development pathways and resulting GHG emissions. Two types of model are often used for climate simulation: Earth System Models (ESM) and Global Climate Models (GCM). The latter models are commonly used to determine the impacts of climate. The combination of the GCM and GHG emission scenario will result in a climate pair for the variable under consideration, here precipitation, at an anticipated year and the corresponding river peak discharge rate. The combination of the GCM and GHG will result in a set of scenarios for which various strategies can be developed for the unforeseen future.

5.4 Identification of the Climate Change Threat to Infrastructures

According to the special report on emission (TRB 290 2008), the increase in temperature, intense precipitation events, intense hurricanes, drought, and rising of sea levels will affect transportation primarily through increases in several types of weather and climate extremes. These changes will have both negative and positive effects on the transportation systems. For infrastructures located on the coastal areas, sea level rise and major storms represent serious risks to road infrastructures, while riverine flooding represents a threat for transportation infrastructures in the Midwest and Great Lake regions. The projected increase in temperature in winter and
summer, in addition to earlier spring snow melting pose the risk of flooding in these regions. For land transportation assets, this raises the concern regarding damage or failure of infrastructures due to extreme flooding.

Over the past decade, many studies have been directed toward the evaluation of the effects of climate change on river flows and their hydrological regimes (Tebaldi et al., 2006; Groisman et al., 2001; Lins and Slack 1999; Frumhoff et al. 2007, Melillo et al., 2014). All these studies indicate, with high confidence, an increase in the heavy precipitation and the duration of extreme precipitation events across the United States. Nearly 500,000 bridges in the U.S. are located over streams and rivers. Wardhana and Hadipriono (2003) surveyed 503 bridge failures cases between 1989 and 2000 to conclude that hydraulic failure represents the number one cause of bridge failure in the U.S. with flood and scour accounting for 53% of the hydraulic related failures. A report on the 1993 Upper Mississippi River basin flooding concluded that the widespread damages and collapses observed are the results of scouring around bridge abutments and piers and debris accumulation (NCHRP Report 417). As such, flood induced-scour represents serious threat for the transportation highway bridge infrastructure.

5.5 Adaptive Inventory of Strategies and Cost Estimations

Different adaption strategies should be considered for different implications of climate change. For example, for increasing temperature, new designs for pavements are required while the increase in precipitation may entail increasing the free board elevation of bridges. The focus of this paper will be on the impact of increased precipitation on the flood-induced scour. Depending on the extent of scour, the location of the bridge, and availability of material and equipment, different scour countermeasures are proposed for the bridge foundations. Scour protection methods are generally grouped into two categories depending on the nature of protection
they provide: i) flow altering devices such as sacrificial piles (Melville and Hadfield 1999, Singh et al. 1995) and ii) armoring devices such riprap, grouted riprap, cable-tied blocks, tetrapods, tetrahedrons, gabion mattresses (Lagasse et al. 2006, 2007).

It has been proven that flow altering devices are not very effective when used alone as scour countermeasures (Parker et al. 1998). As such, the present study focuses on armoring devices as the preferred technique to reduce the scour development or increase the resistance against the erosive action on the riverbed. Scour countermeasures considered are partially grouted riprap (M1), gabion mattress (M2), and grout filled/cement bags (M3) used in conjunction with sand filled geocontainers. Previous study by the authors showed that this alternative is cost effective compared to the traditional approach. The cost estimate of these countermeasures will be based on the material and maintenance costs at present value.

5.6 Risk Evaluation

While there is a considerable evidence that the climate is changing, understanding the significance of climate change at temporal and spatial scales relevant to engineering practice is more difficult (ASCE 2015). Although significant progress has been made over the past decade in modeling climate, still, existing models cannot predict with certitude future climate trends. This uncertainty complicates the understanding of the true nature of the problem facing transportation infrastructures. To help evaluate the risks, a matrix of multiple scenarios is constructed by considering X future climates, which are referred to herein as observed climates $O_i$ ($i=1, 2, \ldots, n$) and Y climates assumed in designing the adaptive measures refer to as design climates, $D_j$ ($j=1, 2, \ldots, m$). The following $m$ by $n$ matrix can be used to describe the interaction between the design and the observed climates.
\[
\begin{array}{c|ccc}
O_1 & \cdots & O_n \\
\hline
D_1 & r_{11} & \cdots & r_{1n} \\
\vdots & \ddots & \ddots & \vdots \\
D_m & r_{m1} & \cdots & r_{mn} \\
\end{array}
\]

where \( r_{ij} \) represents the variable used to evaluate the performance of the design over the observed climates.

Figure 5.2 shows a three dimensional view of the climate scenario planning and the adaptive measures. Depending on the level of risk that the decision makers are willing to take, the design of the adaptive measures may be based on: i) minimizing the cost by designing for the least aggressive climate pair, ii) maximizing cost by considering the most aggressive climate pair, or iii) designing for a target design flow rate value, which can then be translated into cost over the regret option to define the interval over which the performance of the design may be evaluated.

Faced with the uncertain nature of climate, ASCE (2015) suggests that engineering adaptive strategies should be based on low-regret decisions to make the project more resilient to future climate and weather extremes. This approach also has a business justification with the recent movement towards designing bridges for a 100-year design life. In this study, four levels of alternative regret measures that decision makers may wish to explore are considered for the evaluation of the countermeasure design. They are: no regret (NR), small regret (SR), acceptable regret (AR), and unacceptable regret (UR). Here, regret represents the change in cost of the adaptation measure designed based on the least aggressive climate and the observed climate. This regret is then compared with conditions where design conditions are identical to the observed conditions. This comparison is expressed as a ratio (Equation 5.1), which gives a quantitative measure to understand the future condition and design for it.
\[ r_{i,j}^l = \frac{\text{Cost}_{i,j}^l - \min\text{Cost}_{j}}{\text{Cost}_{k,k}^l} \] (5.1)

The first term in the numerator of the above equation is the cost for designing the adaptive measure for climate pair \( j \), the second term is the minimum cost of the adaptive measure, \( l \), for all design climate pairs, \( j \), \( i \) represents the observed climate, and \( k \) is representative of the diagonal elements of the matrix.

For a decision involving uncertainty, such as climate, decision tools have been developed that explore multiple alternatives to help decision makers identify actions and outcomes. These actions are evaluated using methods such as the optimization or robustness. The optimum solution approach may yield the best solution when the uncertainty is well characterized, the cause-effect relationships are well understood, and the values are clear (Lempert and Collins 2007). However, according to Dessai et al. (2004), future climate projections, impacts, and adaptive responses are fraught with large uncertainties. Matalas (1997) recommended a robustness decision criterion for selecting the appropriate design for a likely future. Hall et al. (2012) conducted a comparative study between info-gap and robust decision-making methods for climate policies to conclude that although the two methods share many similarities with the underlying objective of finding strategies that are insensitive to uncertainties, they do not result in the same policy recommendations. Lempert and Collins (2007) studied lake pollution by employing the optimum utility, precautionary, and robust decision approaches to conclude that, under deep uncertainty, robust decision-making is the preferred approach for selecting a strategy that addresses both the economic development and future lake pollution concerns. Espinet et al. (2015) suggest that when dealing with climate change, the adaptive measures should be less sensitive to uncertainty. Recent reports have also recommended using the robustness to evaluate alternative strategies (Morgan et
Additionally, the method has the ability to characterize uncertainty with multiple plausible futures. Therefore, in this study robustness decision-making criterion will be utilized to evaluate the design options.

The next step is the identification of a strategy that performs well over a wide range of plausible future scenarios. Espinet et al. (2015) used a robust indicator to select the best course of action for adapting paved roads for five climate scenarios and suggested that an index close to zero indicates that the scenario is less sensitive to uncertainty, while a high value highlights the dependency of the scenario on future climate. In condition where many variables contribute to the response of a system, aggregating the action of all contributing factors is often suggested when quantifying the response. In this case, the robustness index of each design, $RI_i$, is evaluated by computing the square root of the sum of the square (SRSS) of the weighted average of the regret (Equation 5.2). This measure shows how a design will perform if there is a variability in the climate once the measure is implemented. This approach of quantifying the robustness of the design is similar to estimating the response of a structure under a dynamic loading. In such case, all contributing modes are considered in quantifying the response. Here, the observed climate can be seen as dynamic load to the design, which is considered as fix.

$$RI_i = \sqrt{\sum w_j r(i,j)^2}$$  \hspace{1cm} (5.2)$$

$$RI_i = \sqrt{\sum r(i,j)^2}$$  \hspace{1cm} (5.3)$$

where $w_j$ is the probability of occurrence of the future climate $j$.

Since all climates considered in this study are equally probable, Equation 5.2 is further reduced to Equation 5.3. The design condition with the smallest $RI$ value is expected to perform
well over a wide range of plausible future scenarios compared to a single future climate and will be considered as the most robust. Furthermore, the best adaption measure is selected through the performance index ($PI$). Following the identification of the design climate, the $RI$ values of the adaptive measures are compared and the measure with the highest $RI$ (Equation 5.4) is selected as candidate measure to be used at the site.

$$PI = \max_{l}(RI_{s})$$

(5.4)

5.7 Application of the Framework to the Case Study Bridge

The design of infrastructures is based on regional or local characteristics; future climate estimate should be based on similar scales. Recent climate change projection indicates a 37% increase on average of heavy precipitation in the Midwest. This increase in precipitation would affect the hydrology of the Midwest Rivers’ basins (Mellilo et al. 2014). The proposed framework will be applied to the US 30 bridge located on Skunk River basin in Ames, Iowa. This basin has been the object of the climate change vulnerability assessment pilot project conducted by Anderson et al. (2015).

Figure 5.3 displays the time history of the peak annual discharge flow rate of the USGS gage station near US 30 bridge. The circle points indicate the 1993, 1996, 2008 and 2010 flood events, which were all above the 100-year design flood events of this bridge. Due to the difficulty in predicting future greenhouse gas emission, the present study assumes the realization of A1 and A2 scenarios. Under A1 scenario, it was further assumed that a balance across all sources (A1B) and a heterogeneity of the world with high population growth, slow economic development; and slow technological change (A2) will be the future directions. These GHG emission scenarios were combined with seven GCM to generate fourteen climate and emission pairs (Table 5.1) for
which the peak annual discharge rate simulations for the gage station at the bridge location were generated for a hypothetical bridge lifespan of 50 years.

Highway bridge foundation designs are based on 100- and 500-year return floods. However, it may be difficult for climate change projections to distinguish between a future 100-year storm from a future 500-year. Consequently, our analysis focuses on flood events that have once change of occurring in 100 years (1% annual exceedance probability, AEP). Also, because of the uncertainty at regional and local scale is higher than that of the global scale, the design discharge rates were estimated at 95% confidence level. Figure 5.4 shows the estimated peak discharge rate for the two emission scenarios. In general, A1B GHG emission scenario results in the highest peak flow rate with 1% AEP.

In order to evaluate the impact of these design discharge rates on the case study bridge foundation, the induced-scarce depths were computed for each climate pair. The economic analysis for each countermeasure was performed for all climate pairs. This analysis starts by estimating the total scour depth (local + contraction), then, the materials quantities are estimated. Itemized cost are obtained from numerous sources including past bids from DOTs. Detailed of economic analysis, including the cost of the adaptation measures and the maintenance cost following the implementation of the measure are provided in Fioklou and Alipour (2015).

Figure 5.5 displays the relative cost of the partially grouted riprap adaptive measure as a function of the climate model and emission pair. The costs presented in the figure are relative to the cost of the design based on the baseline scenario (current flow data). The figure indicates how the choice of climate models can result in overdesign or underdesign of the countermeasure relative to the observable data. A negative relative cost means that the design based on the baseline
underestimates future events whereas a positive relative cost is seen as an overestimation of the future impact of climate on the structure.

The challenge for planning under uncertain climate is which condition to plan and design for. Because all climate pairs have an equal probability of occurrence, a sensitivity cost analysis was performed to assess all possible future observations. Here, a scenario is constructed by considering a pair of design and observe climates. That is, the adaptive measure is designed for climate A and, then, it is assumed that climate B is observed. In this case, for each GHG emission, a 7x7 matrix was generated. This matrix can be divided into two subsets: i) the cases where the design climate is equal to the observed climates and ii) the cases where the design climate is different from the observed climate. The first subset lies on the diagonal of the matrix while the off-diagonal terms are the elements of the second subset.

Figure 5.6 displays the comparative result of M1 adaptation cost under A1B emission scenario. The data on the horizontal axis represent cases where the design coincide with the observed climates. The difference in adaptation costs between the projected climate and the observed climate are normalized by the observed adaptation cost. As seen from this figure, for design based on all climate pairs, the observation E1C5 resulted in underestimating the impact, whereas E1C6 leads to overestimation of the countermeasure. The difference between the costs of designing an adaptation measure based on the future climate and current observation is shown in Figure 5.7. One can observe that E1C3 and E2C3 result in the lowest relative cost. Adopting a strategy based on minimum expenditure, Figure 5.7 shows the plot of the relative difference in cost based on C3. Under this design criterion it is clear that depending on the global climate model, the emission scenario affects the level of regret. For example, when C6 global climate model is
considered, E1 GHG emission scenario leads to a greater regret compared to E2. Whereas the emission scenario has no effect on the regret for C2.

It is evident from Figure 5.7 that any future climate will require an additional adaptive step to cope with the threat posed by such condition. If the adaptation measure in place cannot handle the surprises in the future climate, more economical and safety problems may ensue. However, if the design was based on the most aggressive climate, no additional steps are required to adjust the design to meet the observed situation. In fact, it should be noted that overdesign is not a bad decision for such uncertain future conditions. Due to the financial constraints on the transportation departments, designing for the worst future condition might not be the best financial choice. Therefore, rather than designing for maximum expected future condition or the least aggressive future climate, the decision makers might want a design that performs well over a range of future climate conditions.

For this design approach, the objective is to define ranges of regret as a decision criterion and evaluate the robustness of the adaptive strategies on a satisfaction basis. Considering the regret levels define earlier (NR, SR, AR, and UR), Figure 5.8 shows the histogram of the regret analysis. Three countermeasures are considered and the regrets are evaluated for each of them. The number on top of the bars indicates how many instances the design meets the specified regret level. It is worth mentioning that each countermeasure was designed for 49 possible combinations. The results suggest that the decision makers might want a design based on acceptable regret (AR). In this range, the adaptive measures perform better over more than 40% of the design-observed climate pairs for E1 emission and more than 33% of the planned scenario for E2 emissions. Under small regret, the analysis indicates a poor performance under E1 emission for the adaptive measures considered in the present study. This figure also shows a difference between the two
emission scenarios. Under AR and UR criterion, the three adaptation measures perform better under E1 than E2. However, under SR, the design of the adaptive measures under E2 tend to outperform E1, while there is equal performance of the adaptive measures under the two future GHG emission scenarios, if the decision makers opt for no regret.

The possible combinations available to decision makers to meet specific future goal out of 49, as seven projected climates and seven observed climates, have been considered in the present study (Figure 5.8). As the size of the planned scenario increases, it is expected that the frequencies will also vary accordingly. Thus, a further step is needed to reduce the overwhelming choices available to decision makers once a planned scenario is identified. This additional step is necessary for multiple reasons. First, under specific performance criteria, there is no significant difference between the adaptive measures in order to select a specific adaptive measure. Second, climate change is expected to trigger more extreme weather events, such as more intense precipitation, that are likely to produce area-wide emergencies and may require evacuation of areas vulnerable to flooding. As the vulnerable areas might contain a large number of bridges, it is desirable to narrow the design climate and countermeasure pair as much as possible. Consequently, the robustness decision criterion is applied to reduce the regret interval and eliminate the uncertainty in the selection. It is expected that the best adaptive measure will be the ones with the smallest performance index for the design climate. For each design, performance index for all plausible future climates is computed using Equation 5.3. The results of these analyses are shown in Figure 5.9. From this figure, it can be observed that C1 and C4 yielded the lowest robustness indicator values under E1 emission scenario, whereas C4 and C6 have the lowest robustness indicator values under E2 emission scenario. Further analysis of the performance index (Table 5.2) shows that in the case of E1, C1 and M2 will be the preferable choice. This table shows that depending on the
measure the policymakers decided to use, designing it assuming C4 global circulation model independently of the emission scenario will perform well over the range of climates considered in this study.

5.8 Conclusions

Climate change poses a threat to the transportation infrastructure that is indispensable for the economy and society. While its impacts will be non-uniformly distributed, the extent of its impacts at specific location remains uncertain. In addition, the existing climate prediction models performed well at larger special and temporal resolution, however, they are highly uncertain at local and regional levels. This creates serious challenges for infrastructure management in preparing for the future. Although, civil engineers consider uncertainties in their designs but these uncertainties do not account for the climate. The possibility that these uncertainties hold surprises, which could be unpleasant and quick to appear requires that transportation systems should be reassessed using most recently available data and considering possible future climate conditions. This will ensure that these infrastructures, which are central to the socio-economic of the country will remain operational under both extreme events and gradual changes in design floods.

As existing infrastructures must adapt to new sets of climates, a new methodology is presented in this paper to help decision-makers prepare for the uncertain future and under a budget constraint. The method is based on scenario planning by first identify the uncertain drivers of the climate. Furthermore, the selection and design of adaptation measure that will perform well over a wide range of uncertain climate are derived from regret and robust decision-making criteria. Because civil infrastructures, particularly bridges are designed to remain functional and safe for their design life which typical range from 50 to 100 years, the framework considers the remaining
life of the infrastructure as well as the cost of maintenance of the adaptive measure in evaluating the robustness of the strategy proposed herein.

The proposed approach is applied to US 30 highway bridge located over Skunk River basin. For this study, the identified threat to the infrastructure is flood-induced scour. In order to explore the possible adaptive measure, a set of scenarios was constructed considering two drivers of the GHG emissions and combined with seven plausible global climate models. This resulted in a set of 14 scenarios for which three adaptation measures were designed and test against regret and robustness criteria. The performance index analysis helped narrow the overwhelming choices to two under each greenhouse emission scenario.

References


Susskind L. *Policy and Practice: Responding to the Risk Posed by Climate Change-Cities Have no Choice but to Adapt*. Town Planning Review, Vol. 81, No. 3, 2010


**TABLE 5.1.** Global climate model and greenhouse gas emission scenario pairs used

<table>
<thead>
<tr>
<th>E1C1: A1B + CGCM3_T47</th>
<th>E2C1: A2 + CGCM3_T47</th>
</tr>
</thead>
<tbody>
<tr>
<td>E1C2: A1B + CGCM2_T63</td>
<td>E2C2: A2 + CGCM2_T63</td>
</tr>
<tr>
<td>E1C3: A1B + CNRM</td>
<td>E2C3: A2 + CNRM</td>
</tr>
<tr>
<td>E1C4: A1B + ECHAM5</td>
<td>E2C4: A2 + ECHAM5</td>
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</tr>
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<tr>
<td>E1C7: A1B + HADGEM</td>
<td>E2C7: A2 + HADGEM</td>
</tr>
</tbody>
</table>
Table 5.2. Performance index for adaptive measures

<table>
<thead>
<tr>
<th></th>
<th>A1FI</th>
<th></th>
<th>A2</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>CGCM3T47</td>
<td>ECHAM5</td>
<td>ECHAM5</td>
<td>HADCM3</td>
</tr>
<tr>
<td>M1</td>
<td>0.50</td>
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<tr>
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Figure 5.1. Proposed framework
Figure 5.2. Adaptive measure for scenario planning

Figure 5.3. Annual peak flow discharge rates
**Figure 5.4.** Effect of greenhouse gas on the 1% AEP of the design discharge flow rate

**Figure 5.5.** Countermeasure costs for future climate pairs relative to the baseline cost (current condition)
Figure 5.6. Normalized adaptive costs comparison for partially grouted riprap under A1B

Figure 5.7. Grout/filled cement bag countermeasure cost analysis for of the scenario planning
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CHAPTER 6. SUMMARY AND CONCLUSIONS

The performance of reinforced concrete bridges located in flood prone and high seismic regions has been evaluated in this study. Using the annual discharge history of the Sacramento River, near Bend Bridge, CA, the best-fit probability distribution function for the discharge rate was determined. The flow at the case study river was found to follow a Pearson Type III distribution. The design discharge flows were estimated for floods with annual exceedance probability (AEP) of 4%, 2%, 1%, 0.5% corresponding to 25, 50, 100, and 500-year return period, respectively. Because design and stability evaluation of highway bridge foundations are based on floods with 100-year and 500-year return periods, respectively, the corresponding local and contraction scour depths were computed for performance evaluation. The influence of the river channel shape was investigated using rectangular, trapezoidal, and triangular shapes. Through a Latin hypercube sampling technique, $10^6$ flood-induced scour depths were estimated which allows for the generation of risk curves for three different channel shapes.

The finite element models of the studied reinforced concrete bridges were generated in OpenSees platform. The models incorporated the soil-structure-interaction, which is an important part that influence the bridge response to earthquake loading. The important parameters of the soil layers such as the damping and shear modulus were evaluated for the soil layers. The $p$-$y$ spring model adopted is capable of capturing the slippage and gap forming during lateral loading. The erosion of soil sediment around the foundation was simulated in the finite element model through the removal of soil springs at the pile foundations. The effect of flood-induced scour was first evaluated for the bridge cases through modal analysis. The fundamental periods of the bridges indicate that scour lengthen the period of vibration of the bridge.
Because the structures considered are located in flood and seismic prone region, a multi-hazard scenario comprised of flood-induced scour followed by seismic activities are considered in the performance assessment. First, a probabilistic framework was used to quantify these events whether they are correlated or not. The approach includes the coincidence rate of the events and their durations. The probability of failure of the bridge is evaluated through a fragility analysis. A fragility function expresses the relation between the intensity of a seismic event and quantitative measure of its probable consequence on the performance of the bridge. Analytical fragility functions were defined for the bridge cases for four level of damages: minor, moderate, major, and collapse.

A bridge is a system of components. However, previous research on the seismic evaluation of bridges based the performance of the system on the column fragility analysis. The added flexibility in the system brought by the erosion of soil sediments around the bridge foundation have led some to believe that scour actual improves the seismic performance of the bridge. This conclusion is valid only if the system performance is based on the column performance. Our investigations have shown that in the presence of scour, assuming the column as critical component will result in overestimating the seismic performance of the bridges. As such, a multi-component fragility analysis procedure that considers the performance of the bridge as the collective performance of its individual structural components is proposed. Furthermore, the ability of the bridge to open to traffic following a seismic event suggest that the performance of the columns, piles, bearings, and abutments have the largest influence on the bridges resilience. Consequently, the response of these components were tracked for a given ground motion intensity. The system fragility curves are developed for considering the vulnerability of these critical components to assess the damageability of the bridge cases. To this end, a suite of sixty ground motions,
categorized into three groups having 50%, 10%, and 2% exceedance probability of occurrence in 50-years corresponding to a return period of 72, 475, and 2500 years, respectively, were used in the multi-hazard performance evaluation. Furthermore, the frequency content of the suite of ground motions is such that it covered significant vibration frequencies of the bridge cases. The components responses of interest are the relative displacement of the abutment, drift ratio of the column, shear strain in the bearing, and the formation of failure mechanisms in the pile. The limit state of each component was defined based on experimental results, field observations, and engineering judgement for each damage state (minor, moderate, major, and collapse). The constructed fragility curves show that the presence of scour at the bridge foundation influences the failure probability of the components. While a decrease in the failure probability is observed for the column, the failure probability of the bearing, abutment, and piles increased with the increased in scour depth under the same ground motion intensity. These results indicate that under multi-hazard scenario, the component governing the failure probability of the bridge system varies depending on the damage state and the magnitude of the ground motion.

Recent bridge failures due to scour indicate that the erosion of soil around bridge foundations is not uniform. Therefore, in this study, the unintended consequences of uniformity in local scour depth at the foundations are evaluated through a comparative case study that considers both uniform and non-uniform local scour depth at the support foundations. A sensitivity analysis is conducted to evaluate the effect of non-uniformity in scour depth at the bridge foundations on the performance of the various components of the bridge. This component level performance evaluation helps clarify the beneficial effect of scour on the seismic behavior of bridge reported in the literature. The modal analysis indicates that when there is a 25% difference in scour depth between the upstream and downstream pier, there is up to 9% increase in the first fundamental
period of the bridge. This non-uniformity in scour depth significantly influences the bridge’s higher modes of vibration. The largest difference in modal vibration was observed in the third mode suggesting a significant participation of the foundation in such vibration mode.

An increase in the maximum deck displacement up to 25% was observed under the erosive action of the 100-year return period flood for a differential scour depth of 25%. This increase was even higher under a flood with 500-year return period. This observation suggests that assuming a uniform scour depth at the foundation for seismic performance evaluation or design will not yield the worst condition. The moment demand at the column base shows that under uniform scour the energy absorbed decrease with the increase in scour depth. Furthermore, the comparative analysis results of the demand at the columns experiencing the same scour depth in both uniform and non-uniform cases indicates an increase in the maximum moment induced at the column base as one move from the non-uniform bridge cases to the uniform bridge cases. This observation led to conclude that the condition at the downstream pier influences the response at the upstream pier and vice versa. At the abutment, increase in energy dissipation is observed as the scour depth is increasing.

The shear force demand on the other hand decreases as scour depth increases. When non-uniformity in scour depth is considered, the analysis indicates an increase in the shear force at the column with the same scour depth as the uniform case. Under the effect of 100-year return flood, a 25% difference in scour depth between the upstream and downstream piers results in up to 10% increase in the shear demand compared to the uniform scour case. As much as 40% increase is observed under 500-year return period flood. This difference in the shear demand observed might be attributed to the difference in the frequency of vibration of the piers as result of differential scouring. The analysis of pile behavior under the combine action of the flood-induced scour and
earthquakes shows that the soil response is concentrated in the top 4.0 m of the soil layer and the maximum demands are located between 2D and 7D depth below the soil surface, where \( D \) is the pile diameter. The results of the investigation help improve our understanding of the performance of highway bridges subjected to multi-hazard actions of flood-induced scour and earthquakes.

Bridge foundations are now and ever in great danger of constant flooding as many analyses indicate change in the climate variability and projected with higher confidence an increase in precipitation. Under the increasing risk of flooding, transportation stakeholders and transportation policy makers must adapt their infrastructure to the new environmental stressor. This study is significant because the impacts of climate change are intertwined with socio-economic trends like demographic change. Furthermore, the change is unevenly distributed through the world and the effects depend on the geographic location and local climate.

In other to assist transportation planners adapt infrastructures to the change in the climate, a framework was proposed and applied to a case study bridge. The proposed framework evaluates, specifically, the potential increase in flood-induced scour as result of increase in the precipitation projected for the Midwest. As such, the framework starts by evaluating the risk associated with the failure of transportation infrastructures. For longtime, climate has been considered stationary when designing bridge foundations. However, current observation in the climate suggest that this assumption is no longer valid. Consequently, the proposed framework assumes two technological developments and seven global climate models to construct a matrix of projected climate scenarios then estimated the resulting annual peak discharge flows for a period that extend from 1961 to 2070. Face with uncertain future, one approach will be to design the adaptation measure for the worst case scenario. Although this approach might be on the safe side of the spectrum, it is not economically feasible considering the financial constraints on the state DOTs. As a result,
transportation planners need a tool that incorporate deep uncertainty in the climate into the design and maintenance planning in the future.

The survey of cost of the tradition armoring devices use as adaption countermeasure to the erosion of soil at the foundation indicates that the cost is influence by the extent of scour, the location of the bridge, and availability of material. An alternative measure that use the traditional armoring devices in conjunction with sand filled geocontainers was designed and compared against the traditional approach. This alternate measure result in the reduction of up to a third the cost of the adaption when compared with the tradition approach. A scenario-based analysis is then used to design the countermeasures for 49 projected-observed climates. In addition to low regret design, an alternative design evaluation is proposed based on four level of regret decision makers is willing to take, to select the best climate and design pair. Furthermore, the robustness of the design is quantified through a robustness index. The proposed designed-observed pair that result in the lowest value of regret index (RI) is considered as the most robust design.

Faced with the projected consequences of climate change on the transportation sector, financial constraints, and the inability of current climate models to predict with certitude future climate as it is related to many factors among which the technological development, decision makers have the choice of adapting or do nothing. The methodology proposed in this study is to assist stakeholders and policy decision makers for adapting transportation infrastructures face with uncertainty in the future state of the climate since the stationary assumption on climate used to design bridge foundation is not long a valid assumption in the light of recent observations. It is worth noting that the findings of this study are limited only to precipitation as climate stressor.