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Quantifying the Seismic Vulnerability of Bridges in Low to Moderate Seismicity Regions

John Edward Lens
University of Vermont

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QUANTIFYING THE SEISMIC VULNERABILITY OF BRIDGES IN LOW TO MODERATE SEISMICITY REGIONS

A Dissertation Presented

by

John E. Lens

to

The Faculty of the Graduate College

of

The University of Vermont

In Partial Fulfillment of the Requirements for the Degree of Doctor of Philosophy Specializing in Civil and Environmental Engineering

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Dissertation Examination Committee:

Mandar M. Dewoolkar, Ph.D., Advisor
Eric M. Hernandez, Ph.D., Co-advisor
Dryver R. Huston, Ph.D., Chairperson
Keith Klepeis, Ph.D.
Ting Tan, Ph.D.
Cynthia J. Forehand, Ph.D., Dean of the Graduate College
ABSTRACT

The U.S. Congressional Research Service issued a report for Congress in May 2016, entitled “Earthquake Risk and U.S. Highway Infrastructure: Frequently Asked Questions” which highlighted the absence of a national database on the status of seismic vulnerability of bridges or other infrastructure, and thus no estimate of costs to retrofit vulnerable bridges. Low to moderate seismicity regions exist in each of the continental United States, with over 30 states having mostly or entirely low-to-moderate seismicity. Resources at state transportation agencies and municipalities are focused on higher seismicity regions, creating a gap in quantifying the system-wide seismic vulnerability despite an overall aging bridge inventory, much of which was built before current seismic design standards.

This research addressed this data gap and reduces barriers to quantifying seismic vulnerability of existing bridges in low-to-moderate seismicity regions. The work included nonlinear dynamic numerical modeling of typical multiple span bridge configurations in both pristine and deteriorated conditions, by subjecting them to seventy ground motions across four low-to-moderate seismic hazard levels, to evaluate their seismic performance. These typical bridge configurations represent over 160,000 bridges, which comprise 55% of the multiple span bridges nationwide.

The research results indicate that there is an overall low probability of significant seismic damage to these typical bridges in such regions. The results also show that current seismic hazard thresholds used for the design of new bridges, and for retrofit of existing bridges, which provide the basis for exempting some bridges from specific seismic analysis and design, can underestimate the expected seismic forces. Those results can be used to refine those exemption thresholds to provide appropriate protection against potential seismic damage in those cases. The study results also formed the basis for a system-wide rapid seismic vulnerability screening algorithm developed for the Vermont bridge inventory, which is applicable to other states with low to moderate seismicity regions.
The research and work described in this dissertation was funded by the Vermont Agency of Transportation (VTrans) with administrative support provided by the University of Vermont Transportation Research Center (UVM TRC). VTrans personnel supporting the project work included Emily Parkany, Pam Thurber, and Wayne Symonds. Chris Benda was instrumental through sharing his vision of the project’s value to agency leadership. Key background data and field visit support to bridge sites came from Jason Cloutier, Callie Ewald, Tom Mancini, Justin White, and Joshua Martineau of VTrans.

I deeply appreciate the unwavering support, encouragement, and friendship of my advisor Mandar Dewoolkar, and co-advisor Eric Hernandez, throughout the span of this work. I also appreciate the support and encouragement of my thesis committee members. My fellow graduate students including Ian Anderson, Kalil Erazo, Scott Hamshaw, John Hanley, Kate Johnson, Jim Montague, Lalita Oka, Nestor Polanco, and Kristin Underwood, each in their own way, whether they were aware of it or not, provided vital support to this research. Other UVM students who helped with work on this project included Connor Butwin, Lucas Howard, Michael Koch, and Tyler Kuehl.

I am very grateful for the sustained support of my family and friends, for they underpinned my ability to accomplish this work - you know who you are and what you did. Finally, I offer my deepest gratitude to Barb, for her kindness, humor, cheer, encouragement, and love throughout this effort.
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CHAPTER 1  INTRODUCTION

1.1  Motivation

This research focuses on quantifying the probability of seismically induced damage to existing bridges which typically may not have been specifically designed for seismic hazard and which are in low to moderate seismic hazard zones in the United States. Although there is no nationally adopted formal definition of what constitutes low, moderate, or high seismic hazard zones, moderate hazard zones for this research are characterized as regions where the expected seismic ground motions defined by Peak Ground Acceleration (PGA) are less than approximately 0.25g. Low to moderate seismic hazard regions as designated by the corresponding analysis requirements in American Association of State Highway and Transportation Officials (AASHTO) and the Federal Highway Administration (FHWA) standards and guidance, respectively, comprise a majority of the continental United States, as illustrated in Figure 1.1.

![Figure 1.1. Extent of most low-to-moderate seismic hazard (PGA < 0.25g) areas in the continental United States.](image)
A significant factor motivating this research is the low hazard characterization given to most of the continental United States whereby bridges built in this region have been and continue to be exempt from rigorous seismic analysis and design requirements, according to current highway structures codes and guidelines (AASHTO 2017, FHWA 2006). The 1953 edition of AASHTO Standard Specifications for Highway Bridges states that earthquakes should be considered but does not provide quantitative values for the earthquake loads. The 1961 edition specifies a lateral force of between 2 and 6 percent of the structure dead load, to be applied at the center of gravity of the structure, for seismic loads, with that same requirement continuing until the 1977 edition. The 1977 edition instituted a seismic force requirement which was based on the structure characteristics, seismic hazard at the site and soil conditions. It also specified a lateral force allowance for design of restraining features, such as bearings, of 25 percent of the contributing dead load.

The latest bridge design requirements (AASHTO 2017), which are applicable solely for new bridges, provide a no-analysis exemption from requiring specific seismic design for all single span bridges, and for multiple-span bridges in low seismic hazard regions. However, the AASHTO 2017 design code provisions require for all bridges that the lateral load capacity at each bridge element accommodate horizontal forces of either 10 or 25 percent of the vertical loads (depending on the seismic hazard at that location) on corresponding bridge elements. While this does not explicitly address seismic loads, those capacities are intended to accommodate seismic demands in low-to-moderate seismic regions. Answering the question of whether those force capacities are sufficient is one of the motivations of this research.
There has not been, nor is there currently, a seismic analysis and design requirement for retrofitting existing bridges as there is for new bridge design through the AASHTO bridge design standards. The latest national guidance for evaluating and retrofitting existing bridge was published by the United States Federal Highway Administration, in the Seismic Retrofitting Manual for Highway Structures: Part 1-Bridges (FHWA 2006). This guidance document provides for a tiered approach to seismic analysis required to evaluate seismic vulnerability and retrofit requirements, which depends on the remaining service life of the bridge, and the criticality of the bridge to the transportation system in post-earthquake response. No seismic analysis or retrofit is required for any bridge with a remaining service life less than 15 years. This also applies to bridges in low seismic hazard regions. Successively more comprehensive seismic analysis requirements apply with increasing seismic hazard at the bridge location.

Quantifying the vulnerability of bridges in these regions is important for purposes of planning and prioritizing bridge retrofit or replacements, and emergency preparedness and response planning. Such quantification proved itself valuable in expediting post-earthquake bridge inspections required as part of prudent engineering practice before reopening potentially affected bridges to traffic in Virginia, following the August 2011 Mineral, Virginia earthquake (TRB, 2012). Research has been performed toward quantifying the seismic vulnerability of bridges (e.g., Bignell and LaFave 2009, Pan et al. 2010a, 2010b, Nielson and DesRoches 2007a, 2007b, Padgett and DesRoches 2008, Ramanathan et al. 2010a, Ramanathan 2012, Tavares et al. 2012) in low to moderate seismic regions, specifically including the central, southern and eastern United States and eastern Canada. This research focused on the vulnerability of the bridges, on a component
and overall basis, along with evaluating analytical methods. These are comprehensive evaluations to develop fragility curves for seismic vulnerability based on hypothetical earthquake demands, ranging to well above low-to-moderate seismic hazard levels. Consequently, there is relatively low resolution of the vulnerability in the low seismic hazard range, and there is limited attention to deteriorated conditions, leading to another motivation for this research.

A survey of state transportation agencies made as part of this study revealed that seismic vulnerability is not being systematically evaluated in low to moderate seismicity regions in the United States, nor in some instances, in higher seismicity regions. This exposes bridges in these areas to seismic vulnerability which is not explicitly being addressed in terms of specific risk mitigation programs, emergency planning, nor in financial planning for disaster response and reconstruction. Moreover, the survey responses suggest that addressing seismic vulnerability is not of enough concern and/or a high enough priority to warrant directing already scarce transportation agency resources in the United States to making system wide evaluations.

This lack of priority being given to evaluating seismic vulnerability of existing bridges likely stems from the fact that earthquakes are infrequent in these areas, and except for the August 2011 Mineral, Virginia earthquake, have not imposed significant damage. The exemption from seismic design requirements in low to moderate seismic regions in the applicable transportation facility design codes and guides (AASHTO 2017, FHWA 2006) further de-emphasizes seismic vulnerability. This research provides opportunity to either substantiate or refine the current threshold values for exemption from seismic analyses and design in these regions.
A major challenge for embarking on a program to evaluate seismic vulnerability is that thousands of bridges exist in each state and conducting a bridge-specific analysis to assess seismic vulnerability is not practical. Therefore, seismic vulnerability screening methods for conducting assessments of the seismic vulnerability of existing bridges have been developed in the United States beginning in 1983 (FHWA1983, FHWA 1987, FHWA 1995, FHWA 2006). These methods rely on targeting a series of bridge characteristics which, from experience and observations of seismic damage, are key indicators of seismic vulnerability. Some of these characteristics are quantifiable from bridge inventory records, such as skew angle, and bearing types. Other characteristics require reviewing design records for subsurface conditions and accessing construction plans or making measurements at the bridge (e.g., bearing seat dimensions).

Overall these existing screening procedures are simpler and faster than an individual detailed analysis of a specific bridge and follow a relatively intuitive line of reasoning. This suggests that they may be a good basis to expand upon to achieve an improved vulnerability screening framework. However, two important limitations associated with these screening procedures first need to be addressed:

1. The NBI database is the only readily accessible resource of bridge inventory information available. However, it alone does not provide enough data with which to perform the analyses. Additional data needs to be gathered from other sources, such as the bridge plans, or by site visits, which hinders using these screening procedures, particularly on a system wide basis.
2. The evaluations do not account for the existing conditions of the bridges. Given the advanced ages of most bridges, they can be substantially deteriorated and more vulnerable, which is not accounted for in the current screening procedures.

Attention to seismic vulnerability of bridges in both practice and research focuses on multiple span bridges. This is exemplified in the AASHTO design standards which exempt single span bridges from specific seismic analysis requirements (AASHTO, 2017). The approximately 600,000 total national bridge structure count in the National Bridge Inventory (NBI, 2012) contains approximately 291,000 multiple span bridges. Most of these bridges are of a single type, consisting of reinforced concrete substructures with multiple steel girders with concrete deck superstructures, as shown in Figures 1.2 and 1.3. This provides a fortunate circumstance allowing this research to apply to more than half of the U.S. bridge inventory.

![Figure 1.2. Bridge with a two square column bent at 30-ft-tall supporting simple span multiple beams (Photo courtesy of VTrans).](image)

![Figure 1.3. Bridge with a three round column bent with 20-ft-columns supporting continuous span multiple girders (Photo courtesy of VTrans).](image)
The most common bridges consist of multiple span steel girder bridges with concrete bent frame substructures. A review of the National Bridge Inventory statistics (NBI, 2012) indicates that 73 percent of the bridges in the northeast United States (NEUS), defined as the New England states plus, New York, New Jersey, Pennsylvania, are at least 20 years old. Approximately 9 percent are over 80 years old. Aging bridges often have deterioration of concrete and exposed steel reinforcing, as illustrated in Figure 1.4, which could make them more seismically vulnerable. Current bridge seismic retrofit guidelines (FHWA, 2006) do not include provisions to account for deterioration when evaluating seismic vulnerability of existing bridges.

This most common bridge type (reinforced concrete substructure with multiple steel beams/girders with concrete deck superstructure) is widely used for interstate highway bridges since the inception of the interstate highway system in the 1950’s. With most of the interstate highway system having been constructed between the mid 1950’s and mid 1960’s, a substantial portion of these bridges have been subjected to upwards of 50 years of use and consequent weathering. In colder regions such as the NEUS, this means that both the concrete and steel are often deteriorated and corroded, primarily from deicing salts, as illustrated in Figure 1.4.
Quantifying seismic vulnerability of bridges requires two key capabilities. First, there needs to be a reliable method of modeling the vulnerable aspects of the bridges subjected to earthquake ground motions. There is general agreement within the profession that available modeling tools and techniques adequately model earthquake loading on bridges and overall deformations and stresses in such bridges. Second, there needs to be a reliable metric of what constitutes seismically induced damage, i.e., vulnerability, in the bridges. For this aspect, metrics such as damage indices based on maximum imposed displacement and cycles of displacement post-yield are adopted in research and practice.
A critical input for this seismic vulnerability evaluation is to incorporate the characteristics of the earthquake ground motions applicable to the hazard level, and tectonic region where the bridges are located. Those characteristics are not individually predictable due to the quantity and complexity of the factors which contribute to a particular earthquake ground motion reaching a site located some distance from the earthquake origin. Research progress in the past few years in predicting ground motions relatively near (i.e., within about 10 km) to shallow fault sources does not apply to ground motion predictions in low to moderate seismic hazard regions which are, almost by definition, devoid of shallow faults, and with few known active faults. Instead, ground motions are predicted based on seismicity models incorporating recorded regional seismicity and applying attenuation relationships for probabilistic seismic hazard predictions usually described with three commonly used ground motion characteristics. Those are the PGA, and spectral accelerations at periods of 0.2 and 1 second, which are extended to provide a smoothed design acceleration response spectrum over the range of structural period of interest. Bedrock motion amplification associated with the overlying soil profiles at each bridge site is accommodated through empirical scaling factors specified for five soil profile categories (AASHTO 2017, FHWA 2006) designated as A through E, depending on soil stiffness.

This approach creates a challenge for modeling bridges using recorded ground motions. There are few ground motion records available from past earthquakes specifically from low to moderate seismic regions with similar tectonic conditions as those in the central and eastern United States (CEUS). This limited selection inhibits developing statistically robust results using motions from similar intraplate tectonic settings and
matching soil profile categories. This limitation requires using ground motions from other, higher seismic hazard regions and accommodating in some manner, the potential for those results to differ from what could result if ground motions from the local seismic hazard and tectonic region were used in the analyses. A promising development from recent research effort to improve ground motion prediction models is a forthcoming database of ground motions from eastern North America (PEER NGA East 2015). While there will still be a relatively limited amount of larger magnitude earthquake records, the expanded database will eventually provide strong-motion records which are from this intraplate tectonic region.

In the meantime, this study uses existing earthquake ground motion records which match both the site soil profile categories bracketing the range from stiff and soft-soil conditions and predicted seismic spectral acceleration levels for low-to-moderate seismic hazard. This provides a novel alternative to the usual practice of scaling ground motions to match the predicted spectral acceleration.

In summary, the motivation for this research arises from the following:

- There is an extensive inventory of older bridges in the U.S. which have not been specifically designed and constructed for seismic hazards and a lack of quantification of seismic vulnerability of those bridges.
- The bridge population is aging with rehabilitation and replacement not keeping pace with their deterioration, thereby potentially making the inventory overall more seismically vulnerable.
• A rapid, low-cost seismic vulnerability screening methodology for the entire bridge inventory is needed to address the existing current lack of quantification in that regard, and to facilitate timely updates coincident with annual bridge inspection findings.

• A quantitative evaluation of existing no-seismic analysis exemption thresholds and compensating requirements for new and existing bridges is needed to allow assessing whether those provide the intended seismic hazard protection for areas of low to moderate seismicity.

1.2 Objectives and Research Questions

The research objectives are to evaluate seismic vulnerability of bridges in low to moderate seismic regions to achieve the following:

i. Quantify the vulnerability of typical bridges using displacement and energy-based damage measures. This is to include accounting for deteriorated conditions of aging bridges.

ii. Translate this into a framework to estimate system wide vulnerability of multiple span bridges in the low to moderate seismic hazard region comprising the northeast United States

iii. Employ the framework developed for Item ii to expand its applicability to seismic vulnerability assessment for bridges in low to moderate seismic hazard region throughout the continental United States.

iv. Using the results from Item i, evaluate no-analysis thresholds for low to moderate hazard regions of the continental United States, and their suitability for protecting bridges from significant seismic damage.
v. Based on findings from Items i through iii, develop a methodology to perform rapid system wide seismic vulnerability screening and quantification for low to moderate hazard regions.

1.3 Organization of the Dissertation

Chapter 2 provides a summary of the background information obtained from literature review.

Chapter 3 describes the non-linear time history analysis of bridge models to evaluate vulnerability of the typical concrete bridge bents supporting multiple span bridge investigated for this research, and corresponding results.

Chapter 4 describes the investigation of current thresholds of bridge seismic vulnerability depending on seismic hazard level, and conclusions regarding their suitability in protecting bridges from significant seismic damage.

Chapter 5 describes the system-wide rapid seismic vulnerability screening algorithm developed for the State of Vermont Agency of Transportation (VTrans).

Chapter 6 provides the conclusions and recommendations for quantifying seismic vulnerability of bridges on a system-wide basis in low-to-moderate seismic hazard regions.

Chapter 7 provides a bibliography of literature accessed and reviewed for this research.
CHAPTER 2 BACKGROUND LITERATURE

2.1 Characteristics of Bridge Seismic Vulnerability

Bridge seismic vulnerability considered for this research relates solely to the stability and post-earthquake functionality of the structures themselves. The associated transportation system vulnerability is important, and a necessary consideration for ranking bridge vulnerability, but system vulnerability characteristics are not needed for the structural analyses made in this work, and no research aside from review of a limited amount of publications on the topic, was performed. Guidance on that topic is available in references such as the REDARS (Risks from Earthquake Damage to Roadway Systems) methodology (Werner et al. 2006) which is a comprehensive software package to evaluate seismic vulnerability of large-scale transportation networks.

Bridge seismic vulnerability characteristics are determined from multiple sources including empirical observations in post-earthquake reconnaissance, from field and laboratory testing, and analytical studies such as those performed for this research. This research has used the combination of published work on these methods, plus original analytical work to investigate the vulnerability of a large subset of bridges in low-to-moderate seismic regions.

2.1.1 Post-earthquake Reconnaissance Publications

Formalized documentation of earthquake reconnaissance of bridge damage has provided much empirical basis for estimating bridge seismic vulnerability and has led to characteristics-based screening methods developed by the U.S. Federal Highway Administration (FHWA) Seismic Retrofitting Manuals (FHWA 1995 and 2006). Since strong earthquakes are what warrant post-earthquake reconnaissance, there could be a
bias in the bridge characteristics identified to be more vulnerable. Another source of potential bias arises from reconnaissance of previously recognized higher seismicity areas where seismic retrofitting has already been done or where bridges have been designed to newer seismic codes, thereby reducing the extent of future seismic damage. This seems to be reflected in the bridge reconnaissance report (Astaneh-Asl et al. 1994) from the magnitude 6.7 January 17, 1994 Northridge earthquake in the Los Angeles area which indicated relatively limited seismic damage to steel girder with concrete deck bridges, despite ground motions estimated to have reached at least 0.35 g in the vicinity (Astaneh-Asl et al. 1994). This is also reflected in the post-earthquake reconnaissance observations reported by Virginia DOT (VaDOT), following the magnitude 5.7 August 23, 2011 Mineral, Virginia earthquake. The VaDOT reported limited damage to bridges, many of which had been subject to seismic vulnerability screening in 1994, with subsequent attention to vulnerabilities identified in that screening prior to 2011. Nonetheless, reconnaissance reports have been used to provide pertinent guidance on expected vulnerability of typical highway bridges subjected to earthquakes, leading to the recommendations in the Seismic Retrofitting Manuals (FHWA 1995 and 2006).

2.1.2 Field Seismic Testing

There have been a limited number of applicable field-testing type evaluations related to bridge seismic vulnerability for highway bridges designed prior to adoption of seismic design developments in the 1970’s. The field testing project with most relevant results to this study is a concrete bent and girder bridge in the state of Washington subjected to static lateral load testing before demolition (Eberhard and Marsh 1997a, 1997b). That project provides valuable insights applicable to this study because it
involved a typical multiple-span, concrete bent interstate highway bridge constructed in 1966, which was not designed for significant seismic loads, of the typical configuration evaluated in this study. The bridge had two-column bents comprised of twenty-five-foot long, three-foot-diameter reinforced concrete columns with minimal transverse reinforcement comprised of No. 3 hoops at 12-inches on-center, and lap splices in the longitudinal column reinforcement near the footing level. The superstructure of this continuous three-span bridge consisted of prestressed concrete girders cast into the abutment back-walls. The pair of bents was laterally loaded for six cycles, incremented up to a final 769 kips horizontal force, which comprised 65 percent of the total corresponding dead load. Maximum bent displacement reached 3% with spalling at the columns but without shear failure. The study concluded that this bridge configuration would readily withstand a horizontal acceleration of 0.4 g with minimal spalling at columns and could withstand 0.6 g horizontal acceleration without collapse.

2.1.3 Analytical Studies of Seismic Vulnerability

Analytical studies evaluating bridge seismic vulnerability are the largest body of published information on bridge characteristics leading to seismic vulnerability. There are three types of studies: empirically-based fragility studies using earthquake damage records; fragility studies of bridge types; and, studies of individual bridges or types focused on a specific bridge feature or group characteristic.

The study of the Loma Prieta 1989 and Northridge 1994 earthquake bridge damage data by (Bazos and Kiremidjian 1998) provides a system-wide fragility evaluation of the seismic vulnerability of bridges based on analysis of bridge feature characteristics, estimated ground motions, and recorded damage, with resulting empirical fragility curves
for expected damage and cost of repair or replacement. Peak ground accelerations in both earthquakes were estimated to have reached at least 0.6 g near the epicenters. Findings and conclusions relevant to this study include the observation that bridges with high skew, discontinuous spans, single column bents, and non-monolithic abutments performed poorly, as did bridges designed before 1971, when improved seismic design was adopted following the damaging 1971 San Fernando earthquake in California. While it is fortunate that less than five percent of the bridges exposed to ground shaking were damaged it is important to recognize that seismic design criteria for bridges in California was substantially revised in the 1970’s, potentially accounting for the low damage proportion despite the substantial peak ground shaking values.

The fundamental concepts of the analytical fragility curve analysis methodology applied to bridge models is described in the work by Vamvatsikos and Cornell (Vamvatsikos and Cornell 2002) and supplemented by Karim and Yamazaki (Karim and Yamazaki, 2003). The method consists of incrementally scaling a demand on a structure through, in the cases considered here, a non-linear time-history analysis, to produce corresponding responses based on the intensity of the applied demand.

There have been numerous analytical fragility studies focused on bridges in the central and southeastern United States to address vulnerability from medium to high seismicity (Nielson and DesRoches 2007a; Choi, DesRoches, and Nielson 2004; Bignell and LaFave 2009; Ramanathan, DesRoches, and Padgett 2010; Ghosh and Padgett 2010). These fragility studies include multiple span multiple girder bridges which are common throughout the U.S. and offer insights on several fragility aspects including skew, retrofitting outcomes, aging considerations and including focus on bridge types typical of...
the region. Often the demand is related to peak ground acceleration although other measures such as spectral acceleration are used. Overall, these studies show a low probability of catastrophic damage at low to moderate peak ground acceleration values corresponding this study’s seismic hazard interest.

Similar studies of seismic fragility of bridges in eastern Canada (Tavares, Padgett, and Paultre 2012) and the state of New York (Pan et al. 2010a, 2010b), respectively, provide comparative findings to the work done for the central and southeastern U.S, multi-span simply supported and continuous bridges. That is, the analyses show a probability under about three percent of moderate damage with a PGA of up to 0.25g, and slight to negligible probability of higher damage.

2.2 Damage Quantification

Quantifying the extent of seismic damage which occurs in a modeling analysis is fundamental to evaluating bridge seismic performance in those studies. A state-of-the-art review by Billah and Alam (Billah and Alam 2015) cataloged analytical methods, bridge components evaluated, demand parameters, intensity measures, and uncertainty parameters of 48 published highway bridge fragility assessments. Most assessments addressed column curvature or displacement ductility, or drift as demand parameters, most often based on peak ground acceleration, with about ten percent of the analyses using spectral acceleration. The Park and Ang (1985) damage index based on hysteretic energy dissipation was used in one study. The damage index is comprised of displacement past yield, and the energy dissipated into the structure in cyclic deformations past the yield point. In addition to the original damage index work by Park and Ang (1985), energy dissipation-based methods for quantifying seismic damage potential have been developed
and used by others including Bozorgnia and Bertero (2002), and Hernandez and May, (2013). The Park and Ang damage index characterization incorporates maximum displacement as a component of the damage measurement, and via the energy displacement component also provides a means of removing subjectivity in interpreting column hinge damage.

2.3 Existing Seismic Vulnerability Rating Methods

Two types of seismic vulnerability rating methods have been described in the published literature. Relatively rapid screening methodologies based on evaluating cataloged features of bridges are one type. Work by Filiatrault, Tremblay, and Tinawi, 1994 for all categories of bridges, focused on developing a rapid screening procedure accounting for structural type, complexity, span type, support redundancy, bearing features, and skew, along with non-structural features relating to traffic volume, detour lengths, and the route types. Work by Dicleli and Bruneau (Dicleli and Bruneau 1996) for steel column supported multiple girder bridges focused on developing a ranking index based on bearing seat dimensions, bearing damage potential, and steel column damage potential. Both are based on calculating vulnerability of specific bridges using local seismic hazard data. The calculations are of the type that can be organized in a spreadsheet type approach. Published examples where these have adopted into transportation agency screening methods were not located.

The second category requires more detailed evaluation of individual bridges, as described in the FHWA 1995 and 2006 Seismic Retrofit Manuals. The New York State Department of Transportation Bridge Safety Assurance Seismic Vulnerability Manual (NYSDOT, 2004), originally published in 1995 and subsequently revised through 2004,
incorporated elements of the 1995 version of the FHWA Seismic Retrofit Manual. The FHWA and NYSDOT methods incorporate various aspects of bridge importance, operational performance requirements, seismic hazard level, and bridge features including such as skew, seat widths, column and foundation types, and site soil conditions. Consequently, these screening methods require either access to construction plans and geotechnical reports or site visits to gather data. For older bridges, geotechnical explorations may be required if boring data is absent.

2.4 Bridge Deterioration

Deterioration of the structural elements of typical bridges, composed of reinforced concrete foundations and substructures, and concrete decks, with either steel or concrete beams, occurs as corrosion of the steel elements, and as reinforced concrete deterioration resulting from corrosion of reinforcing steel. Corrosion may result from proximity to marine environments, bridges being in cold climates where deicing solutions are used, or a combination. Work on bridge deterioration sources and rates by Kim and Yoon, (2010) and Agrawal, Kawaguchi, and Chen (2010) addressed the deterioration rates and sources in typical cold climate regions of the Midwest and NEUS, respectively. Both studies indicate that bridge age is the primary predictor for deterioration.

The Agrawal, et al., (2010) work was aimed at predicting the progression with time of condition ratings ranging from new to serious deterioration based on the component of interest, design type, materials, and incorporating life-cycle experience of the New York State Department of Transportation design and maintenance engineers. That work emphasized decks and girders, and did include pier caps, although not specifically columns. The mean-value prediction to start exceeding minor deterioration is 50 to 60 years for pier
caps, depending on the predictive model. This is consistent with bridge conditions observable during every-day travel, where we see bridges built during the interstate highway expansion of the 1960’s showing deterioration ranging from insignificant to spalled concrete and exposed reinforcing in columns and pier caps. The implications of this milestone point for this study are to support modeling the deteriorated state of existing bridges as coinciding with conditions reflecting the start of problematic deterioration, that is, at the onset of full spalling of the concrete covering the longitudinal reinforcing. Bridges in such condition are more likely candidates to remain in service, while more deteriorated bridges are apt to be scheduled for major rehabilitation or replacement, which would encompass seismic considerations as part of that work.

Another source of reinforced concrete deterioration is occurring as Alkali-Aggregate Reaction (AAR). Published work on implications of AAR on seismic vulnerability of reinforced structures, including bridges, was not identified in the literature search. However, bridges in Vermont are affected to significant extent by AAR (Wild, Eliassen, and McMahan, 2008), mostly in the north-central portion of the state. AAR creates expansive forces which can cause spalling of cover concrete. The tendency for volumetric expansion has been found in work (Olave et al. 2015a, 2015b) for the Texas Department of Transportation to mitigate to a limited extent the outer concrete spalling, in that lap spliced longitudinal reinforcing bar pullout increased. This tendency has been observed by others (Huang et al. 2014) at lower levels of reaction with decreased pullout capacity at higher levels of reaction. This specific deterioration source is not studied separately as part of this research, based on the limited specific information available on
the implications of AAR on seismic vulnerability, and the similarity in resulting behavior, in the form of concrete cover spalling, to reinforcing steel corrosion.

Effects of corrosion on cyclic ductility of steel were investigated by Bruneau and Zahrai, (1997) on a limited number of plate steel elements. The investigation identified that steel member ductility and hysteretic energy dissipation capacity could be substantially reduced because of localized pitting, leading to stress concentrations where cracks initiate under relatively low levels of cyclic loading. The study indicated that this was applicable to features such as passive energy absorbing devices which would be subjected to localized flexural demands and suggested further study. This research is focused on the overall structure vulnerability, mainly reinforced concrete substructures, and for structures which are not specifically designed for seismic loading, so this potentially diminished ductility capacity for such features is important to recognize but does not need special attention.
CHAPTER 3 QUANTIFYING SYSTEM-WIDE SEISMIC VULNERABILITY OF AGING MULTIPLE SPAN MULTIPLE-GIRDER BRIDGES IN LOW TO MODERATE SEISMIC HAZARD REGIONS

3.1 Introduction

Quantifying the seismic vulnerability of existing bridges within a transportation agency portfolio is a vital aspect of managing those transportation assets. A survey of state transportation agencies in the United States conducted by the authors indicate that only two of thirty transportation agencies in low-to-moderate seismicity regions are conducting system-wide seismic vulnerability evaluations. In the rare cases when system-wide seismic vulnerability evaluation of bridges is conducted, the evaluations are performed by acquiring bridge data and site subsurface information from design or record plans and analyzing vulnerability by following seismic retrofit analysis calculation procedures per guidelines prepared by the U.S. Federal Highway Administration (FHWA) in 1995 and 2006 (“Seismic Retrofitting Manual for Highway Bridges “Seismic Retrofitting Manual for Highway Structures: Part 1 - Bridges: MCEER-06-SP10” 2006). In one instance, a state transportation agency (NYSDoT, 2004) developed their own screening protocol, which was modeled after the FHWA 1995 procedures. However, the screening protocols to-date are labor resource intensive when applied to entire inventories.

The absence of an alternative to the resource intensive work of gathering record plans and performing detailed calculations needed for the FHWA based seismic retrofit is likely a major reason that few transportation agencies in low-to-moderate seismic regions conduct seismic vulnerability screening. The effort required to analyze individual bridges for the FHWA-based evaluation was measured for this research to require four to six hour
for a conventional bridge, when including both the additional data gathering and calculations. In addition, the FHWA based evaluations do not have explicit provisions to account for aging aspects such as concrete cover spalling type deterioration. To our knowledge, there is no rapid seismic vulnerability screening algorithm available which allows that screening to be rapidly performed using only bridge characteristics data based on that compiled in the National Bridge Inventory (“National Bridge Inventory, Federal Highway Administration” 2018.) database. Such an algorithm would allow both a rapid initial screening, that could account for changes in bridge conditions with aging and could be rapidly updated as bridge inspections are conducted.

The progress to-date on seismic vulnerability evaluations for bridges offers the opportunity to develop a rapid seismic screening algorithm to enable state transportation agencies to routinely prepare and maintain an up-to-date bridge inventory which incorporates an accounting of risk associated with seismicity in their region. The example closest to this is the seismic vulnerability scoring approach used in the NYSDoT 2004 (NYSDoT 2004) seismic vulnerability manual which employs numerical scoring of vulnerability characteristics. Those include whether spans are simple or continuous, bearings toppling potential, degree of centerline skew, and the degree of redundancy in deck girder arrangement, which are features observed in post-earthquake inspections associated with damaged bridges. A missing element in that approach is a way to rapidly assign a quantified vulnerability score to concrete bridge bents, in both pristine and deteriorated condition. This work explains the analyses that have been done to provide a basis for enabling that scoring.

Research studies have been performed to develop fragility curves using incremental dynamic analysis (Vamvatsikos and Cornell 2002) for representative bridge types for
moderate seismicity hazards in the central and southeastern U.S. (Nielson and DesRoches 2007b; Choi, DesRoches, and Nielson 2004; Nielson and DesRoches 2007c; DesRoches et al. 2004; Ramanathan, DesRoches, and Padgett 2010), the northeast U.S. (Pan et al. 2010b, 2010b), and eastern Canada (Tavares, Padgett, and Paultre 2012). There has also been research performed to evaluate seismic vulnerability in bridges subjected to corrosion, particularly in regions of higher seismic vulnerability (Zhong, Gardoni, and Rosowsky 2012; Kumar and Gardoni 2012; Ghosh and Padgett 2010; Alipour, Shafei, and Shinozuka 2011). These studies address seismic hazard ranges which extend well above the low-to-moderate range. This wide spread demand, ranging to upward of 1 g, peak ground acceleration, leads to a relatively low resolution in estimating the fragility of bridges at low-to-moderate seismic hazard levels under 0.25g, and motivates this study which is focused on seismic hazard up to about 0.25g.

The category of multiple span bridges with multiple girder supported decks represents 55% of the 291,000 multiple span bridges in the U.S., as illustrated in Figure 3.1. This is one-third of the 473,000 total, non-culvert, U.S. highway bridge inventory tracked through the NBI, and is only surpassed in quantity by single span bridges, which themselves are generally not regarded as seismically vulnerable (Buckle 1991).
This research evaluated those multiple span girder bridges, which are widely used for interstate and urban highways, examples of which are shown on Figures 3.2 and 3.3. These bridges are referred to as Bridge A and Bridge B, respectively, herein. The bridges have reinforced concrete column supported bents configured in repetitive type configurations of square or round columns, often in the range of 3 ft side width or diameter, respectively. The cross-beams supporting the girders are typically square or rectangular, typically about 4 ft in dimension. We did not evaluate wall and hammer-head type bents in this study, as those have already been considered for the mid-America seismic region in southern Illinois (Bignell and LaFave 2010a, 2010b) and found to be less vulnerable than multiple column bents.
The interstate highway bridge building expansion in the 1950’s through the 1960’s led to standardization efforts among state transportation agencies including sharing of plans and typical details (*Catalog of Highway Bridge Plans*, 1959). The AASHTO bridge design standards in the time frame between 1953 and 1977 required using seismic design lateral forces ranging between 2 and 6 percent of total dead load. The seismic force requirements were gradually increased over time, but remained as a fraction of the vertical tributary loads, as summarized in Table 3.1.

**Table 3.1. Historical Record of AASHTO Seismic Loading Requirements**

<table>
<thead>
<tr>
<th>Year</th>
<th>Reference</th>
<th>Criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td>1931</td>
<td>AASHTO(^1) 1st Ed.</td>
<td>None (Earthquakes not mentioned)</td>
</tr>
<tr>
<td>1953</td>
<td>AASHTO(^1) 6th Ed.</td>
<td>Earthquakes mentioned but no quantifications given</td>
</tr>
<tr>
<td>1961</td>
<td>AASHTO(^1) 8th Ed.</td>
<td>EQ = (C)(D) provides lateral force at center of gravity of structure; where C = 0.02/0.04/0.06 depending on supporting soil (i.e., spread footing bearing pressure or if piles are used), D = dead load (Live load may be neglected)</td>
</tr>
<tr>
<td>1973</td>
<td>AASHTO(^1) 11th Ed.</td>
<td>Same as 1961</td>
</tr>
<tr>
<td>1977</td>
<td>AASHTO(^1) 12th Ed.</td>
<td>EQ = (C)(F)(W); where C = (A)(R)(S)/(Z), F = framing factor (either 1.0 or 0.8), W = total dead weight of structure (lb.), A = maximum acceleration of bedrock (using risk map), R = normalized rock response, S = soil amplification spectral ratio, Z = reduction for ductility and risk assessment; Design of Restraining Features: EQ = (0.25) * (contributing DL) - column shears due to EQ</td>
</tr>
<tr>
<td>1981</td>
<td>FHWA(^2)</td>
<td>Numerous classifications and factors.</td>
</tr>
</tbody>
</table>

1. *Standard Specifications for Highway Bridges*

The beneficial consequence to this standardization effort is common bent and cross-beam dimensions for multi-girder bridges independent of the span lengths and configurations. A small number of bridge configuration models can be used to represent this...
inventory of 160,000 multiple span with multiple girder bridges across the country. This allows concentrating analysis on the influence of ground motion variability, and the influence of deterioration, on the seismic vulnerability.

The expected behavior of two actual bridges typical of those constructed as part of the interstate highway program, was analyzed using commercial structural analysis software, SAP2000 (SAP2000 vers. 17.3, 2015), and ground motion ensembles selected to match current AASHTO seismic design spectra bounding the range of low-to-moderate seismic hazard. That hazard range has been divided into two steps for this work. The first step covers from the minimal to low seismic hazard range, reflected in Peak Ground Accelerations (PGA) between about 0.01 and 0.06g. The second step covers a low to moderate seismic hazard range reflected in PGA’s of about 0.06 to 0.25g. Table 3.2 summarizes the range of spectral acceleration values at both seismic hazard conditions. The study analyses consider both pristine bridge conditions, matching the originally constructed concrete and steel design properties, and deteriorated conditions reflecting the concrete cover over transverse reinforcing steel being fully-spalled to the outside face of the confining bars.

Table 3.2 Spectral acceleration values used for low and low-moderate seismic hazard scenarios

<table>
<thead>
<tr>
<th>Hazard Scenario</th>
<th>PGA (g)</th>
<th>0.2 Second Spectral Acceleration (g)</th>
<th>1-Second Spectral Acceleration (g)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low</td>
<td>0.01-0.06</td>
<td>0.02-0.14</td>
<td>0.01-0.04</td>
<td>1,2</td>
</tr>
<tr>
<td>Low-Moderate</td>
<td>0.06-0.15</td>
<td>0.14-0.25</td>
<td>0.04-0.06</td>
<td>1,2</td>
</tr>
</tbody>
</table>

1. Values are derived from the USGS 2002 Seismic Hazard maps as published in AASHTO Bridge Design Specifications beginning in 2007.
2. Values are for Seismic Site Class B conditions and boundary values are approximate.
3.2 Analysis Features

3.2.1 Descriptions of Analyzed Bridges

Two bridges were analyzed in both their pristine states as constructed, and with complete spalling of the concrete cover over the transverse reinforcing steel. These bridges are shown in Figures 3.2 and 3.3 and referred to herein as Bridges A and B, respectively.

These bridges were selected so that their analysis findings would apply to most of the multiple span inventory in the low-to-moderate hazard regions of the central to eastern United States. To achieve this using a minimum number of example bridges, we selected bridges by considering bridge bent natural periods of transverse vibration, $T_n$, in the first mode. We started by estimating bridge bent natural periods for the combination of column heights and shapes (round and square) and span lengths in the bridge inventory. We analyzed the transverse stiffness, $k$, using the direct stiffness matrix approach (Ghali and Neville, 1989) using the mass of the bent and supported spans, to arrive at:

$$T_n = 2\pi \sqrt{\frac{m}{k}}$$

(eq. 1)

where $T_n =$ initial elastic natural period, $m =$ mass of the bent and tributary spans, and $k =$ transverse displacement stiffness of the bent frame. The results are in Table 3.3.
Table 3.3 Relationship of bridge natural period of vibration to span and column Height for multiple span with multiple girder bridges for two column bent case

<table>
<thead>
<tr>
<th>Pier Height, Feet</th>
<th>15%</th>
<th>75%</th>
<th>10%</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Span Length, Feet</strong></td>
<td>15</td>
<td>20</td>
<td>25</td>
</tr>
<tr>
<td>30</td>
<td>0.15</td>
<td>0.22</td>
<td>0.31</td>
</tr>
<tr>
<td>40</td>
<td>0.17</td>
<td>0.25</td>
<td>0.34</td>
</tr>
<tr>
<td>50</td>
<td>0.18</td>
<td>0.27</td>
<td>0.37</td>
</tr>
<tr>
<td>60</td>
<td>0.19</td>
<td>0.29</td>
<td>0.40</td>
</tr>
<tr>
<td>70</td>
<td>0.21</td>
<td>0.31</td>
<td>0.43</td>
</tr>
<tr>
<td>80</td>
<td>0.22</td>
<td>0.33</td>
<td>0.45</td>
</tr>
<tr>
<td>90</td>
<td>0.23</td>
<td>0.35</td>
<td>0.48</td>
</tr>
<tr>
<td>100</td>
<td>0.24</td>
<td>0.36</td>
<td>0.50</td>
</tr>
<tr>
<td>110</td>
<td>0.25</td>
<td>0.38</td>
<td>0.52</td>
</tr>
<tr>
<td>120</td>
<td>0.26</td>
<td>0.39</td>
<td>0.54</td>
</tr>
<tr>
<td>130</td>
<td>0.27</td>
<td>0.41</td>
<td>0.56</td>
</tr>
<tr>
<td>140</td>
<td>0.28</td>
<td>0.42</td>
<td>0.58</td>
</tr>
<tr>
<td>150</td>
<td>0.29</td>
<td>0.44</td>
<td>0.60</td>
</tr>
</tbody>
</table>

1. Tn values computed for transverse stiffness of two pier concrete bent frame with no contribution from adjacent span sections accounted for.
2. Values in *italics* indicate the approximate percentage of the multiple span with multiple girder bridges estimated with the corresponding span lengths and pier heights.

The next step for selecting example bridges considered the statistical distributions of bridge column height and span lengths within the inventory. Evaluation of the NBI (NBI, 2012) inventory indicates that 75% of the low-to-medium hazard states have multi-span bridges with column heights between 15 and 30 feet, with approximately 10% having greater column heights, and 15% having shorter column heights. Similarly, approximately 75% of the bridges have maximum spans of 60 to 120 feet. This serves as the focus range of bridge bents for this evaluation.
Our evaluation required using proxy information from the NBI database for column height since the NBI database does not include specific data on column heights. The NBI database category Item “54B – Minimum Vertical Under-Clearance” provides the distance between the bottom of the superstructure to features below, comprised of “highways, railroad, other features,” typically a river. Column heights extend between top of pile cap or footing to the bottom of the bent cap. Since most bent caps are about 4 ft in vertical dimension, which approximately matches the typical depth below ground surface to top of pile cap or footing. Consequently, the Item 54B dimension provides a close approximation of the column height.

The last step in selecting the example bridges was to search the record plan archives in Vermont for bridges which had a combination of span length, column height, construction year, record plan availability, bent configuration, and computed natural period of vibration which placed them at the chosen bounding column heights and spans. Two such bridges were identified and chosen for analysis. Bridge A, supporting two lanes of interstate highway over a town highway, and Bridge B supporting a two-lane state highway over the twin two-lane interstate highway section were constructed in 1964 and 1967, respectively. They reflect bent and superstructure configurations of multi-beam/girder bridges typical of that construction period. The bridge model parameters are given in Table 3.4.
<table>
<thead>
<tr>
<th>Bridge</th>
<th>Bridge A</th>
<th>Bridge B</th>
</tr>
</thead>
<tbody>
<tr>
<td>Model Parameter</td>
<td>Units</td>
<td>Pristine</td>
</tr>
<tr>
<td>Span Lengths</td>
<td>ft</td>
<td>61.5 both spans</td>
</tr>
<tr>
<td>Span Mass Placement</td>
<td>-</td>
<td>At top of bent at beam centerlines</td>
</tr>
<tr>
<td>Total span load applied to bent cap</td>
<td>kips</td>
<td>366</td>
</tr>
<tr>
<td>Column Height</td>
<td>ft</td>
<td>30</td>
</tr>
<tr>
<td>Column Cross-section</td>
<td>ft</td>
<td>3.0 (sq)</td>
</tr>
<tr>
<td>Column Transverse Reinforcing</td>
<td>#4</td>
<td>#4</td>
</tr>
<tr>
<td>Column Transverse Reinforcing Spacing</td>
<td>in</td>
<td>12 center-to-center</td>
</tr>
<tr>
<td>Column Concrete cover</td>
<td>in</td>
<td>2</td>
</tr>
<tr>
<td>Column Longitudinal Reinforcement Areas</td>
<td>%</td>
<td>1.2</td>
</tr>
<tr>
<td>Column Hinge Lengths</td>
<td>ft</td>
<td>1.5</td>
</tr>
<tr>
<td>Bent Cap Length</td>
<td>ft</td>
<td>33</td>
</tr>
<tr>
<td>Bent Cross-section</td>
<td>ft</td>
<td>4 x 4</td>
</tr>
<tr>
<td>Bent Transverse Reinforcing</td>
<td>#4</td>
<td>#5</td>
</tr>
<tr>
<td>Bent Concrete cover</td>
<td>in</td>
<td>2</td>
</tr>
<tr>
<td>Bent Hinge Lengths</td>
<td>ft</td>
<td>2</td>
</tr>
<tr>
<td>Concrete unconfined compressive strength, $f'_c$</td>
<td>psi</td>
<td>3000</td>
</tr>
<tr>
<td>Concrete confined compressive strength, $f''_c$</td>
<td>psi</td>
<td>3000</td>
</tr>
<tr>
<td>Reinforcing Steel Grade</td>
<td>40</td>
<td>40</td>
</tr>
</tbody>
</table>

Notes:

The seismic analysis advanced sequentially in time steps not exceeding that of the earthquake ground motion records, which were typically 0.0024 to 0.01 seconds each, with the records typically lasting for 30 to 100 seconds.

The difference in the column heights, shapes, and overall bent configurations was important in the selection of these two bridges for evaluating the influence of those characteristic on the bridge seismic behavior. The bent stiffness calculations described above, providing the computed natural period of vibration, indicate that column heights and shapes
are the primary influence on the bent transverse stiffness, $k$. The span lengths are the primary influence on the mass supported by the bents, $m$, in equation (1). Specifically, the span lengths can vary by a factor of up to two within the chosen bounds. Keeping the column features constant while varying the span lengths from 60 to 115 feet changes the $T_n$ by approximately 30%. Conversely, maintaining a constant span length while doubling the column height from 15 to 30 feet changes the $T_n$ by about 270%.

### 3.2.2 Descriptions of the Bridge Models

The SAP2000, version 17.3, structural analysis software was used to model the bridges for both non-linear static pushover and non-linear seismic time-history analyses. The model non-linearity was incorporated through a moment-curvature representation of hinges placed at the top and bottom of each supporting column and in the supporting bent beams, on the interior of each support column, as shown on Figure 3.4a and b. The moment-curvature was computed for each of the column and bent member hinges based on the section geometry with and without spalling and applied to the two-dimensional models of the bents which were supported on a fixed base. The models were subjected to transverse horizontal seismic shaking from 70 unique ground motion time histories in a non-linear direct integration of the model response for a total of 380 combinations of bridges and ground motions as shown in Figure 3.5. Model input and analysis parameters are shown in Table 3.4.
a) Bridge A model configuration of joints and hinges

b) Bridge B model configuration of joints and hinges

Figure 3.4. Bridge A and B SAP2000 model schematic illustrations
The seismic analysis required solving, via direct integration, for each node, the structural displacements necessary for equilibrium according to the following equation of motion:

\[ M \ddot{x}(t) + C \dot{x}(t) + F(x) = M \ddot{g}(t) \]

(eq 2)

Where:

\( M \) = mass of the individual elements within the structural model
\( C \) = velocity-based damping coefficient applied to the model elements
\( F \) = the displacement position based restoring force on the member
\( \ddot{x}(t) \) = acceleration of individual elements within the model, i.e. inertia, (per time step)
\( \ddot{g}(t) \) = the applied earthquake acceleration at the base of the model (per time step)
\( \dot{x}(t) \) = the velocity of the individual elements in the model (per time step)
\( x(t) \) = the displacement of individual elements in the model (per time step)
Corresponding base shear and member displacements are available at the end of each incremental analysis step, and these are used to evaluate:

- Structure lateral displacement
- Hinge rotations and corresponding moments in the hinges
- Horizontal shear forces at the column bases and at girder bearing level

### 3.2.3 Overall Considerations in the Evaluation

A key motivation for this study was to support development of a rapid system-wide seismic bridge screening approach. Bridge seismic vulnerability results from the potential for overstressing or excessively displacing individual components. System-wide seismic vulnerability screening needs to focus on the most consequential of those and needs to be done within a short schedule and limited staffing budget to be an effective screening tool. Utilizing the NBI database information for screening offers the benefit of a standardized cataloging focused on bridge condition, which is regularly and systematically updated, and so is well-suited for that purpose. Its limitation for screening is that it is a partial catalog of the bridge characteristics, not specifically intended to be used for seismic vulnerability evaluation. This means that where there is no NBI database category to quantify a characteristic directly linked to the seismic vulnerability of an important bridge component, the screening methodology will need to infer vulnerability using one or more other recorded characteristics.

The bridge characteristics observed in post-earthquake reconnaissance as most often seismically vulnerable separate into structurally and geotechnically related categories,
with considerable overlap between some of those characteristics. The structural characteristics relate to column hinge ductility, or lack thereof, and resulting substructure damage vulnerability from excessive column hinge ductility demands. Column hinge related vulnerability also includes column lap splice damage and longitudinal reinforcing bar pullout, at splices, and loss of the foundation and beam end-restraints in the column connections. Structural characteristics also include vulnerable span bearings, especially those which are subject to toppling or excessive sliding. Span unseating potential relates to the structure configuration, principally whether there are simple or continuous spans and secondarily whether there are bent features such as large skews, high pedestals under the beams and girders, and insufficient dimensions of the span supports.

The subsurface soil profile thickness and stiffness at each foundation are key geotechnical characteristics at bridges which influence seismic vulnerability, as these affect the seismic ground motions reaching each substructure from the underlying bedrock, and thus the structural response of each bridge.

This evaluation used the seismic vulnerability of the two reinforced concrete bent substructure models typical of multiple span multiple beam and girder bridges as indicators for the overall seismic vulnerability of this bridge category. These bent frame analyses allowed evaluating column hinge vulnerability, bearing vulnerability, and span unseating potential. This was done by using the computed maximum base shear forces and bent cap displacement computed from two seismic hazard conditions, each considering seismic amplification potential from two site subsurface conditions, and each of those evaluated for pristine and spalled bridge conditions.
The potential for foundation instability due to liquefaction is outside of the scope of this paper, which focuses on the structural response of the bridges.

The reinforced concrete bent vulnerability was evaluated based on the combination of the computed maximum displacement, and inelastic energy dissipation occurring in column and bent beam hinges according to the methodology described by Park and Ang (Park and Ang 1985a).

### 3.2.4 Detailed Considerations in the Modeling

Detailed considerations for representative seismic modeling include accounting for the deterioration and aging effects for the reinforced concrete bents and bearings, and the specific features of the reinforced concrete ductility detailing.

The influence of deterioration in the reinforced concrete bent frames is vital information for system-wide vulnerability rating since these bridges have been constructed on a mass scale since the 1950’s starting with the U.S. interstate highway construction initiative, and earlier instances in individual cases. Exposure to freeze-thaw cycles and de-icing chemicals in northern climates compounds the age-related deterioration potential. Design features such as, until recently, the common use of simple spans with expansion joints situated over the concrete bents in most bridges, exacerbate that deterioration potential in cold climates. Those direct water laden with deicing solutions toward bearing and concrete bent caps. This translates to bridge concrete and bearing conditions within the inventory ranging from slightly weathered to severely deteriorated.
The modeling accounted for reinforced concrete deterioration by means of both a pristine reinforced concrete cross-section, and a fully-spalled model member cross-section, as illustrated in Figures 3.6 a and b. The spalling reduces the mass of the bent frame, and, most importantly, the section modulus and hinge strength and ductility.

![Figure 3.6. Column and beam section illustration with and without spalling](image)

a) Bridge A Column (left) and Beam (right) Sections

b) Bridge B Column (left) and Beam (right) Sections
Deterioration at bearings, from corrosion of the bearings themselves, or deterioration of the concrete at the anchorages, was addressed indirectly through estimates of applied shear loads compared with shear capacity estimates of typical anchorages.

The ductility of the reinforced concrete columns and beams was accounted for with four hinge models representing both the pristine and spalled cross-sections for the two bridges. Ductility in the column and beam hinge rotational capacity is modeled by specifying a moment-curvature relationship for the reinforced concrete based on Mander confinement model parameters (Mander, Priestley, and Park 1988b, 1988a). Bridge A reflects a common square cross-section and widely spaced (i.e., 12-inches on-center) transverse reinforcing configuration of earlier bridges while Bridge B, with round columns and spiral reinforcing at 3-1/2 inches on-center spacing. This close spacing is similar to more recent practice.

The resulting column hinge moment-curvature relations are shown on Figure 3.7. The yield moment strength of the square column concrete section for Bridge A is approximately 35 percent greater than that of the concrete section for Bridge B, explained by the larger section modulus of the square Bridge A column cross-section. There was a higher concrete compression force achieved for Column B resulting from the closely spaced spiral reinforcing, compared with no compressive strength increase benefit from the transverse reinforcing at Column A, but that was not enough to overcome the benefit of the larger square section modulus. The beam section moment-curvature relations are both stronger and stiffer than the columns at either bridge because of larger overall dimensions and increased reinforcing steel compared with the columns.
Figure 3.7. Bridge model column hinge moment-curvature relationships

3.2.5 Ground Motion Time Histories Used

Ground motion time histories were obtained from the Pacific Earthquake Engineering Research (PEER) center NGA West 2 database (“PEER Ground Motion Database - PEER Center” 2018). The time histories were selected to bracket the target design spectra for two seismic hazard conditions as previously mentioned: minimal-to-low, and low-to-moderate. The target spectra were derived per the AASHTO 2009 Guide Specifications for LRFD Seismic Bridge Design (AASHTO 2014, 2017) which correspond to a 7% in 75-year probability of exceedance (1033-year return period).

These target design spectra are shown on Figures 3.8 a-d along with the ground motion ensembles chosen to match those spectra. The chosen hazard conditions represent the range of minimal to moderate seismic hazard within the continental United States, as shown in Figure 3.9. Those hazard conditions cover the region from about the western edge of the Rocky Mountains to the east coast, excluding the higher seismic hazard New Madrid and Charleston, South Carolina regions. This encompasses nearly three-fourths of the continental United States land area.
Figure 3.8.a–d  Seismic Hazard Level 1 through 4 target spectra with ensemble recorded ground motions
Figure 3.9. Extent of most low-to-moderate seismic hazard (PGA < 0.25g) areas in the continental United States.

The second component of seismic hazard, relating to the ground motion amplification resulting from site subsurface conditions, was addressed by selecting target spectra for both seismic site class B, and seismic site class E conditions in accordance with AASHTO seismic bridge design guidelines (AASHTO, 2011), and AASHTO LRFD bridge design standards (AASHTO, 2017). Site class B conditions represent firm ground conditions (i.e., bedrock) with no amplification from expected bedrock ground motions provided in the USGS seismic hazard mapping. Site class E conditions reflect seismic ground motion amplification from bedrock propagating through a soft soil profile. These amplify bedrock motions by up to 3.5 times. The resulting four target spectra reflect both the regional and site condition seismic hazard variability for most bridges in the inventory studied. There would be limited exceptions at sites with either harder rock at the surface or thick deposits with loose liquefiable soils (which require individual site-specific analyses).

Specific attention was given to the filtering used to search and select the ground motions. This was done with the aim of having applied ground motions be as close to what
could be expected at an actual bridge, in the same site conditions. Specifically, motions were obtained from ground surface recordings at sites with either seismic site class B or E conditions. In this approach, the influence of the soil profile on the resulting ground surface motions after propagating through bedrock was included. It is understood that this does not eliminate the influence that differences in regional bedrock and tectonic conditions have on ground surface motions. It does, however, offer a measure of consistency to the source ground motion data applied through this analysis.

The ground motion time history filtering of the PEER database was accomplished by applying specific constraints to obtain motions which met the following criteria:

- All motions were unscaled.
- Motions were selected from source locations which met either Seismic Site Class B or E conditions, based on $V_{s30}$ values at the source sites. (Note: $V_{s30}$ values are the weighted shear wave velocity values corresponding to the top 30 meters of the subsurface profile).
- Motions were from earthquakes of Moment Magnitude 5 to 8, and were not pulse motions (typical of near-fault locations), with a minimum distance to faulting of 5 kilometers, and usually greater than 20 kilometers.

These constraints were used to benefit from the recent data validation made for the PEER ground motion database in terms of well-documented site condition and related source information, and to obtain motions which were as close as feasible in bracketing the range of typical bridge site conditions, namely Seismic Site Class B and E.

The PEER NGA East Coast ground motion database became available during the latter portion of this work and was searched for ground motions meeting the target spectra.
Unfortunately, the available motions are all too low to match the target spectra without scaling, which we chose not to do in this study.

The ground motion summary information is provided in the appendix.

3.3 Analysis Results

The analyses consisted of applying 70 ground motion time-history records in four ensembles of between 22 and 26 motions, with each ensemble corresponding to one of four seismic hazard condition target spectra. Each of the four bridge cases were evaluated for seismic shaking associated with ground motions corresponding to those target spectra. These combinations are illustrated in Figure 3.5. The summary results are described in the following sections.

3.3.1 Static Pushover Analysis

Non-linear static pushover analysis was performed on each of the bridge bent models to estimate the yield and ultimate displacement drift values, and corresponding transverse displacement ductility for each bent frame in both pristine and spalled conditions, corresponding to unfactored dead-loads only. The moment-curvature results for the loading increments, typically amounting to between 12 and 18 increments, were extracted from the SAP2000 analyses into a spreadsheet to calculate the rotational plastic work-energy expended in the pushover using the following formulation:

$$\text{Work-Energy} = \sum_{n=1}^{x} \left[ \left((M_n + M_{n-1}) / 2\right) \ast (R_n - R_{n-1}) \right]$$

(eq 3)

where:

$$x = \text{number of steps to reach rupture of all column hinges}$$
n = loading step (with initial loading condition occurring at n = 0)

\[ M_n = \text{Hinge moment at each loading step} \]

\[ R_n = \text{Hinge plastic rotation at each loading step} \]

Because of the substantially greater beam stiffness relative to the columns, the beam rotations remained in the elastic ranges in the pushover. Pushover shear force versus bent transverse displacement was compared to the rotational hinge work-energy as a computational check, and to graphically illustrate the pushover behavior. The pushover results are on Figure 3.10 and in Table 3.5.

![Figure 3.10. Pushover force-displacement for Bridge A pristine and fully spalled and Bridge B pristine and fully spalled](image-url)
Table 3.5 Bridge deformation and strength properties from pushover analysis

<table>
<thead>
<tr>
<th>Bridge Properties</th>
<th>Bridge A Fully Spalled</th>
<th>Bridge A Pristine</th>
<th>Bridge B Fully Spalled</th>
<th>Bridge B Pristine</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yield Point Deflection</td>
<td>feet</td>
<td>0.106</td>
<td>0.077</td>
<td>0.044</td>
</tr>
<tr>
<td>Yield Point Base Shear</td>
<td>kips</td>
<td>145</td>
<td>160</td>
<td>185</td>
</tr>
<tr>
<td>Ultimate Displacement Capacity</td>
<td>feet</td>
<td>0.48</td>
<td>0.48</td>
<td>0.44</td>
</tr>
<tr>
<td>Maximum Displacement Base Shear</td>
<td>kips</td>
<td>173</td>
<td>188</td>
<td>245</td>
</tr>
<tr>
<td>Displacement Ductility Capacity</td>
<td></td>
<td>4.5</td>
<td>6.2</td>
<td>9.9</td>
</tr>
<tr>
<td>Total Static Pushover Energy Capacity</td>
<td>ft-kips</td>
<td>57.8</td>
<td>65.3</td>
<td>83.1</td>
</tr>
<tr>
<td>Bridge Bent Transverse Tn (initial)</td>
<td>seconds</td>
<td>0.62</td>
<td>0.51</td>
<td>0.48</td>
</tr>
</tbody>
</table>

The pushover analyses indicate that the Bridge B bent frame combinations have greater base shear resistance, stiffness, and displacement ductility compared with the Bridge A bent frame combinations. This is understandable given that bent has added resistance from a third column, and more transverse confining reinforcement in the columns. Additionally, the columns are 20 ft long compared with the 30 ft long Bridge A columns. Table 3.5 and Figure 3.10 provide the comparative results.

The ultimate displacement capacities of the two bridges are similar at between 5.5 and 6.5 inches. The differences in displacement behavior are primarily in the yield displacements, with both pristine and spalled yield displacements for Bridge B at about 40 percent of the corresponding Bridge A yield values. This is reflected in the displacement ductility of the bridges. Bridge B pristine and spalled condition bent frames have displacement ductility’s of nearly 20 and 10, respectively, compared with displacement ductility of 6.2 and 4.5 for the corresponding Bridge A bent conditions.
The pushover results indicate that column deterioration equivalent to spalling of the outer concrete cover increased the lateral displacement yield point by 40 and 60 percent, with a corresponding 10 and 20 percent reduction in column lateral yield resistance, for bridges A and B, respectively. Ultimate displacement and strength capacity were reduced between 0 and 20 percent.

3.3.2 Damage Index

There is generally low potential for seismic damage in the form of cyclic energy induced hinge deterioration in the concrete bent frames for these types of bridges in the low-to-moderate seismic hazard regions of the continental United States, as evaluated in this study. Figures 3.11 and 3.12 illustrate via histograms the likelihood of damage potential thresholds as categorized by Damage Index, based on the seismic hazard conditions, and the bridge type and condition, respectively. Most of the damage was related to maximum displacement, with few instances of significant energy dissipation adding to the total damage index.

![Figure 3.11. Binned Damage Index by seismic hazard level](image-url)
3.3.3 Potential to Exceed Shear Force Capacity

The maximum shear forces developed in the bridges are predominately below the displacement yield values, as illustrated in Figure 3.13. The maximum shear forces developed in the bent frames occur in Bridge B, the overall stiffest bridge.
3.3.4 Potential for Bent Cap Displacement (Drift)

Figure 3.13 also illustrates the maximum base shear force-displacement behavior of the bridge models. The maximum bent cap displacements were usually below yield displacement amounts. Once base shear exceeded yield, the bridge displacements were at or less than those indicated by the static pushover analysis.

3.3.5 Bridge Configuration and Aging Effects

The influence of bridge configurations is inherently accounted for through the four bridge models to which ground motions have been applied. Specifically, the 20 and 30-foot column heights and three versus two column configurations reflect the range of relatively stiffer to more flexible, respectively, bridge conditions in the multiple beam/girder inventory. Comparing the stiffer Bridge B to Bridge A indicates that displacements, base shear forces generated, and damage index values are generally between 1.1 and 2.5 times larger in the stiffer Bridge B. Similarly, comparing between the stiffer, pristine bridges and more flexible spalled bridge cases indicates about the same relationships.

3.3.6 Influence of Ground Motion Characteristics

The analyses show a general relationship of increasing damage potential corresponding to increasing applied ground motion spectral acceleration at bridge natural period. Total Damage Index (DI$_T$) values were all below 0.1 for applied ground motions with spectral accelerations below 0.3 g. Damage Index values increased at higher spectral accelerations, although mostly remaining below 0.2 at spectral accelerations reaching to about 0.6 g. Figure 3.14 illustrates the relationship between the spectral acceleration at the
bridge initial natural period of transverse vibration and resulting DI$_T$ values, for the analyses where the DI$_T$ values were above approximately 0.05. This indicates a general trend of increasing DI$_T$ with increasing spectral acceleration, though with relatively few (18 above DI$_T$ of 0.1) instances of slightly to more elevated DI$_T$ out of the 280 simulations.

![Graph showing DI$_T$ vs spectral acceleration](image)

**Figure 3.14.** Total Damage Index (DI$_T$) in relation to the computed spectral acceleration of the applied ground motion (note: only DI$_T$ values above 0.05 are shown).

### 3.4 Conclusions

This work investigated seismic vulnerability of the older and aging bridges of the United States. The investigated bridges, comprised of multiple span multiple-girder bridges, were built during the interstate highway construction era of the 1950’s and ‘60’s. This bridge type represents 55 percent of multiple span bridges in the U.S, amounting to 160,000 bridges. A majority of these are in low-moderate seismic regions. Analyses considered the spalled-concrete deterioration frequently associated with aging of this vintage of bridges. The analysis focused on low-to-moderate seismic hazard regions, specifically addressing two seismic hazard conditions (low, and low-to-moderate, per the definitions in
Table 3.2), using two sample bridges in pristine and spalled condition, and two site subsurface conditions, a soft and a stiff ground condition, bounding the common range. Analyses excluded considering implications of potentially liquefiable site conditions.

The study findings are applicable to the low-to-moderate seismic hazard geographic area comprising most of the continental United States. The exceptions are the well-documented higher seismic regions of the west coast, mountain west, and the New Madrid and Charleston areas.

The study leads to the following conclusions:

1) The results suggest that the seismic vulnerability, based on seismic hazard corresponding to a 7 percent probability of exceedance in 75 years, of this multiple span with multiple beam/girder bridge inventory in low to moderate seismic hazard regions is overall low.

2) The results also indicate that age-related deterioration leading to loss of concrete section due to spalling creates a relatively minor increase in seismic vulnerability, less than that which can develop from a higher seismic hazard condition resulting from different locations and soil conditions.

3) The pushover yield displacements are smaller for the stiffer Bridge B bent, with similar ultimate displacement capacities of both bridges. This results in ductility capacities which are two to three times greater in Bridge B. This can be attributed at least in part to the closer transverse steel spacing in the Bridge B columns, leading to a greater moment-curvature capacity than the Bridge A columns.
4) The seismic vulnerability for these bridges is comprised primarily of the following:

- These types of bridges are typically configured such that they are strong beam-weak column structures such that the displacement capacity is entirely related to the reinforced concrete column hinge development in the bents.

- Transverse shear forces can reach significant levels, that is, above the nominal lateral restraint values mandated by the design codes, such that bearing damage is plausible, particularly where the bearing anchors have reduced capacity due to corrosion.

- Transverse bent displacements are expected to be relatively low, under two inches, suggesting that girder unseating due to displacement is not a high potential.

The conclusions presented above are subject to the following limitations:

1. These analyses exclude the beneficial influence of bridge girder-deck transverse restraint which will occur when the girders are sufficiently anchored to the concrete bents.

2. The reported seismic vulnerability is for the substructure and superstructure, separate from any bridge vulnerability associated with site soil liquefaction, lateral spreading, or abutment slumping due to weak soils.
3. The conclusions are based on adequate longitudinal reinforcing embedment into the foundation footing or pile cap, and cross-bent and adequate longitudinal reinforcing splice length such that pull-out does not occur at the hinge locations.
CHAPTER 4 SEISMIC HAZARD DEMAND ON AGING MULTIPLE SPAN MULTIPLE-GIRDER BRIDGES

4.1 Introduction

This chapter describes the approach, methods, and results of quantitative analysis of the expected seismic hazard demand on typical multiple-span multiple-girder bridges in low-to-moderate seismic areas of the continental United States. These are compared with current seismic hazard demand thresholds for exempting requirements for seismic design for new bridges, and retrofit for existing bridges. Most low-to-moderate seismic hazard areas in the United States are exempt from requiring specific seismic analysis for design, per current American Association of State Highway and Transportation Officials (AASHTO 2017) seismic design standards for new bridges. A smaller, yet still significant, portion of these areas can be exempt from specific seismic analysis for existing bridge retrofit design, per current Federal Highway Administration (FHWA 2006) seismic retrofit guidelines for existing bridges. These exemptions apply when expected seismic spectral acceleration values are below specific thresholds.

Analyses in this research were performed by applying actual recorded ground motions matched to probabilistically determined smoothed, mean acceleration spectra corresponding to low-to-moderate seismic hazard levels, to models of typical bridges. Matching to these smoothed mean acceleration spectra involves developing ensembles of individual ground motions, each with irregular spectra, which on-the-whole, match the target spectra. This inherently results in using individual ground motions with considerable variability in their spectral accelerations through their period range. This is evident in Figures 4.12a-d, which show the spectral accelerations of ground motions fitting the four
target spectra in this study. Those spectra represent four levels of seismic hazard, bracketing the upper and lower bounds of seismic hazard, with two intermediate levels. Those ground motions have many spectral accelerations well above 0.25 g and as high as 1g in the 0.3 to 0.7 second range, which corresponds to most fundamental periods of this bridge type. These spectral accelerations significantly exceed the 0.15 to 0.25g minimum horizontal design connection force requirements for bridge elements prescribed by AASHTO. They also exceed the FHWA criteria for no-analysis situations, which suggests the threshold criteria may be unconservative, in at least some situations. This study evaluated the resulting base shear forces in the bridge bents from these motions, and the damage potential of those ground motions. These findings can be used to determine threshold values of mapped probabilistic spectral accelerations in the AASHTO and FHWA standards and guidelines, respectively, which appropriately allow for seismic analysis exclusions, based on quantitative estimates of low damage potential for these types of bridges.

### 4.1.1 Seismic Performance Zones and Hazard Levels

New bridge design in the United States is governed by the American Association of State Highway and Transportation Officials, AASHTO, Load and Resistance Factor (LRFD) Bridge Design Specifications, Eighth Edition, November 2017 (AASHTO 2017). The seismic analysis requirements for new multiple-span bridges under AASHTO 2017 depend on the seismic hazard level, the criticality of the bridge under normal and emergency situations within the transportation network, and the regularity, or lack thereof, in the span configuration. There is a four-tier seismic performance zone rating system, cited in Section 3.10.6 of the code. These seismic performance zones are based on the
probabilistic 1-second period spectral acceleration values mapped for the project site and adjusted as applicable, based on the seismic site classification. The degree of sophistication and complexity in the seismic analysis required in each seismic performance zone depends on the combination of seismic hazard level and the other factors noted above.

Existing bridge evaluations and retrofits are not covered by the AASHTO standards, and can be evaluated according to the FHWA Seismic Retrofit Guidance (FHWA, 2006). That guidance has different criteria from AASHTO new bridge design standards (AASHTO, 2017) for determining the seismic analysis requirements and the subsequent retrofit requirements. Those criteria depend on the remaining anticipated service life of the bridge, whether the bridge is essential within the transportation network, the degree of post-earthquake access required from the bridge, and the seismic hazard level. The seismic hazard level is defined in Section 1.5.4 of FHWA 2006, and is based on the probabilistic 0.2-second and the 1-second period spectral acceleration values mapped for the project site and adjusted as applicable, based on the seismic site classification.

The geographic extent over which existing bridges may qualify for the FHWA exclusion is smaller than AASHTO for new bridges. This is because the FHWA seismic retrofit recommendations include two seismic hazard level criteria compared to a single criterion in the AASHTO new bridge design specifications. Discussion on that difference and the implications is in the following section.

The flow chart for establishing the seismic analysis requirements for new bridges according to the AASHTO 2017 bridge design standards is illustrated on Figure 4.1. The flow chart for establishing the seismic analysis requirements for retrofit of existing bridges
according to the FHWA 2006 seismic retrofit guidelines is illustrated on Figure 4.2 for existing bridges.

**Step 1**

Obtain seismic hazard horizontal acceleration coefficient values $S_1$, $S_s$, and PGA for the project site per seismic hazard maps in AASHTO LRFD Bridge Design Specifications.

**Step 2**

Obtain Seismic Site Classification (SSC) for project site through subsurface explorations for 30-meters (100-ft) of profile and corresponding $F_v$ value based on SSC.

**Step 3**

Compute Acceleration Coefficient $S_{D1} = S_1 \times F_v$.

**Step 4**

Determine AASHTO Seismic Performance Zone based on $S_{D1}$.

If $S_{D1}$ is $\leq 0.15g$, then no-seismic analysis is required for a multiple span bridge. Note: No-seismic analysis is required for a single span bridge regardless of $S_{D3}$.

Figure 4.1. Flow chart for establishing seismic analysis requirements for new bridges per AASHTO 2017

**Step 1**

Obtain seismic hazard horizontal acceleration coefficient values $S_1$, $S_s$, and PGA for the project site per seismic hazard maps in AASHTO LRFD Bridge Design Specifications.

**Step 2**

Obtain Seismic Site Classification (SSC) for project site through subsurface explorations for 30-meters (100-ft) of profile and corresponding $F_v$ value based on SSC.

**Step 3**

Compute Acceleration Coefficients $S_{D1} = S_1 \times F_v$ and $S_{D3} = S_s \times F_a$.

**Step 4**

Determine anticipated remaining service life (ASL) for bridge and Performance Level (PL) of bridge based on whether bridge is standard or essential and ASL.

**Step 5**

Determine Seismic Performance Zone based on $S_{D1}$ and $S_{D3}$.

If both $S_{D3} \leq 0.15g$ and $S_{D1} \leq 0.15g$, then no-seismic analysis is required if PL = 1, and no seismic analysis is required, regardless of PL, if ASL = 0 to 15 years.

Figure 4.2. Flow chart for establishing seismic analysis requirements for retrofit of existing bridges per FHWA Seismic Retrofit Manual (FHWA 2006)
Seismic design ground acceleration values depend on the hazard level, relating to the geographic location of the site, and the amount of amplification occurring through the soil profile overlying bedrock, relating to the seismic site classification. The seismic site classification depends on the stiffness of the top 30 meters of the subsurface profile and is the currently accepted measure to quantify the amount of potential amplification expected from the earthquake motions originating in the bedrock. Softer soil sites serve to amplify the seismic motions transmitted from bedrock more than stiffer soil sites.

4.1.2 No-Analysis Exclusion

The current AASHTO LRFD bridge seismic design specifications and guidance (AASHTO 2014, 2017) provide exclusions allowing for no-minimum seismic analysis being required in most low to moderate seismic hazard locations. All new single span bridges are exempt from seismic analysis requirements.

For multiple-span bridges, these exclusions apply to geographic locations which satisfy the AASHTO Seismic Performance Zone criteria of 1-second spectral acceleration of not exceeding 0.15g, as described in the previous section. These locations are identified through the year 2002 seismic hazard maps prepared by the United States Geological Survey (USGS) for the 2009 AASHTO Bridge Design Specifications (AASHTO 2009), and subsequent versions (AASHTO 2014, 2017).

This exclusion can apply to most of the continental United States, excluding the higher seismicity Pacific coast, central Rocky Mountain, New Madrid, and Charleston regions. Figures 4.3a-d illustrate the locations where the AASHTO no-minimum seismic analysis applies depending on the seismic site class conditions at the bridge location.
Figure 4.3. a-d Seismic Design Category (SDC) extents depending on Seismic Site Classification. Per AASHTO 2014 no-seismic analysis is required in green designated locations. Extents depend on Seismic Site Class (SSC) at bridge location (a) SSC=B (b) SSC=C (c) SSC=D (d) SSC=E.

The exclusion is predicated upon the bridges having a limited degree of seismic force restraint built into them through minimum design requirements for horizontal restraint. The AASHTO (AASHTO, 2014, 2017) design criteria have minimum design requirements depending on the design seismic hazard acceleration coefficient value. The lowest required horizontal restraint occurs where the horizontal acceleration coefficient, equal to the peak seismic ground acceleration (PGA) multiplied by the amplification factor corresponding to the seismic site classification, is less than 0.05g. In those locations, “…the horizontal design connection force shall not be less than 0.15 times the vertical reaction due to tributary permanent load and the tributary live loads assumed to exist during an earthquake.” The connection force multiplier on vertical tributary loads is specified at 0.25 where the PGA is greater than 0.05g. Similarly, there are requirements for minimum
superstructure support lengths, to accommodate potential for spans sliding off supports due to earthquake shaking.

The AASHTO no-analysis criteria for new bridges result in situations where the minimum required horizontal restraint of either 0.15 or 0.25, as applicable, multiplied by the vertical reaction of the tributary load can be less than the design spectral acceleration for the short period (0.2-second period) seismic hazard level specified in the AASHTO 2017 short period (0.2 second) mapped hazard level, and accounting for seismic site classification. Figure 4.4 illustrates 37 representative municipalities in the continental U.S. where the no-analysis exclusion can potentially apply, depending on the seismic site classification at the bridge location. Table 4.1 lists for each of these locations how many of the 5 possible seismic site classifications (A through E) meet the no-analysis exclusion, as well as for how many of these site classifications the no-analysis exclusion conflicts with the mapped seismic hazard.

![Figure 4.4. Representative locations where AASHTO LRFD no-seismic analysis exclusion (AASHTO, 2017) can potentially apply. See also Table 4.1 for details.](image)
Table 4.1 Example locations where AASHTO LRFD no-analysis exclusions can apply and how many of five possible seismic site class conditions where AASHTO specified design spectral acceleration values exceed the 0.25 times the vertical connection load specified for no-seismic-analysis situations applicable for AASHTO Seismic Zone 1. Refer to Figure 4.4 for a location map.

<table>
<thead>
<tr>
<th>City</th>
<th>State</th>
<th>Number of seismic site class cases where the AASHTO seismic analysis exclusion applies based on $S_{D1}$ Acceleration Coefficient ≤ 0.15 g</th>
<th>Number of corresponding seismic site class cases where the short-period, $S_{DN}$ Acceleration Coefficient Exceeds 0.25g</th>
</tr>
</thead>
<tbody>
<tr>
<td>Atlantic City</td>
<td>NJ</td>
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</tr>
<tr>
<td>Augusta</td>
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<td>3</td>
<td>1</td>
</tr>
<tr>
<td>Birmingham</td>
<td>AL</td>
<td>4</td>
<td>1</td>
</tr>
<tr>
<td>Boise</td>
<td>ID</td>
<td>3</td>
<td>0</td>
</tr>
<tr>
<td>Boston</td>
<td>MA</td>
<td>5</td>
<td>1</td>
</tr>
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<td>VT</td>
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<td>1</td>
</tr>
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<td>5</td>
<td>1</td>
</tr>
<tr>
<td>Champaign</td>
<td>IL</td>
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</tr>
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</tr>
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<td>0</td>
</tr>
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<td>Greenville</td>
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</tr>
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<td>Knoxvile</td>
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<td>AK</td>
<td>2</td>
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<td>Manhattan</td>
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<td>Monroe</td>
<td>LA</td>
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<tr>
<td>North Hero</td>
<td>VT</td>
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<td>2</td>
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<td>2</td>
</tr>
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<tr>
<td>Pittsburgh</td>
<td>PA</td>
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<td>0</td>
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<tr>
<td>Raleigh</td>
<td>NC</td>
<td>4</td>
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<tr>
<td>Reeves</td>
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</tr>
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<td>Rockland</td>
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<td>1</td>
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<td>Spokane</td>
<td>WA</td>
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<td>1</td>
</tr>
<tr>
<td>Stamford</td>
<td>CT</td>
<td>5</td>
<td>2</td>
</tr>
<tr>
<td>Washington</td>
<td>DC</td>
<td>5</td>
<td>0</td>
</tr>
</tbody>
</table>
A conflict arises with the AASHTO seismic analysis exclusion from the single 1-second period criterion of the spectral acceleration without considering the short-period (0.2-second mapped values) spectral acceleration and bridge characteristics including the natural period. This leads to situations, examples of which are identified in Table 4.1, where the nominal 0.15 or 0.25g minimum horizontal restraint capacity design requirements for are less than the mapped design mean spectral acceleration for that bridge location. This creates a contradiction between what need to be conservative nominal horizontal restraint requirements compensating for the seismic analysis exemption and a higher mapped seismic hazard force, in some areas.

The FHWA seismic retrofit no-analysis exclusion differs from the AASHTO exclusion in two main aspects. It is based on both 1-second spectral acceleration and 0.2-second period spectral acceleration. Both spectral acceleration values cannot exceed 0.15g for a design earthquake with probability of exceedance of 7% in 75-years (approximately 1033-year return period), to obtain the exclusion. Also, the exclusion applies to whether retrofit analysis and corresponding design is required. In some cases, there could be no applicable compensating minimum horizontal restraint capacity design requirements in the FHWA 2006 guidelines. On the one-hand, the addition of the second, 0.2-second period criterion, reduces the contradiction arising in the AASHTO exclusion criteria, as discussed above. Understandably, these more restrictive criteria reduce the geographic extent of the no-analysis exclusion potential. On the other hand, there can be exemption from seismic analysis required to evaluate if minimum seismic protection capacity is available in a bridge. Figure 4.5 illustrates the potential extents of the FHWA no-analysis exclusion.
Figure 4.5. Approximate extent of reduction in AASHTO LRFD no-seismic analysis zone with application of FHWA 2006 seismic retrofit criteria including 0.15g max spectral acceleration at SDS. Shaded area represents reduction. Refer to Figures 4.3a-d for AASHTO LRFD limits

4.1.3 Bridge Seismic Vulnerability

Bridge seismic vulnerability results when the displacement or strength capacities in the bridge elements are exceeded by the earthquake induced displacements or forces, respectively. Excessive displacement vulnerabilities most commonly occur with insufficient span support dimensions, especially with simply supported spans, and with insufficient displacement ductility in the substructures, such as in the bent support columns, and with bearing types which can rotate or slide excessively or topple. Seismic overloading vulnerabilities are most commonly associated with inadequately reinforced concrete columns, footings, abutments. Vulnerability also results from bearings with under-strength connections between spans and supports, allowing spans slide off supports and topple. These features have been identified through post-earthquake inspections and recognized as priority characteristics for screening bridges for seismic vulnerability (Buckle, 1991; Buckle and Friedland, 1995; FHWA 2006)
4.1.4 Applicable Bridge Inventory

Most of the U.S. bridge inventory, at 55%, is comprised of multiple beam/girder bridges, as illustrated in Figure 4.6. This figure also illustrates that another 40% of bridges are a combination of four types: slab deck; T-beam spans; and channel and box beam spans, which typically share a common substructure configuration matching that of beam/girder bridges. The common reinforced concrete beam-column bent substructures of most multiple span bridges allows seismic vulnerability being evaluated for beam/girder bridges to be expandable to most type multiple-span bridges.

Figure 4.6. – Summary of U.S. multiples bridge types
4.2 Considerations in the Analysis

4.2.1 Project Approach

The objective of this research was to evaluate seismic performance of typical bridges subjected to four levels of seismic demand within the range considered to be low-to-moderate seismic hazard. This study used models of actual bridges designed and constructed in the early-to-mid 1960’s. These bridge models were developed from record plans of two actual bridges constructed in the 1960’s in the northeast U.S. Those were designed with low seismic design requirements. Those prescribed seismic loads between 2 and 6 percent of the structure weight, which are well below the 0.15 to 0.25 horizontal restraint factor required for seismic no-analysis situations in the current design criteria. Although the bridges have not explicitly been designed for the higher seismic forces applied in this study, they have performed with limited deformations and damage identified in the analyses. Consequently, they are conservative proxy for newer bridges specifically designed for higher seismic forces.

The other important feature of this study is that the bridges were evaluated in a pristine and spalled condition, to include aging affects.

The considerations for the analysis are described in the following sections.

4.2.2 Non-linear Dynamic Numerical Modeling

The analysis involved performing non-linear dynamic numerical modeling of typical multiple span bridge configurations in both pristine and deteriorated conditions, by
subjecting them to several levels of seismic ground motions, to evaluate their seismic performance. These analyses provided expected reinforced concrete bent transverse displacements and shear forces resulting from 70 unique recorded earthquake motions in each of the four bridge models. The analyses were performed on each of the four bridge models (two bridges each with pristine and spalled configurations) using four ensembles of ground motions representing progressively higher seismic hazard from low to moderate. The bridge models span a 0.35 to 0.62 second natural, pre-shaking, period range.

The SAP2000, version 17.3, structural analysis software following the procedures described in Section 3.2.2. The bridge responses to individual ground motions were compared with the computed yield and ultimate displacement and force capacities for each of the bridge models obtained with static pushover analysis. The results directly address vulnerability characteristics of reinforced concrete column displacement and strength capacity, resulting base shear forces in the bents, and damage developed in the reinforced concrete column hinges for those exceeding hinge yield conditions.

4.2.3 Choosing Representative Target Spectra

The AASHTO and FHWA seismic analysis and design procedures use probabilistic mean values of expected ground motion in the form of seismic acceleration response spectra. These smoothed spectra for a bridge site depend on the probability of occurrence of the earthquake (i.e., the return interval), which is a function of the geographic location (relative to recorded seismic activity), and the amplification characteristics of soils overlying the bedrock, for which spectral acceleration values computed. This study required choosing four target spectra representative of low-to-moderate seismic hazard
zones, which were distributed between the upper and lower bounds of the hazard zones. Several considerations factor into the spectra, which warrant discussion.

The concept of elastic earthquake response spectra resulting in a unique spectral acceleration relationship associated with a collection of earthquake ground motions expected at a particular location originated with M.A. Biot in 1932 (Biot 1932; Chopra 2007). George Housner (Housner, 1952) furthered the concept in the 1940’s, and the elastic design response spectrum was adopted into widespread engineering design practice in the 1970’s and 1980’s following work published by Newmark and Hall (Newmark and Hall, 1982) on applications originally focused on nuclear power plant design and subsequently expanded.

Response spectrum analysis was incorporated into AASHTO bridge design practice in the first edition of the Guide Specifications for LRFD Seismic Bridge Design in 2009 (AASHTO 2009). These remain in use through the current 2017 version of the AASHTO bridge design standards. That specification was a major revision of prior AASHTO seismic design guidance, and reflected a large body of research and development aimed at incorporating knowledge acquired through investigation and research following large earthquakes in the 1980s through mid-1990’s. This new guidance included designing for a larger, less-frequent earthquake probability, and using a spectral acceleration criterion for a no-seismic analysis threshold. That threshold incorporated the hazard associated with both geographic location, and amplification of seismic ground motions in bedrock through overlying soil via categorization of seismic site characteristics.

The spectral acceleration values for this analysis were derived from a probabilistic seismic hazard mapping performed by the USGS for the AASHTO 2009, and current
(AASHTO 2017) bridge design specifications. The hazard maps provide expected peak ground acceleration, and 0.2 and 1-second period spectral accelerations (for an elastic one-degree of freedom system with 5-percent damping) based on regional seismicity history, and ground motion attenuation relationships, for bedrock (AASHTO seismic site class B). They are probabilistic mean acceleration values. Details of the 2002 mapping are described in the narrative accompanying the maps (“USGS 2002). There are tabulated adjustment factor corresponding to each site seismic site class, A through E. These three points are used to construct a smoothed design response spectrum, the construction details of which shown in Figure 4.7.

Figure 4.7 – AASHTO smoothed design spectrum construction method (AASHTO, 2018)

Establishing the design seismic demand for a typical bridge structure is currently most often accomplished by first defining the target spectral accelerations in a region referenced to an exposed bedrock site condition. Those spectral acceleration values are developed to correspond to a defined probability of exceeding the hazard occurrence. For AASHTO, this probability of exceedance for new designs is 7% in 75 years, corresponding to an approximately 1033-year return period. This is the same earthquake applied as the
upper level earthquake in 2006 FHWA seismic retrofit guidelines. The spectral acceleration values are derived using ground motion prediction equations which are statistical models having inputs of earthquake source locations, intensities, and probabilities of occurrence to provide three points from which to define the design spectra. The models account for the expected seismic source motion attenuation and filtering, and with some models, the characteristics of the source motions such as the type and direction of fault movement. The resulting spectral acceleration mapping across a region provides for a uniform hazard depiction.

Numerous seismographic models have been developed to predict probable spectral acceleration for broad-based regional hazard mapping. A combination of these (Atkinson and Boore 1995, Campbell 2002 Frankel, Mueller, Barnhard, Perkins, Leyendecker, Dickman, Hanson, and Hopper 1996, Somerville, Collins, Abrahamson, Graves, and Saikia, 2001, and Toro, Abrahamson, and Schneider, 1997) in a decision tree approach is in the AASHTO bridge design standards. This approach compensates for the tendencies of models to be individually biased toward various aspects of source motions or resulting site responses.

Although seismic hazard models exist for Central and Eastern North America to predict earthquake ground motions which account for the earthquake source types, and attenuation in this particular intraplate region (Mahani and Atkinson 2013), there were no sufficiently strong motions available to within this intraplate region. Consequently, this study included reviewing published work comparing intraplate and interplate (Arango et al., 2012; Stirling and Petersen 2006; Hoult, Lumantarna, and Goldsworthy 2013; Lumantarna, Wilson, and Lam 2012) earthquake ground motions to answer the question of
whether using interplate records was justifiable for this study. The main comparisons are
between intraplate Central and Eastern North America, North and Northwestern Europe,
and Australia, and the interplate areas essentially bounding the Pacific Ocean, and
boundaries of the Asian and Indian plates. The overall findings from these comparisons
indicate that precise distinctions between ground motions from intraplate and interplate
locations are not evident.

4.2.4 Ground motion sources and ensemble selection

Once the target spectra were established for a location and chosen seismic hazard
level, the randomness of each of the potential earthquake motions was captured by creating
ensembles of ground motion time-history records to apply to structural models. There are
currently hundreds of vetted and well-documented recorded earthquake ground motions
assembled into databases such as the Pacific Earthquake Engineering Research (PEER)
institute, which provide recorded ground motions suitable for time-history analyses of
bridge models. Despite an overall large data set, these motions are clustered in areas which
have relatively higher seismicity. This limited the numbers of non-linear time-history
records matching the target spectra which met the criteria of being unscaled and otherwise
unaltered, and from a corresponding seismic site class location. This also meant that there
are no suitable unscaled motions in the NGA West 2 or NGA East database from intraplate
regions.

The seismic site classification is based on the soil site characterization in the top 30
meters of the subsurface profile. The PEER database references the weighted shear wave
velocity within that profile, referred to as $V_{S30}$. This allowed searching for records within
the database which met the $V_{S30}$ criteria corresponding to the Seismic Site Class B and E
profile conditions that were evaluated in this study. These seismic site classes were chosen to bracket the lower and upper bound, respectively, of amplification from bedrock to bridge foundation level, considered to be at about ground surface.

Design seismic hazard levels by geographic location for the AASHTO and FHWA procedures, are mapped for seismic site class B conditions, which consist of bedrock. There is a site class A condition, corresponding to hard crystalline bedrock, which corresponds to an approximately 20 percent decrease in acceleration values, which can be applied where conditions warrant. The upper bound acceleration values are for seismic site class E conditions. Those occur when the soils are loose silts, soft clays, and very loose sands. Sites with soils considered liquefiable or with sensitive clays are classified as seismic site class F. These require site specific seismic analysis, and were beyond the scope of this study.

There are limited options for acquiring numerous ground motion records for large scale simulations because ground motion records are influenced by regional geologic characteristics influencing attenuation, and sources differ depending on the region’s location relative to faulting and the type of faulting. This challenged this study, which was focused on the already scarce amount of seismicity in the study region, which has no unscaled ground records in the range of interest. The options are principally governed by the number of ground motions desired, and whether alterations to existing records or synthetically generated motions are acceptable. Many utilize synthetically generated motions developed to match a target spectrum for the practical reason of needing significant numbers of motions for simulations spanning a broad range of intensities. The most current consensus of practice in the seismic engineering research community tends
toward using scaled and synthetic motions. Guidance on using synthetic motions is available in sources such as (Bradley et al. 2017).

Although the scaled ground motions tend to be used most in studies to obtain many motions, in this case it was considered best to use only unscaled motions, and motions matching the seismic site class B and E criteria. Compensating for the bias in availability of unscaled ground motions, and ground motion time records from interplate locations is the number of ground motions applied to each of the models in this study. Current practice standards (AASHTO 2011, 2017, ASCE-7 2016, Bommer and Acevedo 2004) specify using between three and seven ground motions for site specific analyses, with the envelope of the ground motion spectral responses preferably relatively tightly clustered to the target spectra. These references investigated for this study suggest that the present state of knowledge and availability of recorded ground motions warrants reasonable care but not excessive selectivity in gathering recorded motions for large scale analyses, such as for this study. This suggests that the variability achieved by using 22 to 26 ground motions per hazard level in this study diminish the potential bias effects from using the only available unscaled motions meeting the targets, that is, the interplate ground motions.

The selection of recorded ground motion records followed available published guidance (Bommer and Acevedo 2004; Gomes, Santos, and Oliveira 2006; Shome et al. 1998). Principal recommendations, from (Bommer and Acevedo 2004), followed in this work sought crustal earthquakes and avoided concentrations of motions from too few sources. There were 41 sources for the 70 unique ground motions used in this study. In some cases, there were two ground motion records, orthogonal to each other, which used from a source, while in other cases there was only one motion used from a source. The
ensemble selections were based on trial and error, by viewing the collection results of candidate spectra retrieved from numerous PEER database searches directed at each of the four target spectra. The approach provided the closest appearing matches of individual motions to portions of the target spectrum in our range of interest between approximately 0.3 and 0.7 seconds. The motions were selected to achieve, to the extent possible, ensemble mean spectral values at or above the mean target spectra. This was achievable in most instances, as shown in Figures 4.12a-d.

4.3 Specific Analysis Procedures

The analysis was comprised of the following specific procedures:

4.3.1 Selecting Target Spectra Locations and Values

The four target spectra used for ground motion selection corresponded to those bounding the low-to-moderate seismic regions of the continental United States, per AASHTO seismic design criteria. Figure 4.8 shows the extent and boundaries of the AASHTO seismic no-analysis zone. This figure illustrates that the no-analysis zone comprises most of the continental United States, being bounded on the west approximately by the foothills of the Rocky Mountains, and with two exclusions. The largest exclusion is the combination of the New Madrid Seismic Zone and the Charleston South Carolina Seismic Zone, and the other is the extreme northeast including northern New York and northern New England states.
The four spectra are as follows:

- Seismic Hazard Level 1 – This corresponds to the 0.04 g spectral acceleration at 1-second period.

- Seismic Hazard Level 2 - This corresponds to a 0.06g spectral acceleration at 1-second.

- Seismic Hazard Level 3 – This corresponds to a 0.14g design acceleration coefficient at 1-second.

- Seismic Hazard Level 4 – This corresponds to a 0.22 g spectral acceleration coefficient at 1-second.

The mapped locations of Seismic Hazard Levels 1 and 2, along with the approximate 0.25g PGA contour are shown in Figure 4.8.

The design target spectra are also shown on Figure 4.9.
4.3.2 Search and Select Ground Motion Ensembles

The ground motion time-history records search utilized the Pacific Earthquake Engineering Research (PEER) center NGA West 2 database PEER 2018). The objective was to use as many motions as were available to achieve an ensemble mean value which reasonably matched the target spectra, while constraining the selection to ground motions which as closely as possible with the inventory available could be considered unaltered, matching the seismic hazard regime in terms of moment magnitude of the sources, and matching the seismic site classification to which they would be applied. In summary, the selected ground motions meet the following criteria:

- All motions were to be unscaled.
• Motions were to be selected from source locations which met either Seismic Site Class B or E conditions, based on $V_{s30}$ values at the source sites. (Note: $V_{s30}$ values are the weighted shear wave velocity values corresponding to the top 30 meters of the subsurface profile).

• Motions were to be from earthquakes of Moment Magnitude 5 to 8, were not to be pulse motions, and with a minimum distance to faulting of 5 kilometers, and preferably greater than 20 kilometers.

While moment magnitudes above approximately 6.5 are considerably higher than expected in this region, they are still close to the magnitude 7.5 earthquakes that have been hypothesized for the New Madrid events in 1811-1812, (Hough 2009).

Figures 4.10 a - d provide the target spectra, individual ground motion spectral accelerations, and the mean and standard deviation values of the ensembles. The ensemble mean values for Low-to-Moderate Seismic Hazard Levels 1 through 3 are relatively close to, and in some period ranges, moderately above the target spectra. This occurs at Low-to-Moderate Seismic Hazard Level 4 in the range from approximately 0.5 seconds and above. The spectral accelerations at lower periods are between approximately 0.4 and 0.5 g, slightly above the other ensemble mean peak spectral accelerations, but well below the target spectral short period acceleration of 0.67g for this hazard level. This results from the absence of enough ground motion records from soft soil profile locations, seismic site class E sites, which have spectral accelerations high enough to achieve that mean target value.
a) Seismic hazard level 1

b) Seismic hazard level 2

c) Seismic hazard level 3

d) Seismic hazard level 4

Figure 4.10. a – d  Seismic Hazard Level 1 through 4 target spectra with ensemble recorded ground motions
4.3.3 Applying Ground Motions to Representative Bridge Models

The collection of 70 unique ground motion records obtained from the PEER database was organized into four ensembles, one per seismic hazard level, and applied to the four bridge models. The ground motion time history records were applied individually to each of the finite element bridge models in the SAP2000, version 17.3 structural analysis program. The models incorporated non-linear responses extending past elastic yield range by means of non-linear hinges located at the top and bottom of each column, and on either side of the column-hinge joints in the bent cross beams. These hinges allowed modeling the bridges for elastic response at low seismic loading and then to potential yield of hinges at increased seismic loading, and up through potential collapse under higher seismic loading.

The response of the bridges was evaluated in terms of estimated lateral displacements in the supporting bridge bents, and the maximum base shear forces developed. Lateral displacements were evaluated in terms of whether yield displacements were reached, and if so, how much past yield, as well as the degree of repeated displacement past yield. This latter aspect represented energy dissipated into the structure to cause deterioration, and potentially rupture and collapse. This was recorded as a Damage Index, following the criteria developed by Park and Ang (Park and Ang 1985b).

4.3.4 Data Analysis

The data analysis consisted of a time-history record of both displacement and base shear forces for each bent model, at each time interval of loading, which ranged between 0.0024 and 0.01 seconds per time step, with ground motion records lasting between 30 and 100 seconds. Response through each entire earthquake record where computed
displacements did not exceed the yield value was recorded as a no-damage response, i.e. Damage Index (DI) = 0. The accumulated damage resulting when displacements exceeded yield values was accounted through a cumulative DI comprised of two parts. The first part consisted of the maximum amount of displacement past yield relative to the possible displacement before reaching collapse. The second part consisted of the accumulated hysteretic energy dissipation occurring for each displacement beyond yield relative to the total energy dissipation capacity of each bridge model before reaching collapse.

The maximum base shear values occurring during each earthquake time history were recorded in the models along with the displacement records. Dividing these base shear force values, which occur at the base of the models, by the combined weight of each bridge and supported span provides base shear in terms of horizontal acceleration. This allowed comparing the base shear values with target response spectra for the ensemble motions.

The data analysis also included recording the resulting DI for each ground motion along with the elastic spectral acceleration value for that ground motion at the natural period for each bridge. This provided the ability to evaluate the degree of expected damage based on the spectral acceleration value and considering the shape of the elastic spectral acceleration for each ground motion time history.

4.4 Results and Discussion

This work investigated the relationship between four ground motion ensembles compiled to match low-to-moderate seismic hazard target design spectra and their resulting demand impacting seismic performance of typical multiple span multiple beam/girder bridge bents. Those bridge bent configurations are typical of most of the multiple span bridge inventory in the United States. The focus was on investigating whether there is a
threshold of ground motion spectral acceleration for incurring bridge damage. The purpose was to compare that threshold to the current no-analysis thresholds for seismic analysis and design, which are currently part of the AASHTO (AASHTO 2017) and FHWA (FHWA 2006) seismic design standards and retrofit guidance, respectively. This was evaluated by applying ground motions which matched target spectra for four ground motion seismic hazard levels applicable for low-to-moderate hazard regions in the United States, following the AASHTO 2017 criteria and smoothed design spectrum construction format.

The results from applying seventy unique ground motions, in four ensembles of seismic hazard levels, to four bridge models provided insights to three aspects of concrete bridge bent performance: base shear forces developed, displacement demand, and damage levels reflected in damage index criteria.

**Base shear forces** – The maximum base shear forces developing in the reinforced concrete bents follow a nearly 1:1 correspondence with the spectral accelerations of each applied ground motion matching the initial period of vibration of the bents, up to approximately 0.25g. The developed maximum base shear forces in the bents follow a non-linear trend at higher spectral accelerations, corresponding to cyclic energy dissipated in the bents upon reaching displacement yield. Peak base shear forces reached about 0.37g, corresponding to spectral acceleration values reaching 0.9 g. This is illustrated in Figure 4.11.
Figures 4.12a-d illustrate the relationships between the four target spectra corresponding to the four seismic hazard levels investigated, and the mean and maximum base shear values developed in the concrete bridge bent models. These results show that at the lowest seismic hazard level, the corresponding ensemble ground motions induce a mean base shear slightly below the target spectrum spectral acceleration. However, the maximum generated base shear values extend well above, and reach about 0.25g at the lowest natural period (i.e., stiffest) bridge case of approximately 0.4 seconds.
a) Seismic hazard level 1

b) Seismic hazard level 2

c) Seismic hazard level 3

d) Seismic hazard level 4

Figure 4.12 Seismic Hazard Level 1 through 4 target spectra with ensemble recorded ground motions and resulting bridge base shear forces
In contrast, the three higher seismic hazard level ground motion ensembles result in successively higher mean base shear forces, but, due to the seismic energy dissipation occurring after the structures reach yield as noted above, the maximum base shear forces all crest similarly between about 0.3 and 0.37g, as illustrated in Figure 4.12.

Figure 4.13 provides a histogram showing the relationship between the maximum seismically generated base shear with increasing seismic hazard levels of the ground motion ensembles, in this case reported as a fraction of yield base shear. This shows that only the seismic hazard level 1 ensemble motions generated maximum base shear below the yield values. The highest base shear to yield shear ratio was 1.4 and was reached with ground motions from each of the other seismic hazard level ensembles.

![Figure 4.13](image1.png)

**Figure 4.13. Histogram of maximum developed base shear as a fraction of yield base shear from analyses performed from ground motions at each hazard level.**
**Displacement demand** - Figure 4.14 illustrates in histogram form the relationship between the increasing amount of maximum bridge bent seismic lateral displacement with increasing seismic hazard level. As spectral acceleration, and ground motion energy levels increase with seismic hazard level, the likelihood increases for ground motions to cause structure displacements above yield. As with base shear, only seismic hazard level 1 ensemble motions resulted in maximum displacements below the yield values.

![Histogram of maximum displacement as a fraction of yield displacement from ground motions in each hazard level](image)

**Figure 4.14.** Histogram of maximum displacement as a fraction of yield displacement from ground motions in each hazard level

**Damage Index** - Figure 4.15 illustrates the spectral accelerations at the natural period of each of bridge model of individual ground motions which resulted in total Damage Index (DI$_T$) values exceeding 0.05, the generally recognized threshold of noticeable post-yield induced damage. The DI$_T$ range used in this study corresponds to no damage at DI$_T$ = 0, and collapse at DI$_T$ = 1.0. This illustrates that the threshold of spectral acceleration causing noticeable minor cracking type damage (DI$_T$ ~ 0.1) is approximately
0.3 g, and that spectral accelerations reaching 0.9g reach the potentially moderate damage range.

![Figure 4.15](image.png)

**Figure 4.15.** Total Damage Index (DI_T) in relation to the computed spectral acceleration of the applied ground motion (note: only DI_T values above 0.05 are shown).

The bridge models used in this analysis represented designs from the 1960’s where their seismic design requirements were generally lower than the design requirements in current practice, for which this study applies. This could create inherent conservatism in the damage results and should be considered when interpreting these findings.

### 4.5 CONCLUSIONS

The non-linear time history analysis results described above identified how threshold spectral acceleration values correlate with bridge response and damage potential and provide a basis for recommending one or more different thresholds corresponding to a minimal seismic damage potential with a seismic analysis exclusion. This work included defining for which bridge types and conditions, and geographic locations the minimal damage threshold applies.
A critical aspect of this study was using only unscaled recorded ground motion records from sites which had the same seismic site classifications, namely B and E for this study, upon which the target design spectra are based. This incorporated the actual effects of the source site soil profile on the resulting applied ground motions. This allowed examination of how ground motions from different seismic site classes affect bridge response.

The study led to the following conclusions:

1. There is an overall low likelihood of moderate or greater damage to the reinforced concrete bents in the non-seismically designed bridges evaluated in this study. In most cases, there is a low likelihood of reaching minor damage such as limited cracking of reinforced concrete, as illustrated in Figure 4.15, where only 18 of the 280-ground motion and bridge combinations reached a total damage index above 0.10. This suggests that newer bridges designed specifically for seismic loading will perform at least as well, and most likely better, in low-to-moderate seismic hazard regions. In addition, bridge bents designed with seismic detailing to accommodate ductile behavior, such as closely spaced transverse confinement steel in reinforced concrete columns, as in Bridge B, are expected to respond without catastrophic damage in earthquakes up to the approximate 1000-year return level, which is the current design standard.

2. The base shear forces expected in low-to-moderate seismic hazard regions will mostly exceed the two-to-six percent nominal lateral force requirements in the AASHTO design codes in the 1960’s. In most instances, connections designed
to those levels of lateral restraint will likely be taxed to their capacity or greater. The exceptions could be at the lowest seismic hazard level ground shaking, as shown in Figure 4.12a where the mean values of expected base shear imposed on this type bridge in seismic hazard level 1 range from 0.02 to 0.04 g in the initial period range of typical bridges of this type. Seismic hazard level 2 through 4 mean base shear value shown in Figures 4.12b-d, respectively, are well above the 2 to 6 percent nominal lateral force values from the 1960’s versions of the AASHTO design codes.

3. There is low likelihood of reinforced concrete bent damage exceeding minor levels, exceeding a total Damage Index, DI\textsubscript{T}, value of about 0.15, where the spectral acceleration of a ground motion at the bridge bent natural period is below 0.35g, as shown in Figure 4.15. This is limited to seismic hazard level 1 locations. The smoothed seismic design spectra through seismic hazard level 3 are below 0.35g, but spectral accelerations of actual ground motions paired to the seismic hazard level spectra are erratic and often much higher. Figures 4.12 b, c and d show how individual motion spectra reach substantially higher values above the mean target values. Achieving a low likelihood of damage requires the actual spectral acceleration at short periods to not exceed 0.35g, which means the smoothed design spectrum must be lower.

4. Although the mean values of the seismically generated base shear forces meeting low-to-moderate seismic hazard levels are close to the AASHTO lateral force requirement of 0.25g, there are many motions which exceed that force.
The tendency increases with decreasing periods, and while this study had a lowest period structure at about 0.4 seconds, other stiffer structures, or portions of structures such as locally shorter piers, are likely to exist. This indicates that the 0.25g lateral force requirement may be unconservative in some situations.

5. The FHWA Seismic Retrofit Guidelines (FHWA 2006) approach utilizing both the 0.2-second and 1-second spectral acceleration values for analysis thresholds offers a more conservative threshold for no-analysis, since spectral accelerations are generally higher at shorter periods than at 1-second. The results of this study lead to recommending a period-bounded spectral acceleration criterion, either exactly following the FHWA two-period criterion, or at the natural period of the bridge being considered. The would reduce potential of exceeding nominally required, for no-analysis, lateral restraint, with seismic loads.

The conclusions presented above are subject to the following limitations:

1. This evaluation addressed the category of multiple span with multiple beam/girder bridges with reinforced concrete multiple column substructure bents with spans from 60 to 120 ft and column heights of 20 to 30 ft and focused on the vulnerability associated with the substructure bents.

2. The conclusions are based on adequate longitudinal reinforcing embedment into the foundation footing or pile cap, and cross-bent and adequate longitudinal reinforcing splice length such that pull-out does not occur at the hinge locations.
3. The reported seismic vulnerability is for the substructure and superstructure, separate from any bridge vulnerability associated with site soil liquefaction, flow-spreading, or abutment slumping due to weak soils.
CHAPTER 5 DEVELOPMENT OF A RAPID SCREENING ALGORITHM FOR QUANTIFYING THE VULNERABILITY OF VERMONT BRIDGES TO SEISMIC LOADING

5.1 Introduction

This chapter describes recommendations for seismic vulnerability screening methodology and its application for Vermont bridges. The chapter presents the results for a Vermont Rapid Seismic Screening Algorithm (VeRSSA) for Vermont bridges using the recommended methodology developed in this study. This methodology was developed specifically to require only the data contained in Vermont’s National Bridge Inventory (NBI) database.

Quantifying the seismic vulnerability of existing bridges within any transportation agency portfolio is a vital aspect of managing those transportation assets. Quantification is hampered by the number of bridges involved, the degree to which any bridge has deteriorated, the knowledge and effort required for the quantification analysis itself, and the fact that the condition of the inventory is continually changing. That these challenges hamper quantification is reflected by the finding that only two of fifteen responding state transportation agencies in low-to-moderate seismic regions of the United States indicated performing such quantification in a recent survey hosted on the American Association of State Highway and Transportation Agency (AASHTO) listserv as part of this research (Tables 5.1 and 5.2). Seismic vulnerability is a realistic consideration in Vermont given that the seismic hazard potential in northwestern Vermont is the fifth highest in the continental U.S.
Table 5.1 DOT Questionnaire Responses

<table>
<thead>
<tr>
<th>Seismic Hazard Level in State</th>
<th>% of Survey Responses</th>
<th>Q2-Does your DOT rate existing bridges for seismic vulnerability?</th>
<th>Q3-What seismic vulnerability rating method(s) does your DOT follow?</th>
<th>Q4-Does your DOT have specific post-EQ inspection procedures?</th>
<th>Q4-Those procedures are:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low</td>
<td>36%</td>
<td>No</td>
<td>-</td>
<td>Yes</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>No</td>
<td>-</td>
<td>No</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>No</td>
<td>-</td>
<td>Yes</td>
<td>Not specified.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>No</td>
<td>-</td>
<td>No</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>No</td>
<td>-</td>
<td>No</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>No</td>
<td>-</td>
<td>Yes (a.) State specific procedures (latest version in 2004)</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>No</td>
<td>-</td>
<td>Yes</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>No</td>
<td>-</td>
<td>No</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>No</td>
<td>-</td>
<td>Yes (b.) State specific procedures</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>No</td>
<td>-</td>
<td>Yes</td>
<td>Not specified.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>No</td>
<td>-</td>
<td>No</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Yes (c.) FHWA 2006 Seismic Retrofit Manual</td>
<td>-</td>
<td>Yes</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>No</td>
<td>-</td>
<td>Yes</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Yes (d.) State specific developed in 1989/1990 based on FHWA Seismic Retrofit Guidelines for Bridges.</td>
<td>-</td>
<td>No</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td></td>
<td>No</td>
<td>-</td>
<td>No</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Yes (e.) FHWA 2006 Seismic Retrofit Manual</td>
<td>General operations/logistics based. Not inspection specific.</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>No</td>
<td>-</td>
<td>Yes</td>
<td>7</td>
</tr>
<tr>
<td></td>
<td></td>
<td>No</td>
<td>-</td>
<td>No</td>
<td>-</td>
</tr>
</tbody>
</table>
1. Use commercial software for hazard monitoring. Inspection procedures for bridges only. Specifics not provided.
2. Not specified. Tall, movable, and masonry bridges are priority.
3. State specific post EQ procedures are available online.
4. Districts respond first then bridge inspection crews follow up if conditions warrant.
5. State's DOT Structures Emergency Response Plan
7. There is a plan for bridges. Specifics not provided.

   a. Procedures based on a combination of FHWA and state specific guidance.
   b. For certain bridges with widening, deck or superstructure replacement. Very few retaining walls or slopes are evaluated. Procedures based on a combination of FHWA and state specific guidance.
   c. Seismic prioritization is based on a 1995 Study. Bridges designed under the AASHTO code at that time were deemed to be at low risk and not considered in the study. Vulnerability of existing retaining walls and slopes has not been studied.
   d. In 1991 bridge seismic vulnerability ratings were performed on the state highway system. 286 bridges were identified as in need of seismic retrofit. Retaining walls were not evaluated.
   e. Only when preparing plans for major rehabilitation.

<table>
<thead>
<tr>
<th>Seismic Hazard Level</th>
<th>Number of responding DOT’s which are in this seismic hazard level.</th>
<th>Percentage of the Category Responses out of the Overall Survey Responses</th>
<th>Percentage of responding DOT’s that answered yes to the question: “Does your DOT rate existing bridges for Seismic Vulnerability?”</th>
<th>Percentage of responding DOT’s that answered yes to the question: “Does your DOT have a specific procedure for post-earthquake inspection of bridge and associated walls and slopes?”</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low</td>
<td>8</td>
<td>36%</td>
<td>0%</td>
<td>38%</td>
</tr>
<tr>
<td>Low to Moderate</td>
<td>7</td>
<td>32%</td>
<td>29%</td>
<td>29%</td>
</tr>
<tr>
<td>Moderate to High</td>
<td>7</td>
<td>32%</td>
<td>43%</td>
<td>57%</td>
</tr>
<tr>
<td>Totals</td>
<td>22</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 5.2 2013 DoT Seismic Vulnerability Screening Practices Questionnaire Responses

Figure 5.1 shows the locations of Vermont’s approximately 2,800 National Bridge Inventory (NBI) bridges and culverts together with the 1,000-year return period peak ground acceleration from 2002 USGS seismic hazard mapping. The 2002 hazard mapping remains applicable for the 2014 through 2017 AASHTO LRFD specifications (AASHTO
and the 2006 FHWA Seismic Retrofitting Manual for Highway Structures: Part 1 Bridges (FHWA 2006) analysis. The figure illustrates that the highest seismic hazard occurs in the northwestern portion of Vermont with a peak ground acceleration of bedrock estimated at 0.15 g, and upwards of 0.06 g elsewhere in Vermont. Note that this figure illustrates the seismic hazard for exposed bedrock sites only and does not consider bridge or site characteristics, or the resulting risk.
Figure 5.1 - 975 – year return period PGA overlay on ~2800 State Long Bridges in VTrans NBI
Risk is defined as the product of the hazard times the consequences of the resulting potential damage outcomes, which must also be considered in quantifying seismic vulnerability.

Our evaluation began by adapting the existing FHWA Seismic Retrofit Guidelines (FHWA, 2006) guidance on seismic vulnerability rating of bridges for Vermont, to an existing New York State Department of Transportation (NYSDoT, 2004) screening methodology. The NYSDoT screening incorporates tabulated NBI data with additional data acquired from as-built plans and site measurements, and serves as a relatively rapid statewide vulnerability rating tool. This combination of the FHWA and NYSDoT rating methods was further refined by applying findings from detailed seismic vulnerability modeling which was performed for typical Vermont multiple span bridges to develop a vulnerability rating screening tool for Vermont. The analyses also considered that earthquake shaking depends on geographic location and the site subsurface conditions, and the effects of deterioration of the bridges, particularly spalled concrete on reinforced concrete bents, which were also considered.

The study focused attention on multiple span bridges as they are considered seismically vulnerable in contrast to simple span bridges which are generally not considered to be vulnerable (Buckle, 1991). Multiple span bridges with multiple girder-supported decks represent 82% of the Vermont multiple span bridges, as illustrated in Figure 5.2. This bridge type category represents 55% of the 291,000 multiple span bridges nationwide, also as illustrated in Figure 5.2. This single category is one-third of the 473,000 total, non-culvert, U.S. highway bridge inventory tracked through NBI, and is only surpassed in quantity by single span bridges.
The interstate highway bridge building expansion in the 1950’s through the 1960’s led to standardization of bridge designs among state transportation agencies, including sharing of plans and typical details. The result of this standardization is that nearly the same
bent and cross-beam dimensions were used for multi-girder bridges generally independent of the span lengths. Bridge width differences are accommodated by additional columns for the wider bents. The resulting relatively small number of bridge bent configurations and use of multiple girder spans for 82% of Vermont’s multiple span bridges allowed the study to concentrate on the influence of ground motion variability, and the influence of deterioration, on the seismic vulnerability using two actual bridges in Vermont, representative of the inventory.

The AASHTO bridge design standards in the period between 1953 and 1977 required nominal seismic design requirements consisting of minimum lateral force requirements on members as a percentage of the tributary design load acting on the members. These were between 2 and 6 percent of the vertical loads, substantially below the currently specified minimum 15 to 25 percent of tributary vertical load lateral force restraint required in the recent AASHTO codes (AASHTO 2014, 2017). The seismic force requirements were gradually increased over time but did not exceed 6 percent of the vertical tributary loads until 1977, as shown in Table 5.3.

Table 5.3 Spectral acceleration values used for low and low-moderate seismic hazard scenarios

<table>
<thead>
<tr>
<th>Hazard Scenario</th>
<th>PGA (g)</th>
<th>0.2 Second Spectral Acceleration (g)</th>
<th>1-Second Spectral Acceleration (g)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low</td>
<td>0.01-0.06</td>
<td>0.02-0.14</td>
<td>0.01-0.04</td>
<td>1,2</td>
</tr>
<tr>
<td>Low-Moderate</td>
<td>0.06-0.15</td>
<td>0.14-0.25</td>
<td>0.04-0.06</td>
<td>1,2</td>
</tr>
</tbody>
</table>

1. Values are derived from the USGS 2002 Seismic Hazard maps as published in AASHTO Bridge Design Specifications beginning in 2007.
2. Values are for Seismic Site Class B conditions and boundary values are approximate.

The evaluated existing bridges are multiple span girder bridges which are widely used for interstate and urban highways both in Vermont and across the U.S., examples of
which are shown on Figure 5.3. The bridges have reinforced concrete column supported
bents configured in repetitive type configurations of square or round columns, often in the
range of 3 ft side width or diameter, respectively. The cross-beams supporting the girders
are typically square or rectangular, typically about 3 to 4 ft in dimension.

a) Bridge A with a two square column bent at 30-ft-tall supporting simple span multiple beams

b) Bridge B with a three round column bent with 20-ft-columns supporting continuous span multiple girders

Figure 5.3. Bridges evaluated for this study (Photos courtesy of VTrans).

The expected behavior of two actual bridges constructed between 1964 and 1967
as part of the interstate highway program, was analyzed using commercial structural anal-
ysis software, SAP2000, and ground motion ensembles selected to match current
AASHTO seismic design spectra bounding the range of low-to-moderate seismic hazard.
That range has been divided into two parts for this work. The first part covers from the
minimal to low (M-L) seismic hazard range, reflected in Peak Ground Accelerations (PGA)
between about 0.01 and 0.06g. The second step covers the low to moderate (L-M) seismic
hazard range reflected in PGA’s of about 0.06 to 0.15g. Table 5.3 summarizes the spectral
acceleration values at both seismic hazard conditions.
The analyses considered both pristine bridge conditions, matching the originally constructed concrete and steel design properties, and deteriorated conditions reflecting the concrete cover over transverse reinforcing steel being fully-spalled to the outside face of the confining bars. It is important to note that the fully-spalled condition which was analyzed assumes the reinforcement is still connected to the concrete and interacting with it.

5.1.1 The Study Approach

The evaluations for this study incorporated the following elements, beginning with identifying the existing state of practice associated with the various engineering elements comprising seismic evaluation and design of bridges, followed by specific evaluations applicable to the Vermont bridge inventory:

- Reviewed:
  - Published reports and guidance on bridge seismic vulnerability based on observed behavior in earthquakes.
  - Published reports of observed behavior from testing existing bridges/frames (e.g. NYSDoT, and SUNY Buffalo testing)
  - Published reports on bridge weaknesses identified in work on deterioration effects.
  - Publications on spalled concrete beam and column behavior.
  - Publications on Damage Index (DI) as a cumulative damage measure.
- Conducted a survey of state DOTs regarding their practices of conducting seismic vulnerability of bridges.
• Performed a preliminary screening for Vermont bridges using a variation of the NYSDoT screening.

• Analyzed representative examples of existing Vermont bridges subjected to earthquakes meeting the AASHTO LRFD criteria for Vermont.

• Synthesized the state of practice information with the preliminary screening and specific analyses to develop a screening tool applicable for Vermont bridges.

• Performed a final screening of Vermont bridges using the Vermont Rapid Seismic Screening Algorithm (VeRSSA) developed through this study.

A combination of observational, experimental, and theoretical investigations in the literature support the analysis leading to the recommended screening algorithm for evaluating system-wide seismic vulnerability for Vermont bridges.

5.1.2 Background of Bridge Seismic Vulnerability

5.1.1.1 Observational Findings

The bridge seismic vulnerability evaluation benefited from published investigations of seismic damage compiled for several earthquakes beginning with those in Japan and including significant earthquakes in the United States and Chile. Those investigation reports show trends of damage types occurring to bridges where seismic forces were either underestimated or not considered. Such potential underestimation scenarios are also possible for low to moderate seismicity regions in the U.S., which in general, have seen an increased estimated hazard.

The earliest of the post-earthquake investigation reports reviewed was by the Earthquake Engineering Research Center (EERC) at Berkeley of seismic damage and design
practices, which includes worldwide literature on seismic design of bridges particularly focused on work in Japan. It included bridges damaged by earthquakes in Japan between 1923 and 1968, and bridges in the 1964 Alaska earthquake, the Chilean 1971 earthquake and the 1971 San Fernando earthquake in California. The report development coincides with increasing research attention, and more importantly, supportive funding within the United States, for seismic risk mitigation which followed the large earthquakes in the decade preceding the 1971 San Fernando earthquake.

The EERC publication is pertinent to low to moderate seismic regions such as Vermont. AASHTO seismic design requirements before the 1970’s were low. The historical record in the appendix shows that seismic design requirements were initially left to the engineer’s discretion through and including the 1953 AASHTO standards, and subsequently increased in the 1961 AASHTO standards to a minimum horizontal resistance requirement at each member of 2 to 6 percent of the vertical forces, depending on the foundation bearing conditions. Beginning in 1977 the horizontal resistance requirements were increased to 25 percent of vertical loads, and subsequently adjusted through the current (AASHTO 2017) values of either 15 or 25 percent of vertical forces, depending on the design spectral acceleration values at a location.

This underestimation of potential seismic loading is analogous in general terms, to how the seismic demand appears to have been underestimated for those earlier Japanese and Californian earthquakes where damage was cataloged.

5.1.1.2 Theoretical and Analytical Findings

The largest body of published work on seismic damage is from theoretical and analytical work. Most of this work begins in the early 1970’s. An account of the state of the
practice of seismic design at that time, is given by the following quotation from the EERC publication “Chapter IV presents specifications for the earthquake-resistant design of bridges as currently used by many organizations. Emphasis is placed on Japanese specifications as they are judged by the authors of the EERI report to be the most comprehensive and modern of any seismic design regulations used throughout the world. In addition, Chapter IV presents a summary of seismic regulations for 21 countries of the world.”

5.1.1.3 Characteristics of Vulnerable Bridges

The FHWA 2006 seismic retrofitting manual explains the structure characteristics which create seismic vulnerability in bridges. Bridge vulnerability factors evidenced in post-earthquake inspections typically include span unseating (either transverse or lateral), toppling bearings, column hinging (confinement and longitudinal reinforcement splice failures), load concentrations where there are abrupt differences in column stiffness along bridge alignments, and deck and girder impact pounding to abutments, in addition to foundation failure due to soil liquefaction and lateral spreading.

5.2 Analyses for Vermont

5.2.1 Applicable Vermont Bridge Design Standards

5.2.1.1 VTrans Structures Design Manual

The VTrans Structures Design Manual (VTrans 5th Edition, 2010), contains requirements for design of new bridges and for maintaining and rehabilitating existing bridges, earth retaining structures, and buried structures following the AASHTO LRFD design standards. In terms of seismic design requirements, the manual indicates that it is generally not necessary to consider earthquake effects because of the low seismicity in the region. Specifically, Section 3.2 Load Factors and Combinations, of the manual states
under “Extreme Event I: Load combination including earthquake effects.” the following: “Generally, Vermont is in seismic zone 1 (LRFD 3.10.6). The designer need not consider earthquake load effects other than what is required in LRFD Section 3.10.9.2 for most projects. Some locations may have soil conditions where the designer may need to follow the requirements of seismic zone 2. For covered bridge design, refer to Section 3.8 in this manual.”

The 2017 AASHTO 7th Edition LRFD (AASHTO 2017) requirements in Section 3.10.9.2 specify that in Seismic Zone 1, as defined per Section 3.10.6, where the acceleration coefficient, $A_s$, is less than 0.05g, the horizontal design connection force in restrained directions shall not be less than 0.15 times the vertical tributary loads. Section 3.10.9.2 further states that at all other locations in Zone 1, the horizontal design connection force shall not be less than 0.25 times the vertical tributary loads. The acceleration coefficient, $A_s$, is above 0.05g in Vermont except at bedrock sites at the extreme south portions of the state, requiring the horizontal design connection forces to be at least 25% of the vertical tributary loads, in those areas. The historical record table in the appendix shows that the minimum 25% horizontal design connection force requirement was first specified in the AASHTO Standard Specification for Highway Bridges in the 12th Edition, in 1977.

5.2.1.2 FHWA 2006 Seismic Retrofitting Manual

The VTrans Structures Design Manual includes reference to the FHWA Seismic Retrofitting Manual for Highway Structures: Part 1 – Bridges, dated January 2006 (FHWA 2006). This manual outlines prioritization and corresponding seismic design requirements based on importance of the bridges within the transportation system, seismic hazard levels,
and remaining service life. That report provides recommended vulnerability analysis flow chart and threshold values for retrofit decisions, accounting for factors including remaining service life, how essential a bridge is to the transportation network, and the seismic hazard.

Note that the FHWA manual indicates that bridges with less than 15 years of remaining service life do not require seismic analysis for any retrofitting evaluation. While this manual serves as a guideline rather than a standard, our survey of state transportation agency seismic screening practices indicates it has been adopted by some agencies for seismic evaluations.

5.2.2 Vermont Seismic Hazard

The probabilistic seismic hazard prescribed by AASHTO 2017 and FHWA 2006 as estimated by the United States Geological Survey (USGS) for the contiguous United States is illustrated on maps in AASHTO 2017. These figures provide the predicted Peak Ground Acceleration (PGA), and Pseudo-Spectral Accelerations at natural periods, $T_n$, of 0.2, and 1.0 seconds for a single degree of freedom system with 5% of critical damping for a 7% in 75-year probability of exceedance (975-year return period). These values are based on the 2002 probabilistic seismic hazard mapping by the USGS, which remains in effect for the current AASHTO and FHWA recommendations.

The probabilistic seismic hazard values for other probability of exceedance values are also suggested for evaluating structures for seismic vulnerability and corresponding seismic design and retrofit requirements corresponding to FHWA 2006 and AASHTO 2017. The FHWA 2006 guidance references the 50% in 75-year probability of exceedance (108-year return period), corresponding to the Lower Level earthquake threshold criteria in the FHWA 2006 Seismic Retrofit Manual, applicable for performance based seismic
retrofit categories. These hazard values are no longer available as an online USGS seismic hazard tool. Note also that the AASHTO LRFD 2017 specifications indicate in Section 3.10.1 that higher-level earthquakes may be warranted for bridges with non-conventional construction and where higher performance requirements are warranted for special bridges.

5.2.3 Analysis Descriptions

5.2.3.1 Screening by Characteristics

The system-wide screening applied to Vermont bridges references bridge characteristics which are either directly recorded in the NBI database or can be inferred through other NBI catalog data. The vulnerability categories of span, column, and foundations are those prescribed by the FHWA 2006 manual and correspond to the types and frequencies of damage observed in most post-earthquake reconnaissance. Table 5.4 contains the vulnerability characteristic types, corresponding NBI items, and the range of values for each item. These characteristics are further explained in the following sections.
### Table 5.4 VeRRSA Vulnerability Screening Characteristics

<table>
<thead>
<tr>
<th>Item</th>
<th>Characteristic</th>
<th>NBI Item Number</th>
<th>Item Name</th>
<th>Item Description</th>
<th>NBI Item Value</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Least Vulnerable</td>
<td>Most Vulnerable</td>
</tr>
<tr>
<td>V1</td>
<td>Span vulnerability</td>
<td>43A</td>
<td>Kind of Material and/or Design</td>
<td>Is this a continuous span bridge?</td>
<td>Continuous</td>
</tr>
<tr>
<td>V2</td>
<td>Bearing type(s)</td>
<td>224</td>
<td>Type of Expansion Bearing Device</td>
<td>Are the bearings readily subject to toppling?</td>
<td>All others</td>
</tr>
<tr>
<td></td>
<td>Span Skew</td>
<td>34</td>
<td>Skew</td>
<td>Does the bridge skew create more chance of span unseating?</td>
<td>&lt;20 degrees</td>
</tr>
<tr>
<td></td>
<td>Span Type</td>
<td>43B</td>
<td>Type of Design and/or Construction</td>
<td>Does this bridge have girder and floor beam spans?</td>
<td>Not this type</td>
</tr>
<tr>
<td></td>
<td>Structural Condition Rating</td>
<td>239</td>
<td>Deficiency Status of Structure</td>
<td>Is this structure cataloged as structurally deficient?</td>
<td>Not SD</td>
</tr>
<tr>
<td>V3</td>
<td>Fracture Criticality of Structure</td>
<td>801</td>
<td>FCM Detail</td>
<td>Are fracture critical members present?</td>
<td>None present</td>
</tr>
<tr>
<td>Liquefaction</td>
<td>Foundation Stability</td>
<td>225 A-G</td>
<td>Type of Foundation at (Abutment, Pier)</td>
<td>Are foundations likely directly on rock?</td>
<td>B</td>
</tr>
<tr>
<td>Column Vulnerability</td>
<td>Column Ductility</td>
<td>N.A.</td>
<td>Seismic Retrofit Category per FHWA 2006</td>
<td>Is this Seismic Retrofit Category A or B?</td>
<td>A or B</td>
</tr>
<tr>
<td>Abutment</td>
<td>Abutment damage potential</td>
<td>N.A.</td>
<td>Seismic Retrofit Category per FHWA 2006</td>
<td>Is this above or below Seismic Retrofit Category D?</td>
<td>&lt;D</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Item</th>
<th>NBI Item Number</th>
<th>Item Name</th>
<th>Item Description</th>
<th>NBI Item Value</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Least Vulnerable</td>
<td>Most Vulnerable</td>
</tr>
<tr>
<td>V1</td>
<td></td>
<td></td>
<td>Continuous</td>
<td>Simple</td>
</tr>
<tr>
<td>V2</td>
<td></td>
<td></td>
<td>All others</td>
<td>Note 1</td>
</tr>
<tr>
<td>V3</td>
<td></td>
<td></td>
<td>None present</td>
<td>Present</td>
</tr>
<tr>
<td>Liquefaction</td>
<td></td>
<td></td>
<td>B</td>
<td>E</td>
</tr>
<tr>
<td>Column Vulnerability</td>
<td></td>
<td></td>
<td>A or B</td>
<td>C or D</td>
</tr>
<tr>
<td>Abutment</td>
<td></td>
<td></td>
<td>&lt;D</td>
<td>D</td>
</tr>
</tbody>
</table>

### 5.2.3.2 Span Vulnerability

Span damage resulting from seismic shaking ranges from deck settlement to spans unseating from bents. Settlement type damage arises from girders sliding off bearings or support pedestals, or inherently less stable bearings toppling, but remaining on the bents.
Span unseating occurs due to insufficient bearing seat dimensions, with spans sliding off the column bent support. Simple span bridges are most vulnerable by nature of that design type, and skewed alignments exacerbate the risk. Figure 5.4 illustrates types of span vulnerability features.

Figure 5.4 – Bridge seismically vulnerable feature examples (FHWA, 2012)
5.2.3.3 Column and Bent Vulnerability

Column vulnerability arises from insufficient ductility capacity in the hinges which can develop at locations of maximum moment, and from insufficient shear capacity. Maximum moments typically occur at the top and bottom of each column. Reinforced concrete columns and bent frames designed prior to adoption of seismic detailing in more recent design codes can have insufficient shear reinforcement and concrete confinement where hinges develop. This results in brittle fractures and failures of reinforced concrete at those hinges, and decidedly non-ductile behavior which can lead to abrupt collapse of the column and bent frames.

5.2.3.4 Abutment Damage Vulnerability

Abutment damage vulnerability arises from ground settlement under and in front of the bridge approaches.

5.2.3.5 Liquefaction-Induced Damage Vulnerability

Large foundation settlements and lateral movements can occur where the foundation soils loose most or all their shear strength due to liquefaction occurring because of substantial ground shaking at loose and submerged granular soil sites. Liquefaction potential evaluation requires site-specific geotechnical analyses requiring information on the soil types, density, depth to water table, and expected earthquake ground shaking. Liquefaction potential is greatest for loose sands with low silt contents. Properly evaluating soil density requires careful attention to the subsurface exploration procedures used and appropriate laboratory testing is needed to quantify soil gradation, including soil fines content. It is difficult to ascertain whether liquefaction potential was properly identified in subsurface explorations for a bridge project by only viewing the exploration...
logs. Moreover, the NBI database does not include subsurface data in the detail which can be available from boring logs so quantifying liquefaction vulnerability for screening by bridge characteristics requires using proxy subsurface features in the NBI. Fortunately, VTrans has been cataloging the type of foundation at bridges and using foundations bearing on ledge (bedrock) as a category. This study characterized the bridge sites as non-liquefaction susceptible in those cases, and used a conservative default seismic site class category of E for all situations where the bridge foundation is not specified in the NBI to be ledge (bedrock).

5.2.3.6 Earthquake Hazard

The earthquake hazard used for the screening evaluation is the FHWA 2006 criterion of the spectral acceleration at 1-second period estimated by the USGS for the bridge location. The 1-second spectral acceleration is considerably less than the spectral acceleration occurring at the shorter natural period of typical Vermont bridges, of approximately 0.3 to 0.7 seconds. Although there is reason to use the 0.2-second period spectral acceleration, or a weighted average between the 0.2 and 1 second spectral accelerations, this study followed the FHWA 2006 criterion since the hazard value is used to compute a relative rather than absolute vulnerability ranking. It was judged that using the existing criterion was appropriate for that purpose.

Note that for simplicity in setting the spectral acceleration values within the screening tool spreadsheet, the spectral acceleration values correspond to the highest 1-second period spectral acceleration value in the county in which a bridge is located. The conservatism associated with this simplification does not exceed approximately 10 percent.
5.2.4 Individual Bridge Analysis

5.2.4.1 Descriptions of Analyzed Bridges

This describes the results of a detailed evaluation of a subset of VTrans’ bridges which represent 82% of Vermont’s entire multiple span inventory and which are widespread throughout the state’s interstate highways. Most of the Vermont portion of the interstate highway system was constructed between the late 1950’s through about 1967 with remaining links completed in mid-1970’s and the early 1980’s. Approximately 90% of Vermont’s 195 multiple span interstate highway bridges are comprised of these multiple span concrete slab on steel girder structures. Most non-water crossing spans are supported on two to three reinforced concrete columns with concrete pier cap substructures.

Two representative bridges from this predominate category were analyzed for a total of eight cases of bridges from the multiple span, with multiple girder, inventory. These two bridges were each analyzed in their pristine state as constructed, and accounting for spalling type deterioration by removing the concrete cover over the transverse reinforcing steel, with four sets of earthquake motions. Those motions correspond to low and medium seismic shaking, both at firm and soft ground conditions. These bridges are shown in Figure 5.3.

5.2.4.2 Description of the Bridge Models

The bridge models were developed to evaluate the structural capacity available up to the point of collapse during earthquake shaking. These models evaluated the potential for damage to the reinforced concrete bents, and the potential for transverse (to roadway centerline) sliding of the girders from shaking exceeding the girder bearing restraint capacity.
The SAP2000, version 17.3, structural analysis software was used to model the bridges for: (1) non-linear static pushover to compute total transverse displacement ductility capacity, and (2) non-linear seismic time-history analyses to simulate effects of expected earthquake shaking. The models were non-linear finite-element structural representations of the reinforced concrete bents subjected to shaking from actual earthquake acceleration records. These acceleration time-history records are described in the following section.

The bridge frame models consisted of the following overall components:

- Bridge columns and corresponding non-linear hinges
- Bridge rigid-frame elements at the beam-column connection
- Bridge beam frame elements and corresponding non-linear hinges
- Girder tributary loads applied at the top of the cross beams

The analyses were performed in two steps. Step one consisted of analyzing the yield and ultimate moment capacity of the hinges. This provided the basis for estimating the yield rotation for columns and beams, and horizontal yield displacement for the columns, as well as the ultimate rotation capacity for columns and beams, and corresponding horizontal displacement capacity of the bent frames. This provided the non-linear models of the hinge behavior. The criteria used for estimating yield and ultimate displacement of the columns and beams were based on planar deformations occurring across the column and beam cross sections, respectively. The non-linear hinge properties were a moment-curvature representation of the hinges based on the hinge capacity available as the concrete strained during hinge rotation, up to maximum hinge capacity corresponding to the point of concrete crushing failure within the hinge zone.
The concrete compression stress-strain model followed the Mander formulation for reinforced concrete accounting for the confinement possible from reinforcing steel stirrups. In this case, the Bridge B spiral stirrups at 3-1/2 inches on-center spacing enhanced the concrete crushing strain capacity while the Bridge A square stirrups at 12 inches on-center were too widely spaced to increase the concrete crushing capacity beyond that of unconfined concrete.

**Hinge yield rotation** – The yield rotation capacity was chosen as the rotation associated with reaching yield strain on the outermost tension side reinforcing steel, and corresponded to 0.00138 for Grade 40 steel.

**Hinge ultimate rotation capacity** – The ultimate column rotation was chosen to be limited by the maximum computed concrete compressive strain before crushing based on the Mander formulation.

The column moment-curvature relationships are illustrated in Figure 5.5. They are developed using a moment-curvature modeling function within SAP2000 based on the column dimensions and reinforcing shown on the as-built plans for the bridges. Two conditions were modeled. The pristine condition represents the conditions shown on the as-built plans with the design unconfined compressive strength concrete of 3000 psi. The fully spalled condition represents concrete spalled off to the outside face of the transverse reinforcing. The reinforced concrete sections for pristine and spalled conditions for each study bridge are illustrated in Figures 5.6a and b.
Figure 5.5. Bridge model column hinge moment-curvature relationships

Table 5.5 Summary of Bridge Bent Model Lateral Ductility Characteristics

<table>
<thead>
<tr>
<th>Pushover values for</th>
<th>Bridge</th>
<th>Bridge A</th>
<th>Bridge A</th>
<th>Bridge B</th>
<th>Bridge B</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Units</td>
<td>Fully Spalled</td>
<td>Pristine</td>
<td>Fully Spalled</td>
<td>Pristine</td>
</tr>
<tr>
<td>Yield Point Deflection</td>
<td>feet</td>
<td>0.106</td>
<td>0.077</td>
<td>0.044</td>
<td>0.028</td>
</tr>
<tr>
<td>Yield Point Base Shear</td>
<td>kips</td>
<td>145</td>
<td>160</td>
<td>185</td>
<td>224</td>
</tr>
<tr>
<td>Ultimate Displacement Capacity</td>
<td>feet</td>
<td>0.48</td>
<td>0.48</td>
<td>0.44</td>
<td>0.55</td>
</tr>
<tr>
<td>Maximum Displacement Base Shear</td>
<td>kips</td>
<td>173</td>
<td>188</td>
<td>245</td>
<td>228</td>
</tr>
<tr>
<td>Displacement Ductility Capacity</td>
<td></td>
<td>4.5</td>
<td>6.2</td>
<td>9.9</td>
<td>19.6</td>
</tr>
<tr>
<td>Total Static Pushover Energy Capacity</td>
<td>ft-kips</td>
<td>57.8</td>
<td>65.3</td>
<td>83.1</td>
<td>122.2</td>
</tr>
<tr>
<td>Bridge Bent Transverse Tn (initial)</td>
<td>seconds</td>
<td>0.62</td>
<td>0.51</td>
<td>0.48</td>
<td>0.35</td>
</tr>
</tbody>
</table>
Step two consisted of placing the hinge properties into the frame models and performing the static push-over capacity and time-history analyses. The models were subjected to seismic shaking from 70 unique ground motion time histories in a non-linear direct integration of the model response for a total of 380 combinations of bridge configuration and ground motions as shown in Figure 5.7. Model input and analysis parameters including damping are shown in Table 5.6.

Figure 5.6. Column and beam section illustration with and without spalling
Table 5.6 VTrans Bridge Seismic Vulnerability Evaluation Summary of Bridge Model Input and Analysis Parameters

**Damping:**
- Proportional damping by direct specification
- Mass Proportional Coefficient = 0.634
- Stiffness Proportional Coefficient = 3.9E-03

**Time Integration Parameters:**
- Hilber-Hughes-Taylor  \( \gamma = 0.5, \beta = 0.25, \alpha = 0 \)
- Maximum Newton-Raphson Iterations per Step = 40
- Integration Convergence Tolerance = 1.0E-04

Computing the behavior of the bridges was done through an incremental analysis which solved for each node, for each static load increment in the pushover analysis, and for each acceleration increment at each time-step of the time-history, to achieve equilibrium at each node. The static pushover force was applied at the cross-beam and the time-history acceleration was applied at the base of the columns. Each increment of the analysis required multiple iterations of estimated displacements to achieve equilibrium at each of the nodes until the estimated and computed displacements converged within the specified tolerances.
The seismic analysis required solving, via direct integration, for each node, the structural displacements necessary for equilibrium according to the following equation of motion:

\[
M\ddot{x}_s(t) + C\dot{x}(t) + F(x)t = M\ddot{x}_g(t)
\]

(eq 2)

where:

\(M\) = mass of the individual elements within the structural model

\(C\) = velocity-based damping coefficient applied to the model elements

\(F\) = the displacement position based restoring force on the member

\(\ddot{x}_s(t)\) = acceleration of individual elements within the model, i.e. inertia, (per time step)

\(\ddot{x}_g(t)\) = the applied earthquake acceleration at the base of the model (per time step)

\(\dot{x}(t)\) = the velocity of the individual elements in the model (per time step)

\(x(t)\) = the displacement of individual elements in the model (per time step)

The seismic analysis advanced sequentially in time steps not exceeding that of the earthquake ground motion records, which were typically 0.0024 to 0.01 seconds each, with the records typically lasting from 30 to 100 seconds.

Corresponding base shear and member displacements are available at the end of each incremental analysis step, and these are used to evaluate:

- Structure lateral displacement
- Hinge rotations and corresponding moments in the hinges
- Horizontal shear forces at the column bases and at girder bearing level
5.2.4.3 Description of the Ground Motion Time Histories

Ground motion time histories were obtained from the Pacific Earthquake Engineering Research (PEER) center NGA West 2 database. The time histories were selected to match the design spectra for two bounding seismic hazard conditions in Vermont, per the AASHTO 2014 LRFD, which correspond to a 7% in 75-year probability (1033-year return period) of exceedance for the extreme northwest, and southeast of Vermont. These target design spectra are shown on Figures 5.8a-d along with the spectral accelerations for each of the time-history records within the ensembles chosen to match those spectra.
Figure 5.8. a–d Seismic Hazard Level 1 through 4 target spectra with ensemble recorded ground motions
The ground motion time history filtering of the PEER database was made such that the motions match as closely as possible the conditions which could occur with motions acting on Vermont bridges. Specifically, the motions met the following criteria:

- All motions were unscaled from the original recorded motions.
- Motions were selected from source locations which met either Seismic Site Class B (firm ground) or E (soft ground) conditions, based on the site class conditions reflected in the average shear wave velocity values, $V_{S_{30}}$, in the top 30 meters at the source sites.
- Motions were from earthquakes of Magnitude 5 to 8, and were not pulse motions, with a minimum distance to faulting of 5 kilometers, and usually greater than 20 kilometers.

These constraints were used to obtain ensembles of motions which were as close as feasible in bracketing the range of typical bridge site conditions, namely Seismic Site Class B and E, for the seismic hazard conditions in Vermont.

The PEER NGA East ground motion database became available during the latter portion of this work and was searched for ground motions meeting the target spectra. Ideally the time history records from the eastern North America tectonic region could be used for the analyses. Unfortunately, the available motions do not match the target spectra without scaling. Figure 5.9 illustrates locations of the ground motions available in the PEER NGA East ground motion catalog.
Figure 5.9 -PEER NGA East ground motion record locations (PEER, 2018)

The summary of the ground motion time-history record characteristics used for these analyses is provided in the appendix.

5.3 Analysis Results

5.3.1 Individual Bridge Analyses

The following describes results of individual seismic vulnerability analyses made on Bridge A and Bridge B.

5.3.1.1 Static Pushover Analysis

The results of non-linear static pushover analysis performed on each of the bridge bent models to estimate the yield values and ultimate displacement capacities, and corresponding displacement ductility for each bent in both pristine and spalled conditions are shown on Figure 5.10 and in Table 5.5. Highlights of the results are:
Bent frame displacement ductility is greater than 4.5 with the square columns and stirrups at 12-inches on-center, and more than twice that with the round columns and spiral stirrups at 3-1/2 inches on-center.

The bent natural period increases with the loss of the concrete cover, in both cases, and is significant, at 0.1 seconds increase for both bridge models.

The yield displacements increase with concrete cover removed (spalled), with yield occurring at lower base shear forces.

5.3.1.2 Damage Index

The analysis results in histogram form for the ground motions applied to the two bridges applied in the previously described combinations of seismic hazard, seismic site class, and aging condition are shown on Figure 5.11. The figure illustrates that most damage index values are under 0.3, corresponding to negligible or minor damage.
These results indicate low potential for seismic damage to the concrete bent frames for these types of bridges in low-to-moderate seismic hazard regions. Figure 5.12 illustrates the probability of exceeding various damage potential thresholds as categorized by Damage Index for both minimal to low seismic (southeast Vermont), and moderate (northwest Vermont) seismic hazard conditions.
5.3.1.3 Potential to Exceed Horizontal Shear Force Capacity

Seismically imposed shear forces on the bridge models for the two seismic hazard categories considered in this work ranged to nearly 300 kips. This compares with yield capacities ranging between 145 and 225 kips, and ultimate base shear capacities ranging between 175 and 275 kips, depending on the bridge and deterioration level. Figure 5.13 illustrates the probabilities of exceeding yield and base shear capacities depending on the seismic hazard and bridge type and condition.

![Figure 5.13. Maximum displacement vs. maximum base shear during applied ground motions and pushover for bridge A and bridge B, pristine and fully spalled](image)

5.3.1.4 Potential for Bent Cap Displacement (Drift)

Potential for reaching various bent cap displacements is also shown on Figure 5.13. There is negligible computed probability of exceeding more than one-half of the ultimate displacement capacities for the bridges considered in this work except for the case of the low-to-moderate seismic hazard conditions with soil seismic site class conditions softer
than site class B (soft rock). This corresponds with computed displacements reaching collapse values in four of the one-hundred cases evaluated for low-to-moderate seismic hazard with site class E conditions. In those cases, the stiffer bridge (with $T_n=0.35$ seconds) encountered the large displacements.

### 5.3.2 VeRSSA Screening by Vulnerability Characteristics

Results of screening of multiple span bridges by vulnerability characteristics using the VeRRSA are shown on Figure 5.14. The rating range is a relative ranking for this group of bridges and corresponding seismic hazard range. The numerical score indicates relative vulnerability with the lowest scores corresponding with the lowest relative vulnerability.

![Histogram of vulnerability rating values for multiples span bridges from VeRRSA analysis](image)

**Figure 5.14.** - Histogram of vulnerability rating values for multiples span bridges from VeRRSA analysis

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Generalizations regarding characteristics suggested by this rating are:

- The highest proportions of continuous bridges are in the lowest binned vulnerability rating categories. This is at least partly an artifact of the screening algorithm which favors continuous bridges because of their generally lower susceptibility to span dropping. Moderate and higher vulnerability bridges are mostly simply supported span bridges.

- Multiple girder bridges comprise nearly 90% of the lowest rated bridges, and between 70 and 80 percent of the highest vulnerability rated bridges.

- Bridge plan availability reported in the NBI tabulation ranges from about 52% to over 90% with generally more than 80% availability for each vulnerability category. This is promising for adding characteristics into the bridge database for further screening ability.

5.4 Seismic Vulnerability Rating Conclusions and Recommendations

5.4.1 Bridge Seismic Vulnerability Sources

Experience and analyses regarding seismic vulnerability of bridges described in published literature indicate that the vulnerability results from the presence of one or more bridge and site subsurface characteristics, coupled with seismic hazard, enumerated as follows:

- Where there is insufficient ductility capacity in the substructure, principally where reinforced concrete is used, but not limited to concrete. The problem
occurs wherever the substructure displaces so much that it no longer has capacity to support the superstructure, and so masonry, steel, and timber substructures are also susceptible.

- Where there is fragility in the superstructure to substructure connections, such as hinges which topple and bearing to bent frame connections which break.
- Where the superstructure bearing dimensions are insufficient such that main support members fall off their supports. The drop can be several inches off a bearing pedestal, or the entire column height, depending on how much displacement occurs.
- Where subsidence susceptible soils underlie the substructures and approaches such that liquefaction or flow slides can cause settlement and lateral displacement unless these are prevented with proper structure foundations or ground improvement.
- Hazards from these characteristics are compounded where earthquake related scour can also occur, such as due to the catastrophic failure of an upstream dam. This is an uncommon combination of hazards, but it needs to be considered because of the potential extreme consequences.
- Multiple span bridges are considered seismically vulnerable while single span bridges generally are not, based on post-earthquake damage observations.
- Bridge seismic vulnerability also depends on the seismic hazard at the bridge location. The seismic hazard in Vermont is greatest in the northwest and decreases to the south. The expected bedrock ground motion at the northwest
portion of Vermont of 0.15 g is approximately 2.5 times more than along the Massachusetts border.

- Earthquake motions originate in bedrock and can be amplified at the ground surface through overlying soils. The amplification increases with thicker and softer soils overlying the bedrock, as recognized by evaluations made of the ground motions recorded during the 1989 and 1994 California earthquakes, and translated into seismic amplification factors recommended in AASHTO seismic design requirements. Those amplification factors range to 3.5 times the bedrock acceleration. This translates to AASHTO LRFD specified design earthquake ground accelerations ranging between 0.06 g for bearing on bedrock in southern Vermont, and 0.67 g in northern Vermont for bearing on thick, soft soils.

- The Vermont inventory has bridges with each of the vulnerability characteristics described above. Multiple span bridges comprise 22 percent of the highway bridges in the NBI database. Eighty-two percent of the multiple span bridges are multiple girder bridges comprised of steel or concrete girders with concrete decks, with the remaining 18 percent comprised of over 10 other bridge types in proportions illustrated in Figure 5.2. Bridges are widely distributed across Vermont (see Figure 5.1) such that the seismic hazard variation affects the inventory on essentially a state-wide basis.
5.4.2 Comments on the Recommended Seismic Vulnerability Screening Procedures for Vermont Bridges

The FHWA 2006 seismic retrofit manual screening recommendations reflect over 20 years of development and refinement for highway bridges typical of the U.S. inventory. This development record along with the findings from this modeling of the Vermont multiple span multi-girder bridges is the background supporting the recommendation to use the FHWA 2006 retrofit screening criteria as an underlying basis for a system-wide rapid-screening-algorithm using the Vermont NBI database.

The recommended approach for quantifying the seismic vulnerability of Vermont bridges is to: 1) utilize the Vermont NBI database information for a system-wide rating followed by, 2) specific individual analyses of bridges with higher vulnerability ratings. Note that the system-wide ratings consider the criticality of bridge damage to the transportation system, considering average daily traffic, bypass detour length, and whether the bridge is on a National Defense Highway or the Designated National Network for Trucks.

The Vermont Rapid-Seismic-Screening-Algorithm (VeRSSA) uses the NBI database information, as supplemented with some of Vermont’s additional recordings (Category Items above 116 through 823) to rank the bridge seismic vulnerability based on bridge and site characteristics which the FHWA 2006 manual identifies as indicative of vulnerability.

The FHWA 2006 screening protocols also consider factors not currently recorded in the NBI database. These include detailed information on the subsurface conditions and foundation support, bearing seat dimensions for the superstructure, and the column ductil-
ity resulting from the amount of confining steel present. These characteristics are key factors in seismic vulnerability and in their absence from the NBI database, the VeRSSA is based on conservative assumptions for those factors.

Cataloging those characteristics not in the NBI database for all multiple span bridges requires retrieving plans, where available, to get bearing seat dimensions, concrete reinforcing sizes, lengths, spacing, and steel grades, and foundation bearing information including foundation types and their dimensions, and the subsurface conditions which may be shown on the plans. The foundation and subsurface condition evaluation requires evaluation by geotechnical engineers, particularly for bridges constructed before the 1960’s, that is, prior to using the Standard Penetration Test (SPT) for subsurface explorations. The older explorations usually rely on samples retrieved inside a driven pipe and have both limited soil data and descriptions which can be difficult or impossible to interpret in terms of seismic vulnerability. In other instances, the bridges may not have recorded exploration data. Consequently, engineering judgment needs to be applied for those situations unless modern subsurface explorations can be performed.

The suitability of deep foundations to mitigate seismic hazard needs to be evaluated, especially for older bridges constructed before modern subsurface explorations and attention to seismic hazards in design and construction. Such foundations, typically timber or steel piles in older bridges, need to be evaluated in terms of where they obtain bearing, such that they are confirmed to bear below liquefiable zones. They also need to be evaluated for sufficient reserve capacity in the event of liquefaction developing. Also, in the absence of as-built plans, judgment needs to be applied in relying that the foundations have been installed according to the drawings.
Approach fill settlement vulnerability also needs to be considered, and is described as abutment vulnerability in the FHWA 2006 manual. Seismic shaking can cause loose soils to settle or liquefaction of susceptible soils, and slope instability in front of stub-type abutments. The presence and reliability of subsurface explorations at the approaches is important in the same manner as for the foundations.

Cataloging this additional information should be prioritized within the goals of the bridge inspection and asset management efforts. This will substantially improve the granularity of bridge data available for the seismic vulnerability screening, and improve the reliability of the data used in the screening. In the meantime, the current VeRSSA is intended to provide a conservative estimate of seismic vulnerability, although as with any practical screening method, this cannot be considered absolute. Even a conservative screening approach involves uncertainty and risk from underestimating vulnerability.

The chosen vulnerability factors and weightings were judged to be moderately conservative and are based on validation checks on samples from each of the resulting ratings groupings. The findings suggest these groupings are conservative with the caveat that the bridges in each vulnerability rating category should also be individually considered by VTrans engineers who are familiar with them. There is no substitute for engineering judgment to check that the screening is providing reliable results.
CHAPTER 6 CONCLUSIONS AND RECOMMENDATIONS

Low to moderate seismicity regions exist in each of the continental United States, with over 30 states having mostly low-to-moderate seismicity. Resources at state and municipal transportation agencies are therefore understandably focused on higher seismicity regions, creating an absence of quantified system-wide seismic vulnerability. This absence creates uncertainty, which is compounded by an overall aging bridge inventory, much of which was built before current seismic design standards. This research addressed this data gap and attempted to reduce barriers to quantifying seismic vulnerability of existing bridges in low-to-moderate seismicity regions. The work included nonlinear dynamic numerical modeling of typical multiple span bridge configurations in both pristine and deteriorated conditions, by subjecting them to several levels of seismic ground motions, to evaluate their seismic performance. These typical bridge configurations represent over 160,000 bridges, which comprise 55% of the multiple span bridges nationwide.

6.1 Work Performed

The work performed for this research consisted of the following:

1) A literature review was performed on practice and research of bridge seismic vulnerability, retrofitting, and hazard quantification both within the United States and internationally. This included review of literature on:

   a) bridge component strength reduction, including deterioration based on corrosion, age, and exposure to freeze-thaw conditions;

   b) non-linear analysis methodologies used to model bridges under seismic loading;
c) seismic hazard prediction and ground motion characteristic comparisons between interplate and intraplate sources; and
d) damage quantification measures for numerical analyses of seismic loading of bridges.

(2) Bridge characteristics and plans were compiled for the Vermont portion of the National Bridge Inventory (NBI), comprised of all bridges with 20-ft or greater spans. This consisted of:

a) cataloging characteristics related to seismic vulnerability including bridge types and features, along with span lengths, column heights, and age from the NBI database and supplemental data maintained by Vermont as part of that database; and

b) searching the Vermont bridge archives for plans of representative bridges, acquiring the plans, and extracting detailed information on the substructures, spans, abutments, and subsurface conditions.

(3) There was an investigation of the NBI database to compile bridge type, age, and feature statistics for the national inventory focused on regions of low-to-moderate seismicity.

(4) Bridge inspection visits were performed with a VTrans bridge inspection crew and visits were made to bridge repair and reconstruction projects in Vermont to understand the annual inspection practice, observe conditions of existing bridges, and observe typical repair and rehabilitation methods.
(5) There was a survey distributed through AASHTO to state transportation agencies seeking information on the bridge seismic vulnerability screening practices in each agency, with compilation of those survey responses.

(6) There was an investigation of the Vermont and national NBI database of bridge span and column height statistics to quantify the predominant span and column ranges for multiple span with multiple beam/girder bridges, followed by a direct-stiffness method analysis of the natural periods of transverse vibration for typical bridge bents with span lengths and column height to characterize the range of spectral acceleration periods applicable for this type bridge.

(7) The Pacific Earthquake Engineering Research institute (PEER) database was analyzed to select 70 ground motion records for use in non-linear time-history analysis.

(8) There were four representative two-dimensional bridge models developed using the SAP2000 finite element structural analysis software to represent the reinforced concrete bridge bent and steel girder with concrete slab bridges using dimensions from the record plans. These models were analyzed as follows:

a) Moment-curvature relationships were developed for column and beam hinges in each bent based on the reinforced concrete design information in the record plans.

b) There were pushover analyses performed for each of the two-dimensional bridge models to compute the yield and ultimate displacement capacities. This included force-displacement and moment-hinge rotation plastic energy dissipation computations to quantify the non-linear energy dissipation capacity of each bridge model.
c) Each model was analyzed with 70 ground motion time history records in a non-linear time-step approach.

d) Each of the 70 non-linear time-history analysis results from the SAP models was processed in a MATLAB code to compute the hysteretic energy dissipated in the non-linear portions of displacement.

e) Each of the non-linear time history analysis outputs was extracted to obtain the displacement and base shear time history records, with maximum values of displacement, plus the computed dissipated hinge rotation energy compiled to compute a damage index.

(9) The non-linear time history results were compiled to evaluate the performance of each bridge under each of four ensembles of ground motion records, representing four levels of seismic hazard.

6.2 Overall Conclusions

The overall conclusions drawn from this work are as follows:

- The results suggest that the seismic vulnerability of the predominant bridge type in the national inventory, consisting of multiple span with multiple beam/girders supported on reinforced concrete bents, is overall low. Out of the 280 applied ground motions, less than 20 motions cases indicated slight to moderate cracking type damage in the reinforced concrete columns with Damage Index (Park and Ang, 1985a,b) under 0.4.

- The age-related deterioration of the reinforced concrete bents, leading to loss of concrete cover over reinforcing steel, because of spalling, creates a relatively minor increase in seismic vulnerability, which is less than the increase in seismic vulnerability
which can result from a higher seismic hazard level condition within low-to-moderate seismic hazard regions.

- Seismic vulnerability of this predominant bridge type is associated with the following factors:
  
  - They are typically configured as strong beam-weak column such that they are vulnerable due to column hinge yielding failure, particularly bridges designed before improved seismic detailing.
  
  - Transverse base shear forces can reach levels which could shear bearings that are weakened due to corrosion, or lead to toppling where tall bearings are used.
  
  - Transverse displacements expected from the evaluated earthquake loads in most instances are relatively low (i.e., under two-inches) such that girder unseating is not a high potential, provided there is enough restraint against sliding. This will usually consist of at least of friction between bearing plates and the concrete pier caps, and girder flanges, and the stiffness of the deck/concrete slab system, especially with continuous span bridges, plus anchor bolts.
  
  - The likelihood of bridge damage below the threshold of column cracking is with spectral accelerations of ground motion at the bridge natural period below 0.35g. Since design spectral accelerations used in the latest AASHTO design standards (AASHTO 2017) correspond to mean values of spectral accelerations, this requires design spectral values low enough to accommodate variability above the mean.
  
  - The likelihood of bridge damage remains relatively low where the spectral acceleration of a ground motion at the bridge natural period is moderately above 0.35g, due to the
ductility in the bridge components absorbing seismic energy. However, this ductility results in cracking and spalling damage at the column hinge locations.

- Seismic vulnerability screening to quantify on a system-wide level the relative seismic vulnerability risk to a state-wide bridge inventory is achievable, as experienced with the analysis of the Vermont bridge inventory during this work.

6.3 Intellectual Merit/Contributions and Broader Impacts of the Research

This research has made the following contributions to the state-of-the-art:

- This research has investigated the vulnerability of existing bridges not specifically designed for seismic loading meeting current design requirements, and which have deteriorated due to aging with complete spalling of concrete cover over reinforcing, as is observed to occur in many bridges.

- This study investigated the expected seismic demands imposed on typical bridges using unscaled recorded ground motions, adding to knowledge on the influence of motion types on bridge response, particularly applicable to low-moderate seismicity regions which have so far had less study attention.

- This research specifically addressed the vulnerability of reinforced concrete multiple column bents which are commonly used for most multiple span bridges in low-to-moderate seismic hazard regions. The research results indicating generally low vulnerability of this substructure type allow generalizing, with caution, the seismic vulnerability of substructures for bridges with these substructures, without an individual analysis for each bridge. That result supports the reliance on a rapid seismic vulnerability screening on a system-wide basis. Rapid seismic screening
of bridges based on feature characteristics allows for system-wide analysis to be performed, and regularly updated, with modest demand on transportation agency resources. This removes a significant barrier to obtaining reliable seismic vulnerability metrics for asset management and emergency response.

- This research provided system-wide bridge seismic vulnerability quantification to the Vermont Agency of Transportation and showed its applicability to the broader low-to-moderate seismic hazard regions comprising most of the continental United States. This could enable other transportation agencies in similar seismic hazard regions to quantify seismic vulnerability of their bridge inventory and sustain that quantification on a timely basis in the future.

- A broader impact of this research is a template for application of the methodologies followed for bridges to other categories of infrastructure where seismic vulnerability on a system-wide basis is lacking. This includes dams and levees, energy transmission, and utility systems, all of which have existing infrastructure, which is widespread, aging, and usually not designed to current seismic hazard standards.

6.4 Recommendations for Future Work

Future potential work related to this research includes performing additional non-linear time history analyses using ground motion records which become available from earthquakes in intraplate regions, and which meet the low-to-moderate hazard levels considered in this study. There are understandably few motions from such regions due to the overall low seismicity, and corresponding lack of records. As time passes and such
records accumulate, adding bridge model responses from those records to those accumulated in this study will enhance the reliability of the seismic vulnerability estimates.

A second recommendation for future work includes developing a screening tool for scrutinizing new bridge designs which are performed for bridges which are in no-seismic-analysis regions. Seismic vulnerability characteristics used for screening should be checked in new bridge designs, to avoid incorporating vulnerable characteristics. This screening tool, or in other words, expert a review, is an important consideration for the bridge engineering profession because most of the progress in seismic engineering design has largely occurred over the past 40 years. Current researchers and senior practitioners have been involved in, or observed, much of this progress, and learned improved methods “along the way”. This translates to a great deal of institutional knowledge residing in researchers and practitioners that will eventually retire from practice. Seismic engineering and design are especially complex and evolving, such that newcomers to the field have a large body of knowledge to acquire. An expert “system” to check for inadvertent vulnerabilities in designs is worth considering.
CHAPTER 7  COMPREHENSIVE BIBLIOGRAPHY

Bozorgnia, Yousef, and Vitelmo V. Bertero. n.d. Improved Damage Parameters for Post-Earthquake Applications.


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NYSDoT, 2004, Seismic Vulnerability Manual, New York State Department of Transportation, Bridge Safety Assurance Unit, November 2004


## APPENDIX

Ground Motion Summary Information (PEER, 2018)

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<td>3.2.1.(design loads), 3.4.1.(unit stresses)</td>
<td>In both sections, earthquakes are mentioned but no quantifiable details are provided.</td>
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<td>1.2.20.</td>
<td>EQ = (C)(D) provides lateral force at cg of structure; where C = 0.02/0.04/0.06 depending on supporting soil (i.e., spread footing bearing pressure or if piles are used), D = dead load (Live load may be neglected)</td>
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