VERMONT

AGENCY OF TRANSPORTATION

QUANTIFYING THE VULNERABILITY OF VERMONT BRIDGES TO SEISMIC LOADING

Dr. John E. Lens, Graduate Researcher

Dr. Mandar M. Dewoolkar, Principal Investigator

Dr. Eric M. Hernandez, Co-Principal Investigator

The University of Vermont

College of Engineering and Mathematical Sciences

June 2019

Research Project

Reporting on Project 737-12

Final Report 2019-12

The University of Vermont

This material is based upon work supported by the Federal Highway Administration under SPR-B RSCH016. Any opinions, findings and conclusions or recommendations expressed in this publication are those of the author(s) and do not necessarily reflect the views of the Federal Highway Administration.

TECHNICAL DOCUMENTATION PAGE

<table>
<tr><td>-1. Report No.
2019-12</td><td>-2. Government Accession No.</td><td>-3. Recipient's Catalog No.</td></tr>
<tr><td colspan="2" rowspan="2">4. Title and Subtitle
Quantifying the Vulnerability of Vermont Bridges to Seismic Loading</td><td>-5. Report Date
-June 7, 2019</td></tr>
<tr><td>-6. Performing Organization Code</td></tr>
<tr><td colspan="2">-7. Author(s)
Lens, John (https://orcid.org/0000-0001-6141-3157)
Dewoolkar, Mandar (https://orcid.org/0000-0002-8726-0803)
Hernandez, Eric</td><td>-8. Performing Organization Report No.
-</td></tr>
<tr><td colspan="2" rowspan="2">-9. Performing Organization Name and Address
The University of Vermont
College of Engineering and Mathematical Sciences
Department of Civil and Environmental Engineering
213 Votey Hall
33 Colchester Ave.
Burlington, VT 05405</td><td>-10. Work Unit No.
-737</td></tr>
<tr><td>-11. Contract or Grant No.
CA0228</td></tr>
<tr><td colspan="2" rowspan="2">-12. Sponsoring Agency Name and Address
Vermont Agency of Transportation (SPR)
Research Section
One National Life Drive
Montpelier, VT 05633</td><td>-13. Type of Report and Period Covered
Final Report 2013-2019</td></tr>
<tr><td>-14. Sponsoring Agency Code</td></tr>
<tr><td colspan="3">-15. Supplementary Notes
-Conducted in cooperation with the U.S. Department of Transportation, Federal Highway Administration.
https://vtrans.vermont.gov/sites/aot/files/planning/documents/research/publishedreports/2019-12_Seismic.pdf</td></tr>
</table>

-16. Abstract

This report describes recommendations for seismic vulnerability screening for Vermont bridges. These recommendations are based on findings from a study by the authors from the University of Vermont Department of Civil and Environmental Engineering. The study involved evaluating seismic vulnerability evaluation methodology used by state transportation agencies, and those recommended by the Federal Highway Administration. The project work included detailed seismic analysis of two multiple span and multiple girder bridges for comparison with screening methodology findings, to validate the screening tool algorithms. This bridge type and dimensions represent and bound approximately eighty percent of the Vermont multiple span bridge inventory, including most interstate highway bridges. This validation analysis simulated two bridge condition configurations, for both pristine and fully-spalled concrete substructures, for the highest and lowest seismic hazard regions in Vermont, at bedrock and soft ground sites, for a total of eight combinations of structure, structure condition, seismic hazard, and site conditions. The analysis used 70 actual unscaled earthquake time-history records applied to non-linear finite element bridge models for a total of 380 individual analyses.

The report provides 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. The report includes recommendations for using supplemental site and bridge data beyond the database information to refine the screening. The recommended methodology addresses highway bridges, although the principles are applicable to railroad bridges, with appropriate engineering judgment.

-17. Key Words	**-18. Distribution Statement**
Seismic vulnerability screening, Bridge vulnerability, System-wide seismic screening, Deteriorated bridges, Low-to-moderate seismic hazard	No restrictions. This document is available through the National Technical Information Service, Springfield, VA 22161.

-19. Security Classif. (of this report)	**-20. Security Classif. (of this page)**	**-21. No. of Pages**	**-22. Price**
Unclassified	Unclassified	51	

EXECUTIVE SUMMARY

This report summarizes work done to evaluate the seismic vulnerability of Vermont bridges and develop an analytical tool for VTrans to continue assessing seismic vulnerability of bridges in the future. The practical constraints on making individual seismic vulnerability evaluations on an ongoing basis for the nearly three-thousand bridge inventory requires an approach which rapidly identifies which are the more vulnerable bridges, considering their criticality in the transportation network, that warrant closer, and where needed, individual analysis. The earthquake engineering community has expended considerable effort and made progress in methodologies for assessing seismic vulnerability of bridges at the system-wide level to identify more vulnerable structures for prioritized attention. The Federal Highway Administration (FHWA) has developed seismic vulnerability rating systems for bridge inventories, with the most recent version being the Seismic Retrofitting Manual for Highway Structures: Part 1 Bridges, published in January 2006. This manual provides a screening methodology based on bridge vulnerability characteristics identified in post-earthquake inspections and vulnerability research. Some characteristics are recorded in the National Bridge Inventory (NBI) database while others need information from site visits or bridge plans. The New York State Department of Transportation also developed a seismic vulnerability screening methodology in 1995, updated through 2004, based on earlier versions of the FHWA manual, which allows for a system-wide screening of bridges using data from the NBI database, also supplemented with additional bridge-specific information.

The goal of this project for VTrans was to develop a system-wide seismic vulnerability rating method which required only the information in the Vermont NBI database, together with already present supplemental information on certain bridge features, and which also accounted for bridge condition. A rating tool based on readily

available features is needed for low-to-moderate seismic hazard regions, such as Vermont, since while seismic risk is a necessary and appropriate concern, there are limited agency resources available to address the numerous hazards to bridges, in addition to seismicity. The Vermont Rapid-Seismic-Screening-Algorithm (VeRSSA) developed through this study is an approximate quantification of seismic vulnerability for bridges. It provides a quantitative seismic vulnerability rating for the Vermont bridge inventory and can be refined through additional evaluation focused on relatively more vulnerable bridges. It is also important to recognize that the VeRSSA can be applied to each year's installment of the NBI database to maintain an ongoing record of seismic vulnerability in the bridge inventory. Furthermore, the gap between this system-wide screening level vulnerability rating and individual bridge ratings could be narrowed for some bridges by obtaining measures of certain additional bridge characteristics for the NBI database, which are described in this report.

ACKNOWLEDGMENTS

This work was funded by the Vermont Agency of Transportation (VTrans). Administrative support provided by the University of Vermont Transportation Research Center (UVM TRC) is appreciated. The authors thank the following UVM students who contributed to this report: Tyler Kuehl, Ian Anderson, and Connor Butwin. We also thank the VTrans personnel who supported the work of this project including Emily Parkany, Chris Benda, Pam Thurber, Wayne Symonds, Jason Cloutier, Justin White, Joshua Martineau, and George Colgrove.

.

TABLE OF CONTENTS

LIST OF TABLES

Table Page

LIST OF FIGURES

1. Introduction

1.1 Introduction

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 1.1 and 1.2). 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 1.1 DOT Questionnaire Responses					
Seismic Hazard Level in State	% of Survey Responses	Q2-Does your DOT rate existing bridges for seismic vulnerability?	Q3-What seismic vulnerability rating method(s) does your DOT follow?	Q4-Does your DOT have specific post-EQ inspection procedures?	Q4-Those procedures are:
Low	36%	No	-	Yes	1
		No	-	No	-
		No	-	Yes	Not specified.
		No	-	No	-
		No	-	No	-
		No	-	No	-
		No	-	No	-
		No	-	Yes	2
L-M	32%	No	-	No	-
		No	-	No	-
		No	-	No	-
		Yes (a.)	State specific procedures (latest version in 2004)	Yes	3
		Yes (b.)	State specific procedures	No	-
		No	-	Yes	Not specified.
		No	-	No	-
M-H	32%	Yes (c.)	FHWA 2006 Seismic Retrofit Manual	Yes	4
		No	-	Yes	5
		Yes (d.)	State specific developed in 1989/1990 based on FHWA Seismic Retrofit Guidelines for Bridges.	No	6
		No	-	No	-
		Yes (e.)	FHWA 2006 Seismic Retrofit Manual	General operations/logistics based. Not inspection specific	-
		No	-	Yes	7
		No	-	No	-

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 DOT's Structures Emergency Response Plan
6 No specific procedures for earthquakes. State has an Emergency Response Plan for catastrophic events response.
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 1.2 2013 DoT Seismic Vulnerability Screening Practices Questionnaire Responses

Seismic Hazard Level	Number of responding DOT's which are in this seismic hazard level.	Percentage of the Category Responses out of the Overall Survey Responses	Percentage of responding DOT's that answered yes to the question: "Does your DOT rate existing bridges for Seismic Vulnerability?	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?
Low	8	36%	0%	38%
Low to Moderate	7	32%	29%	29%
Moderate to High	7	32%	43%	57%
Totals	22			

Figure 1.1 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 014, 2017) 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.

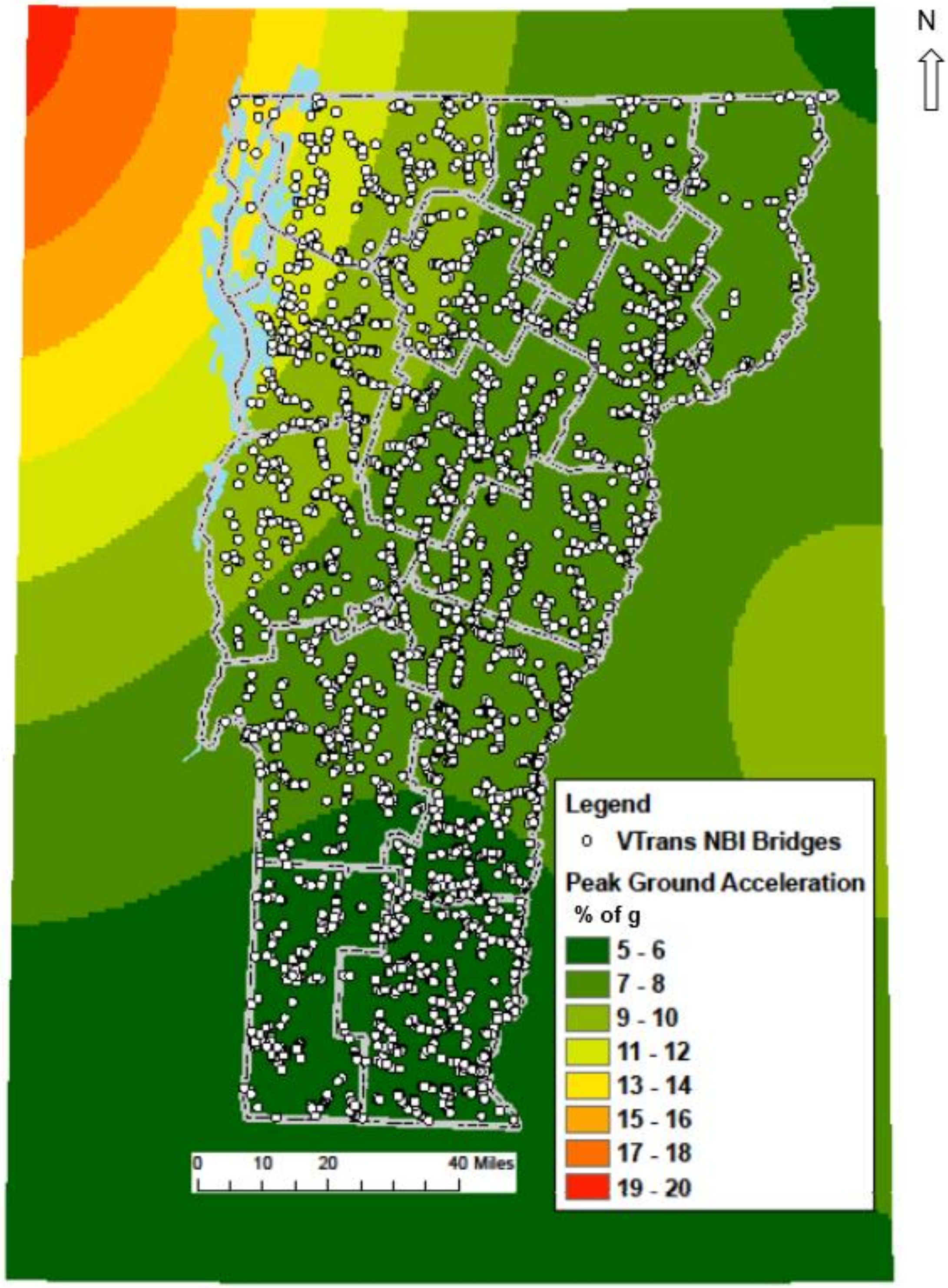


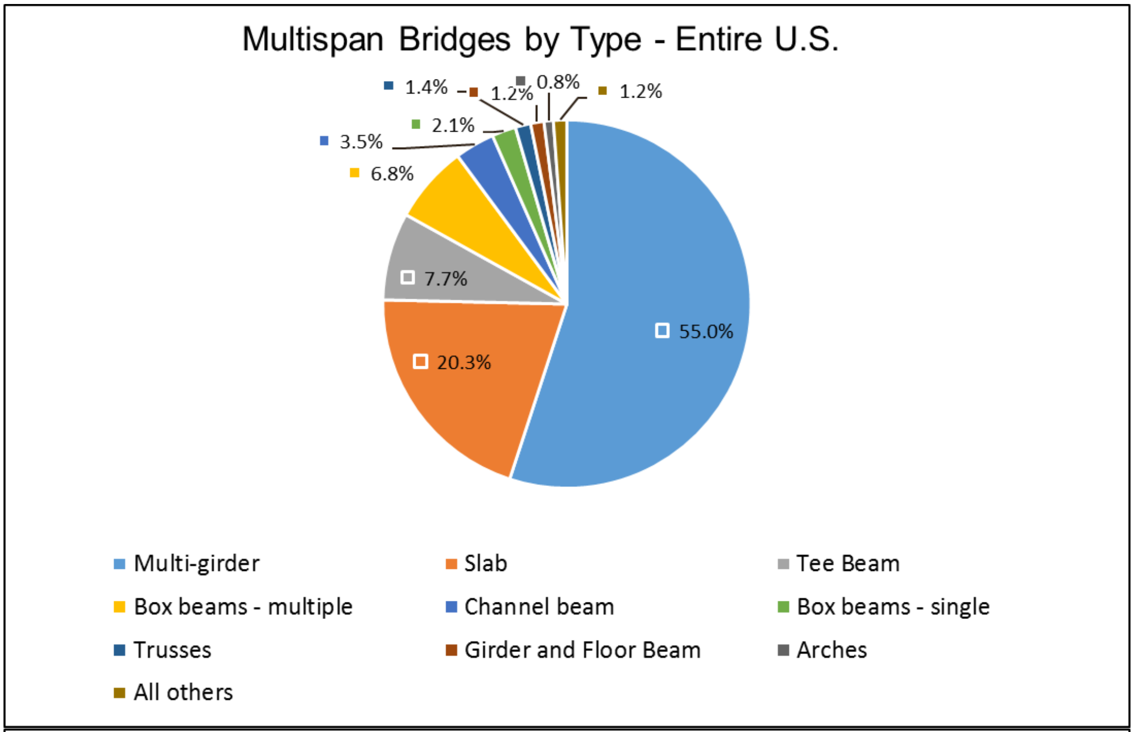
Figure 1.1. 975 – year return period PGA overlay on ~2800 State Long Bridges in VTrans NBI

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.

The project's 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 system-wide 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 modeling considered that earthquake shaking depends on both geographic location and the site subsurface conditions, and the effects of deterioration of the bridges. The latter deterioration was accounted for with reinforced concrete bents in both pristine condition, and with the concrete covering transverse steel being completely absent, to reflect a fully-spalled condition.

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 seismically vulnerable (Buckle, 1991). Multiple span bridges with multiple girder-supported decks represent 82% of the Vermont multiple span bridges, as illustrated in Figure 1.2. This bridge type category represents 55% of the 291,000 multiple span bridges nationwide, also as illustrated in Figure 1.2. 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.



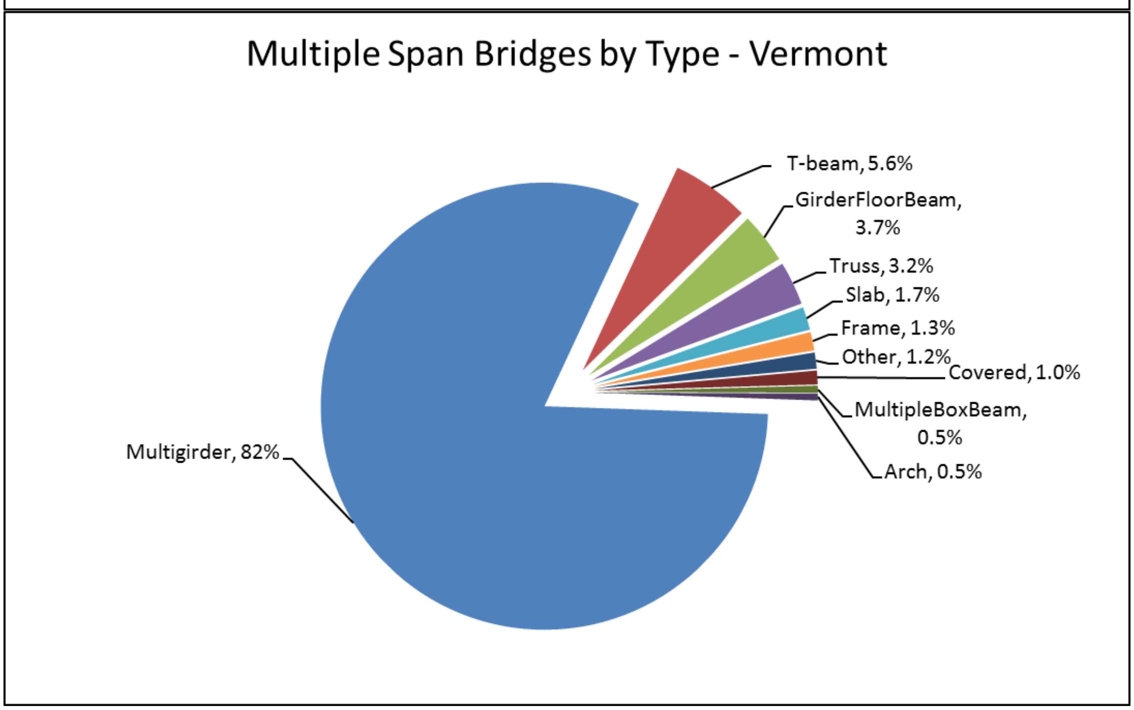


Figure 1.2. – Multiple span bridge types in Vermont and the U.S.

The interstate highway bridge building expansion from the 1950's through the 1960's led to standardization of bridge designs among state transportation agencies, encouraged by sharing of plans and typical details (Catalog of Highway Bridge Plans, 1959). The result of this standardization is that similar 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 the historical record of AASHTO seismic requirements in the Appendix.

Table 1.3 Spectral acceleration values used for low and low-moderate seismic hazard scenarios

Hazard Scenario	PGA (g)	0.2 Second Spectral Acceleration (g)	1-Second Spectral Acceleration (g)	Comments
Low	0.01 - 0.06	0.02 - 0.14	0.01 - 0.04	1, 2
Low-Moderate	0.06 - 0.15	0.14 - 0.25	0.04 - 0.06	1, 2

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 1.3. The bridges have reinforced concrete column supported bents configured in repetitive type configurations of square or round columns, with a 3 ft side width or diameter, respectively. The cross-beams supporting the girders are typically square or rectangular, between 3 and 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 1.3. Bridges evaluated for this study (Photos courtesy of VTrans).

The expected behavior of these two actual bridges constructed between 1964 and 1967 as part of the interstate highway program, was analyzed using commercial structural analysis 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 (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 1.3 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.

1.1.1 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., Eberhard and Marsh, 1997a and b)
    - 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 earth-quakes 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.

The combination of observational, experimental, and theoretical investigations evaluated in published literature support the analyses which led to the recommended screening algorithm for evaluating system-wide seismic vulnerability for Vermont bridges.

1.2 Background of Bridge Seismic Vulnerability

1.2.1 Observational Findings

The bridge seismic vulnerability evaluation benefited from published investigations of seismic damage compiled for several earthquakes through the 1960's and early 1970's 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. Those potential underestimation scenarios are also possible for low to moderate seismicity regions in the U.S., which in general, have seen an increase in the estimated hazard as more recorded earthquake motion data becomes available.

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 of seismic loading requirements 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, Chilean and U.S. earthquakes where damage was cataloged.

1.2.2 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 EERC 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."

1.2.3 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 flow.

2. Analyses for Vermont

2.1 Applicable Vermont Bridge Design Standards

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 seismic load requirement 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.

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). The FHWA 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 a 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.

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.

2.3 Analysis Descriptions

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 2.1 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 2.1 VeRSSA Vulnerability Screening Characteristics							
Item	**Characteristic**	**NBI Item Number**	**Item Name**	**Item Description**	**NBI Item Value**		
					Least Vulnerable	Most Vulnerable	Default Value
V1	Span vulnerability	43A	Kind of Material and/or Design	Is this a continuous span bridge?	Continuous	Simple	N.A.
V2	Bearing type(s)	224	Type of Expansion Bearing Device	Are the bearings readily subject to toppling?	All others	Note 1	N.A.
	Span Skew	34	Skew	Does the bridge skew create more chance of span unseating?	<20 degrees	>20 degrees	N.A.
	Span Type	43B	Type of Design and/or Construction	Does this bridge have girder and floor beam spans?	Not this type	This type	N.A.
	Structural Condition Rating	239	Deficiency Status of Structure	Is this structure cataloged as structurally deficient?	Not SD	SD	N.A.
V3	Fracture Criticality of Structure	801	FCM Detail	Are fracture critical members present?	None present	Present	N.A.
Liquefaction	Foundation Stability	225 A-G	Type of Foundation at (Abutment, Pier)	Are foundations likely directly on rock?	B	E	E
Column Vulnerability	Column Ductility	N.A.	Seismic Retrofit Category per FHWA 2006	Is this Seismic Retrofit Category A or B?	A or B	C or D	N.A.
Abutment	Abutment damage potential	N.A.	Seismic Retrofit Category per FHWA 2006	Is this above or below Seismic Retrofit Category D?	<D	D	N.A.
		34	Skew	Is the span skew greater than 40 degrees?	<40 degrees	>40 degrees	N.A.

2.3.1.1 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 2.4 illustrates types of span vulnerability features.

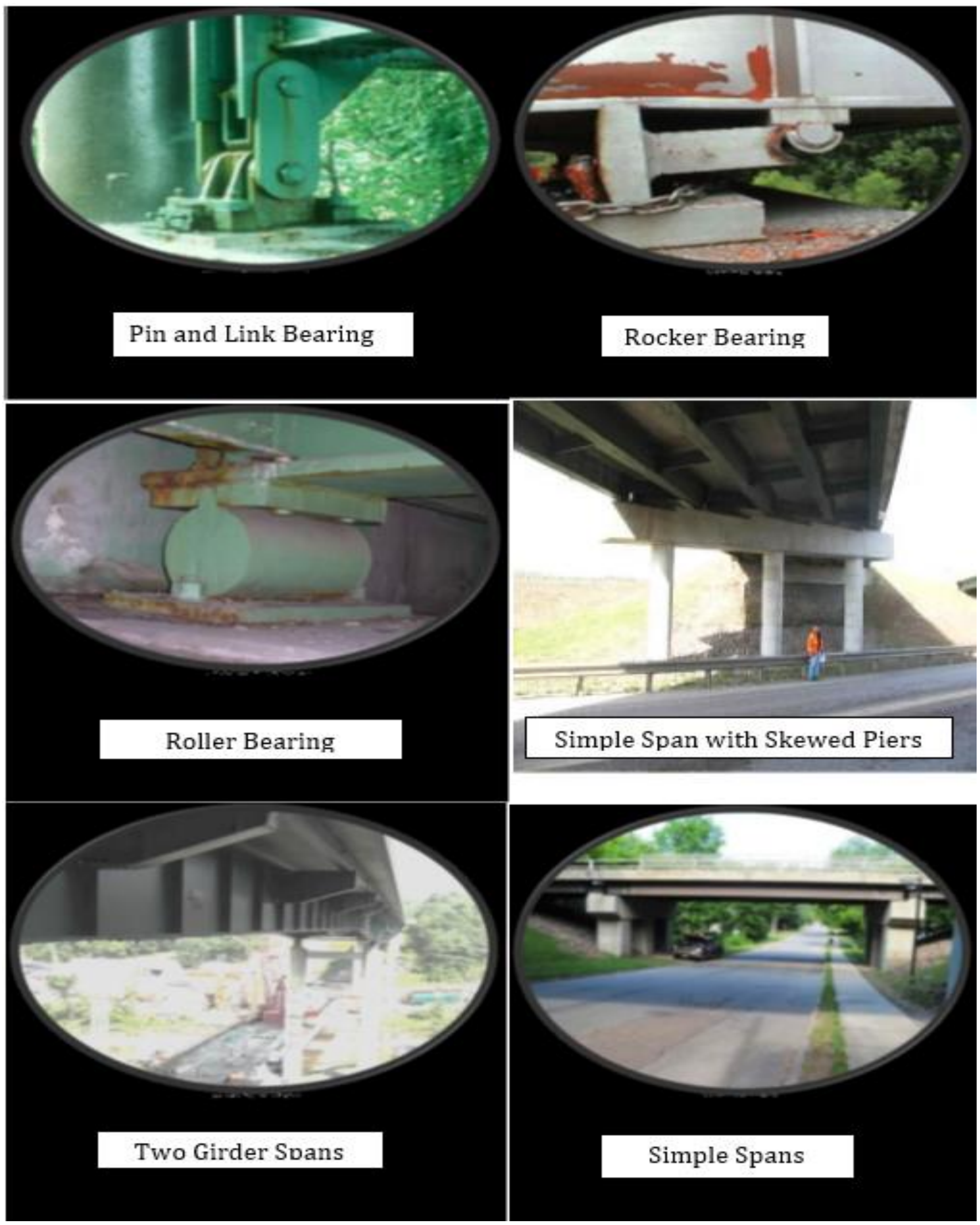


Figure 2.1 – Bridge seismically vulnerable feature examples (FHWA, 2012)

2.3.1.2 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.

2.3.1.3 Abutment Damage Vulnerability

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

2.3.1.4 Liquefaction-Induced Damage Vulnerability

Large foundation settlements and lateral movements can occur where the foundation soils lose most or all their shear strength due to liquefaction occurring because of substantial ground shaking where there is loose and submerged granular soil. 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 is 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).

2.3.1.5 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 is within approximately 10 percent.

2.3.2 Individual Bridge Analysis

2.3.2.1 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 1.3.

2.3.2.2 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.

> <u>Hinge yield rotation</u> – 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.
>
> <u>Hinge ultimate rotation capacity</u> – 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 2.2. 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 2.3 a and b.

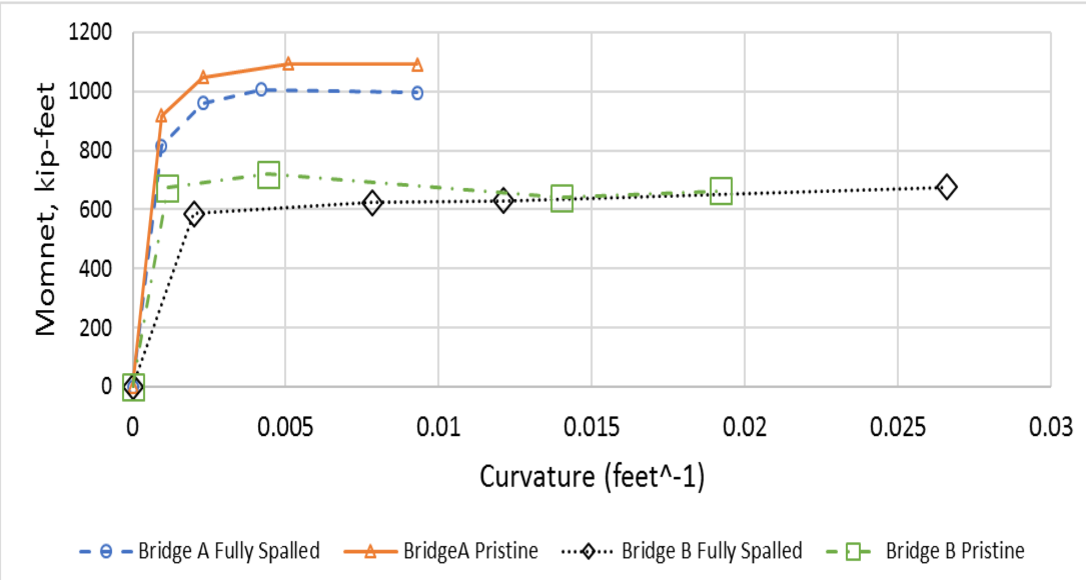
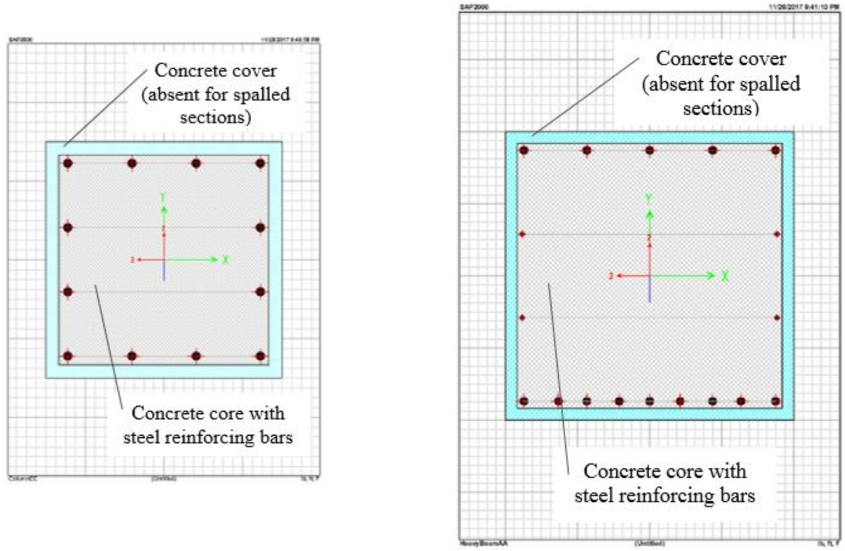


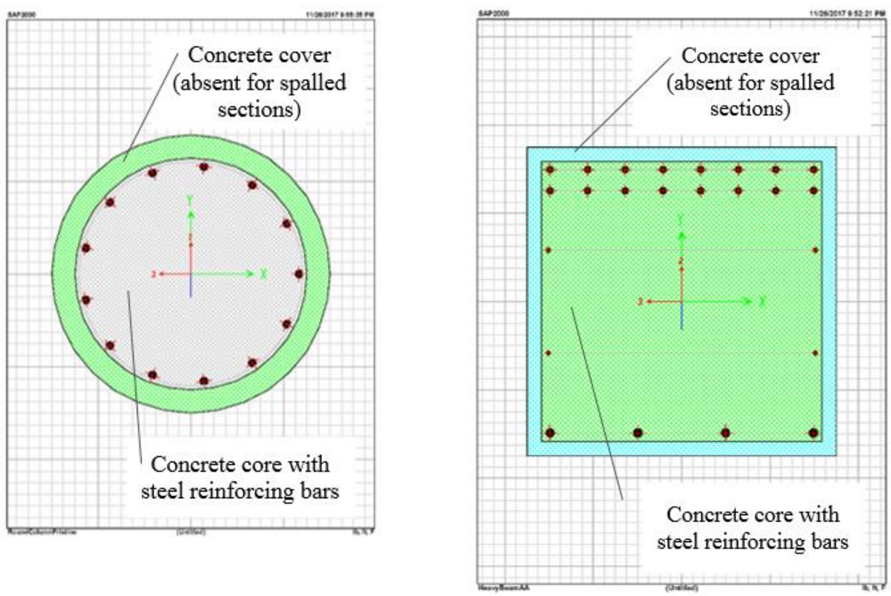
Figure 2.2 Bridge model column hinge moment-curvature relationships

Table 2.2 Summary of Bridge Bent Model Lateral Ductility Characteristics

Pushover values for	**Bridge** Units	**Bridge A** **Fully Spalled**	**Bridge A** **Pristine**	**Bridge B** **Fully Spalled**	**Bridge B** **Pristine**
Yield Point Deflection	feet	0.106	0.077	0.044	0.028
Yield Point Base Shear	kips	145	160	185	224
Ultimate Displacement Capacity	feet	0.48	0.48	0.44	0.55
Maximum Displace-ment Base Shear	kips	173	188	245	228
Displacement Ductility Capacity		4.5	6.2	9.9	19.6
Total Static Pushover Energy Capacity	ft-kips	57.8	65.3	83.1	122.2
Bridge Bent Trans-verse Tn (initial)	seconds	0.62	0.51	0.48	0.35



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



b) Bridge B Column (left) and Beam (right) Sections

Figure 2.3 Column and beam section illustration with and without spalling

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 each 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 2.4. Model input and analysis parameters including damping are shown in Table 2.3.

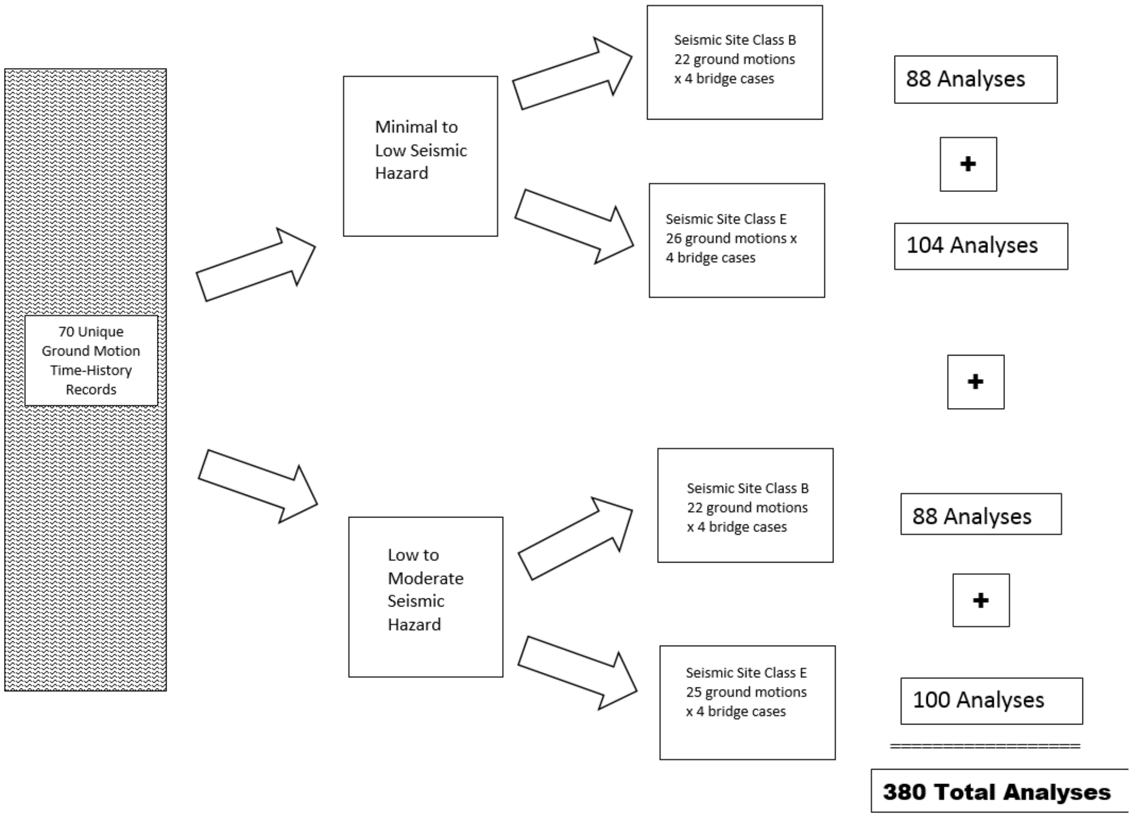


Figure 2.4 Time-history and bridge condition analysis combinations

Table 2.3 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:

$$M\ddot{x}(t) + C\dot{x}(t) + F(x)t = M\ddot{x}_g(t)$$

(eq 2)

where:

M = mass matrix

C = damping matrix

F = nonlinear restoring force function

(t) = relative acceleration vector of degrees of freedom

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

$\dot{x}(t)$ = relative velocity vector of degrees of freedom

x(t) = relative displacement vector of degrees of freedom

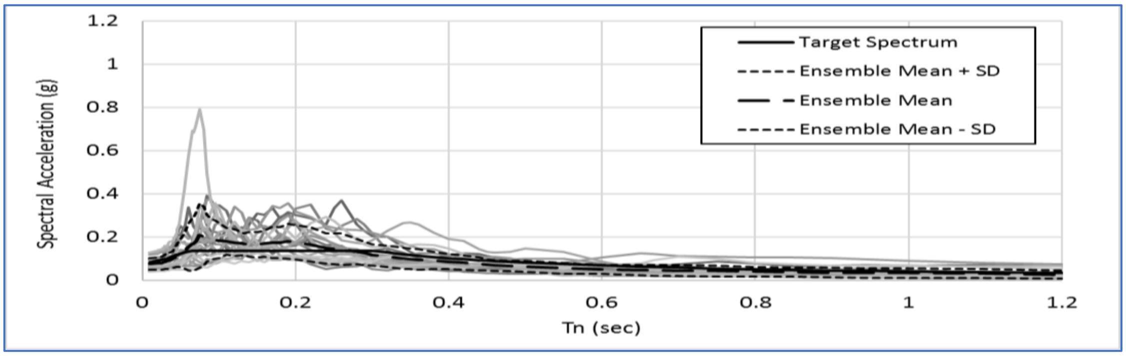
The seismic analysis advanced sequentially in time steps not exceeding that of the earthquake ground motion records, which were typically 0.0024 to 0.01 seconds each, with the records typically lasting from 30 to 100 seconds.

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

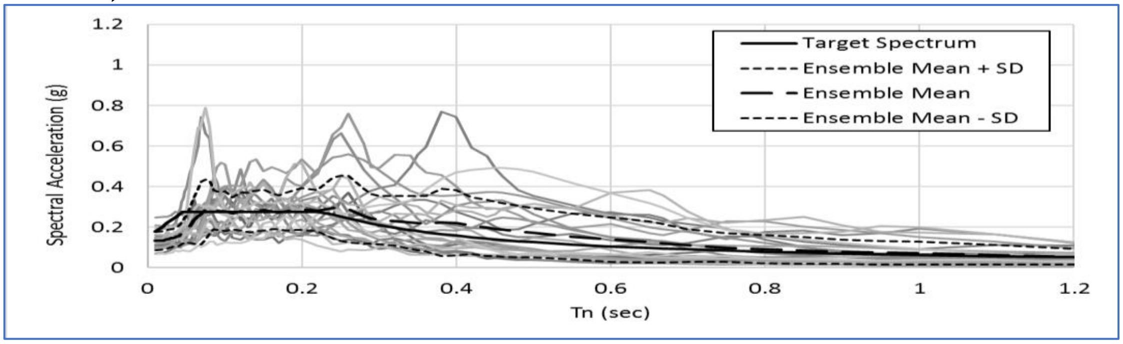
- Structure lateral displacement
- Hinge rotations and corresponding moments in the hinges
- Horizontal shear forces at the column bases and at girder bearing level

2.3.2.3 Description of the Ground Motion Time Histories

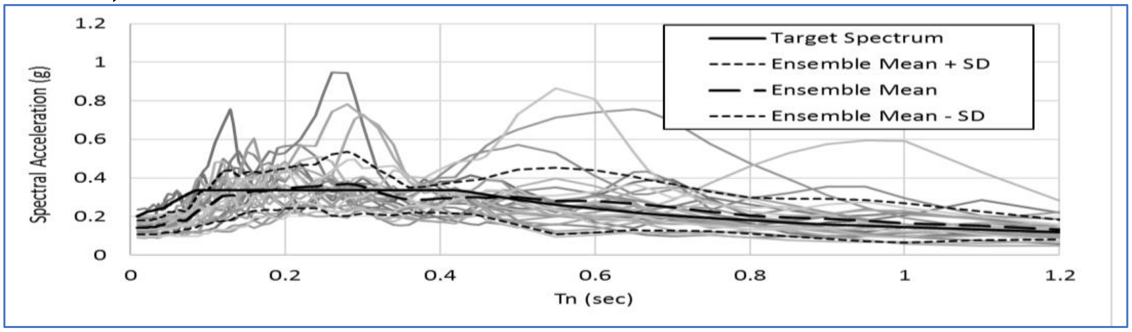
Ground motion time histories were obtained from the Pacific Earthquake Engineering Research (PEER) center NGA West 2 database. The time histories were selected to match the design spectra for two bounding seismic hazard conditions in Vermont, per the AASHTO 2014 LRFD, which correspond to a 7% in 75-year probability (1033-year return period) of exceedance for the extreme northwest, and southeast of Vermont. These target design spectra are shown on Figures 2.5 a-d along with the spectral accelerations for each of the time-history records within the ensembles chosen to match those spectra.



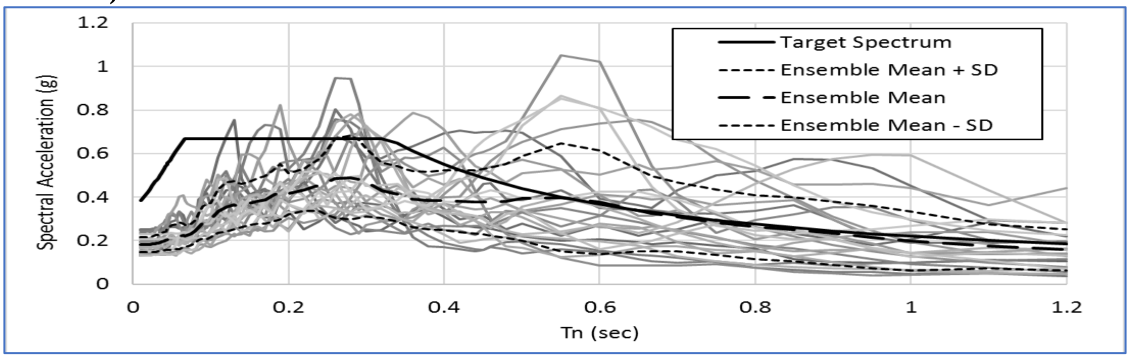
a)Seismic hazard level 1



b)Seismic hazard level 2



c)Seismic hazard level 3



d)Seismic hazard level 4

Figure2.5 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,Vs_{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 2.6 illustrates locations of the ground motions which were available in the PEER NGA East ground motion catalog.

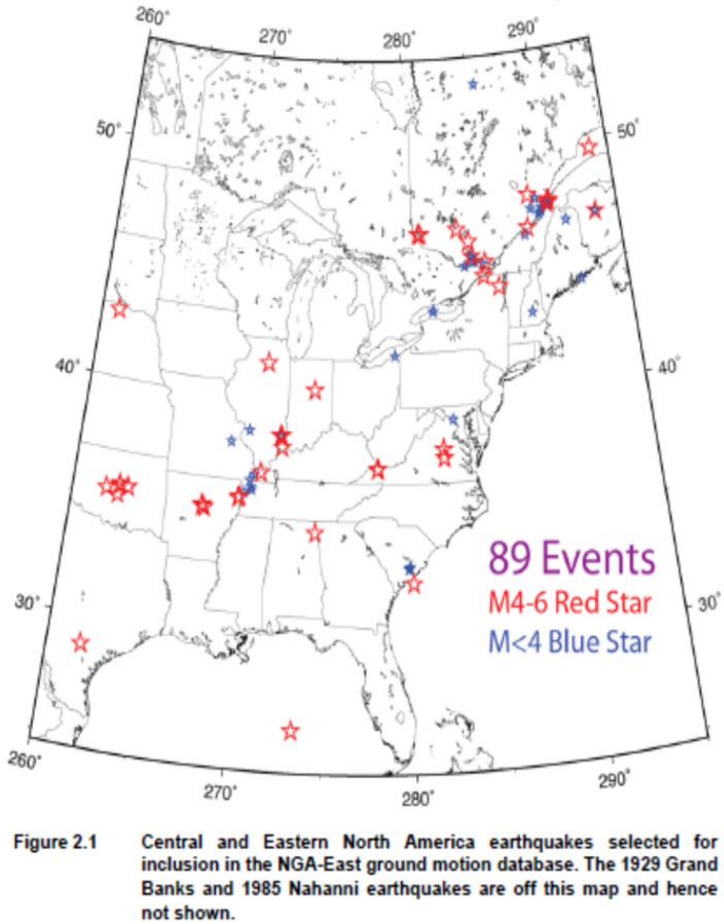


Figure2.6 -PEER NGA East ground motion record locations (PEER, 2018)

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

3. Analysis Results

3.1 Individual Bridge Analyses

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

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 3.1 and in Table 2.2. 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.

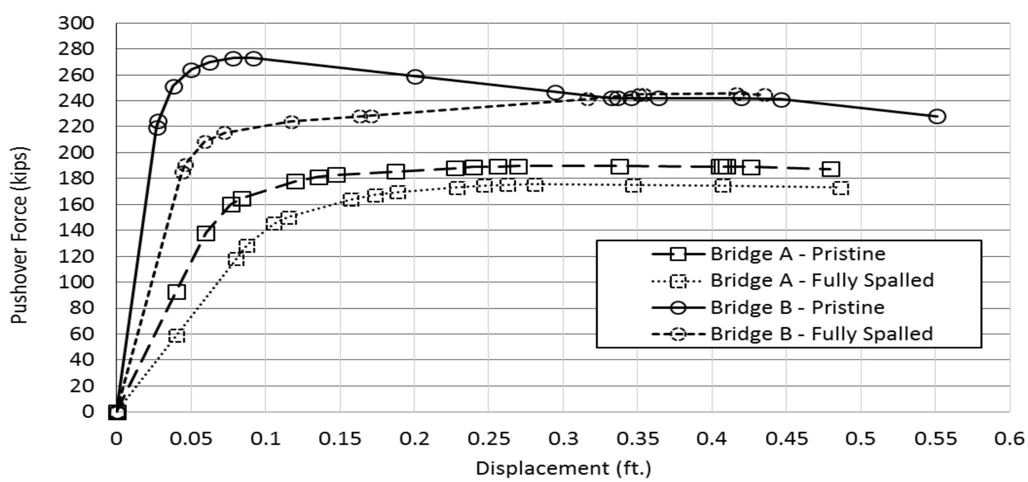


Figure 3.1 Pushover force-displacement for Bridge A pristine and fully spalled and Bridge B pristine and fully spalled

3.1.2 Damage Index

The analysis results in histogram form for the ground motions applied to the two bridges in the previously described combinations of seismic hazard and seismic site class are shown on Figure 3.2. These damage potentials, reflected in the Damage Index (Park and Ang, 1985a and b) distribution, are based on minimal to low and moderate seismic loading. The figure illustrates that most damage index values are under 0.3, corresponding to negligible or minor damage occurring.

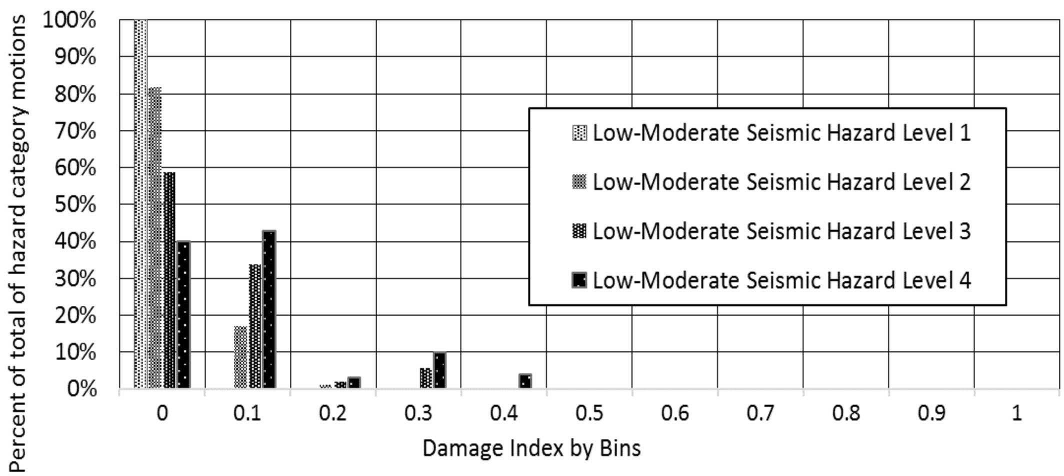


Figure 3.2 Binned Damage Index by seismic hazard level

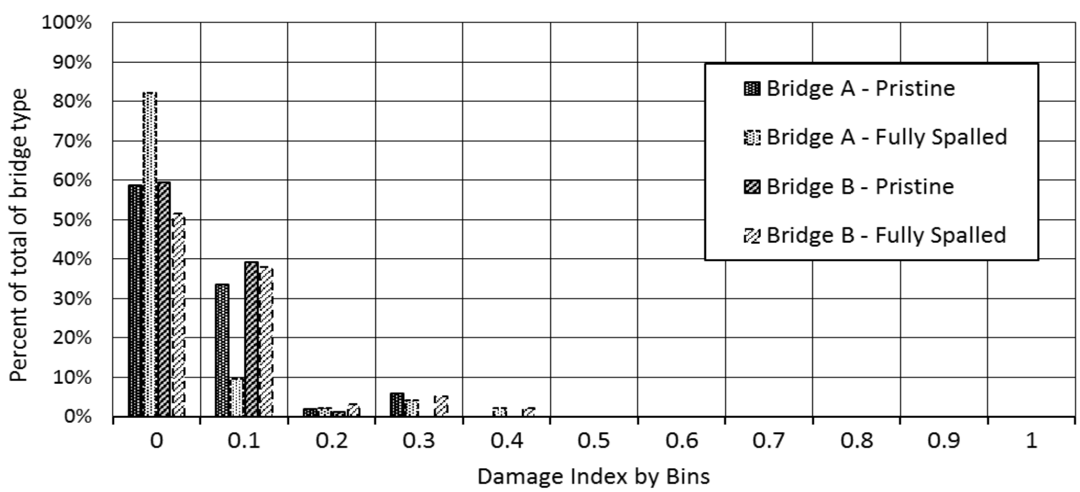


Figure 3.3 Binned Damage Index by bridge type and condition

Figure 3.3 illustrates in histogram form the distribution of damage potential, also as categorized by Damage Index, for the two bridge models, in both pristine and fully-spalled conditions. These results indicate low potential for seismic damage to the concrete bent frames for these types of bridges in this low-to-moderate seismic hazard region.

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 280 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 3.4 illustrates the computed maximum base shear and displacement for each of the bridge models in pristine and spalled conditions.

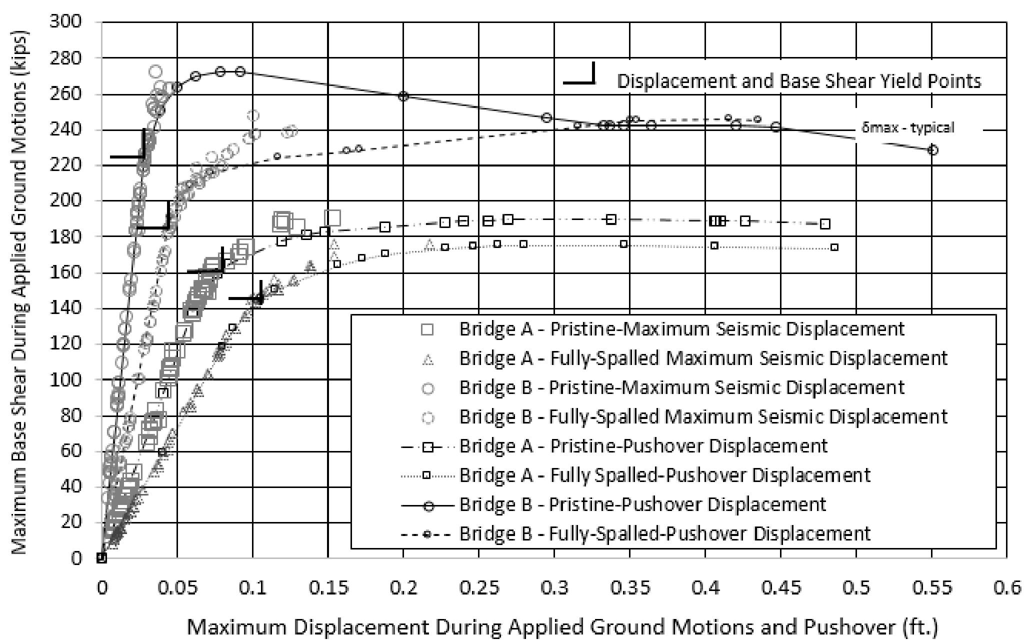


Figure 3.4 Maximum displacement vs. maximum base shear during applied ground motions and pushover for bridge A and bridge B, pristine and fully spalled

3.1.4 Potential for Bent Cap Displacement (Drift)

The range of computed maximum bent cap displacements is also shown on Figure 3.4. Most displacements are less than the pushover yield values with maximum displacements reaching nearly 3 inches at Bridge A and 1.5 inches at Bridge B, although most are under one-half of the maximum values. The fully-spalled versions of each bridge have the largest maximum displacements.

3.2 VeRSSA Screening by Vulnerability Characteristics

Results of screening of multiple span bridges by vulnerability characteristics using the VeRSSA are shown on Figure 3.5. 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.

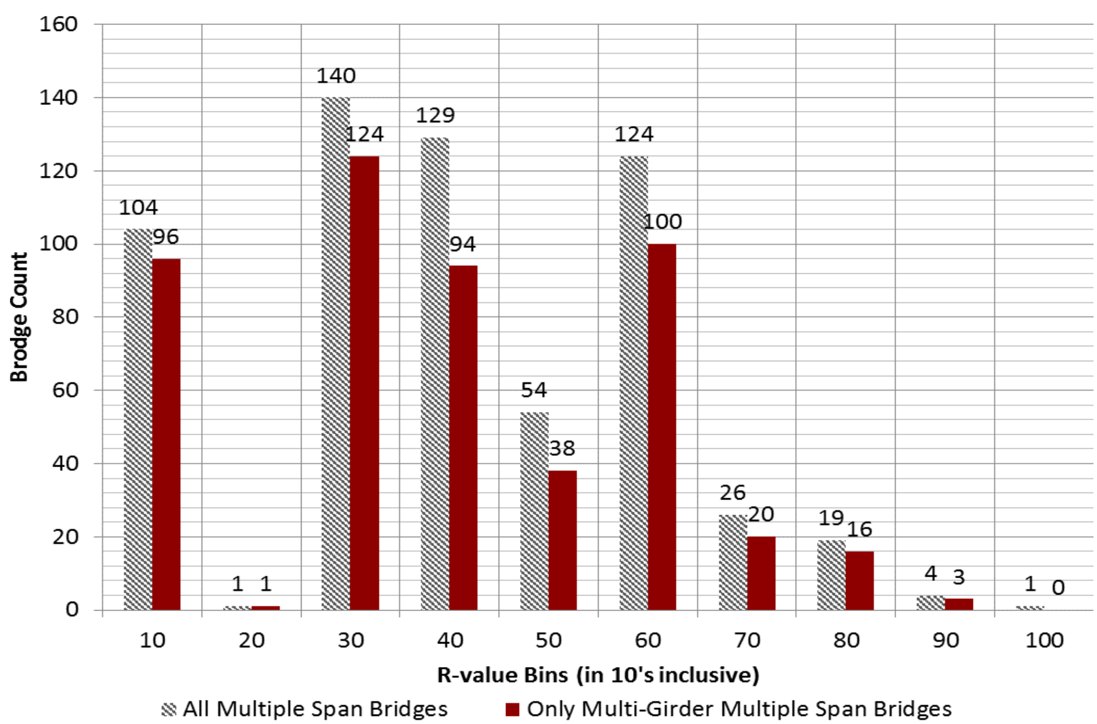


Figure 3.5 - Histogram of vulnerability rating values for multiples span bridges from VeRSSA analysis

Generalizations regarding characteristics suggested by this rating are:

- Most of the continuous bridges are in the lowest binned vulnerability rating categories. This reflects that the screening algorithm 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% 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.

4. Seismic Vulnerability Rating Conclusions and Recommendations

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 or 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 bearings which topple and bearing connections which break.
- Where the superstructure bearing dimensions are insufficient such that main support members fall off their supports when bearings topple or bearing connections break. The drop can be several inches off a bearing pedestal, or the entire column height, depending on how much displacement occurs.
- Where susceptible soils underlie the substructures and approaches such that liquefaction or flow slides cause settlement or lateral displacement, unless these are prevented with proper structural foundations or ground improvement.
- Where seismic hazards and bridge vulnerability are compounded by earthquake related scour, 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 1.2. Bridges are widely distributed across Vermont (see Figure 1.1) such that the seismic hazard variation affects the inventory on essentially a state-wide basis.

4.1.1 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 ductility 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 any 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 the strata 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 undermine abutments by causing loose soils to settle, susceptible soils to liquefy or laterally flow from under abutments. The presence and reliability of subsurface explorations at the approaches is important in the same manner as for the abutment foundations.

Cataloging this additional information should be prioritized within the goals of the bridge inspection and asset management efforts. This will substantially improve the 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.

5. References

AASHTO LRFD Bridge Design Specifications. Seventh Edition, Parts I and II. 2014

AASHTO LRFD Bridge Design Specifications. Eighth Edition, November 2017. Parts I and II. 2017. https://trid.trb.org/View/1488438.

Buckle, Ian G. 1991. "Screening Procedures for the Retrofit of Bridges." In , 156–65. ASCE. http://cedb.asce.org/CEDBsearch/record.jsp?dockey=0073888.

Buckle, Ian G, and Ian M Friedland. 1995 "Improved Screening Procedure for Seismic Retrofitting of Highway Bridges," 12.

Catalog of Highway Bridge Plans,. 1959. Washington. http://hdl.handle.net/2027/mdp.39015021061018.

Choi, E. S., R. DesRoches, and B. Nielson. 2004. "Seismic Fragility of Typical Bridges in Moderate Seismic Zones." *Engineering Structures* 26 (2): 187–99. https://doi.org/10.1016/j.engstruct.2003.09.006.

Chopra, Anil K. 2007. "Elastic Response Spectrum: A Historical Note." *Earthquake Engineering & Structural Dynamics* 36 (1): 3–12. https://doi.org/10.1002/eqe.609.

DesRoches, Reginald, Eunsoo Choi, Roberto T. Leon, Shirley J. Dyke, and Mark Aschheim. 2004. "Seismic Response of Multiple Span Steel Bridges in Central and Southeastern United States. I: As Built." *Journal of Bridge Engineering* 9 (5): 464–72. https://doi.org/10.1061/(ASCE)1084-0702(2004)9:5(464).

Dicleli, Murat, and Michel Bruneau. 1996. "Quantitative Approach to Rapid Seismic Evaluation of Slab-on-Girder Steel Highway Bridges." *Journal of Structural Engineering* 122 (10): 1160–68. https://doi.org/10.1061/(ASCE)0733-9445(1996)122:10(1160).

Earthquake Engineering Research Center, 1972 Report No. 72-11 Literature Search-Seismic Effects on Highway Bridges

Eberhard, Marc O., and M. Lee Marsh. 1997a. "Lateral-Load Response of a Reinforced Concrete Bridge." *Journal of Structural Engineering* 123 (4): 451–60. https://doi.org/10.1061/(ASCE)0733-9445(1997)123:4(451).

Eberhard, Marc O., and M. Lee Marsh. 1997b. "Lateral-Load Response of Two Reinforced Concrete Bents." *Journal of Structural Engineering* 123 (4): 461–68. https://doi.org/10.1061/(ASCE)0733-9445(1997)123:4(461).

Filiatrault, A., S. Tremblay, and R. Tinawi. 1994. "A Rapid Seismic Screening-Procedure for Existing Bridges in Canada." *Canadian Journal of Civil Engineering* 21 (4): 626–42. https://doi.org/10.1139/l94-064.

FHWA 1983, Seismic Retrofitting Guidelines for Highway Bridges, FHWA/RD-83/007, Final Report, December 1983

FHWA 1995, Seismic Retrofitting Manual for Highway Bridges, FHWA-RD-94-052, Federal Highway Administration, Turner-Fairbank Highway Research Center, McLean, VA

FHWA 2006, Seismic Retrofitting Manual for Highway Bridges, FHWA-HRT-06-032, Federal Highway Administration, Turner-Fairbank Highway Research Center, McLean, VA

FHWA, 2012, Bridge Inspectors Reference Manual, FHWA-NHI-12-049 Federal Highway Administration, Turner-Fairbank Highway Research Center, McLean, VA

Frankel, Arthur D., C. Mueller, T. Barnhard, D. Perkins, E.V. Leyendecker, N. Dickman, S. Hanson, and M. Hopper. 1997. "Seismic-Hazard Maps for the Conterminous United States." Report 97–131. Open-File Report. USGS Publications Warehouse

Ghosh, Jayadipta, and Jamie E. Padgett. 2010. "Aging Considerations in the Development of Time-Dependent Seismic Fragility Curves." *Journal of Structural Engineering* 136 (12): 1497–1511. https://doi.org/10.1061/(ASCE)ST.1943-541X.0000260.

Hoult, Ryan D, Elisa Lumantarna, and Helen M Goldsworthy. 2013. "Ground Motion Modelling and Response Spectra for Australian Earthquakes," 10.

Housner, G.A., 1952, Spectrum of Intensities of Strong Motion Earthquakes, Proceedings of the Symposium on Earthquake and Blast Effects on Structures, Los Angeles, California, 1952

Huang, Qindan, Paolo Gardoni, David Trejo, and Alex Pagnotta. 2014. "Probabilistic Model for Steel–Concrete Bond Behavior in Bridge Columns Affected by Alkali Silica Reactions." *Engineering Structures* 71 (July): 1–11. https://doi.org/10.1016/j.engstruct.2014.03.041.

Karim, Kazi R., and Fumio Yamazaki. 2003. "A Simplified Method of Constructing Fragility Curves for Highway Bridges." *Earthquake Engineering & Structural Dynamics* 32 (10): 1603–26. https://doi.org/10.1002/eqe.291.

Mander, Jb, Mjn Priestley, and R. Park. 1988a. "Observed Stress-Strain Behavior of Confined Concrete." *Journal of Structural Engineering-Asce* 114 (8): 1827–49. https://doi.org/10.1061/(ASCE)0733-9445(1988)114:8(1827).

Mander, Jb, Mjn Priestley, and R. Park. 1988b. "Theoretical Stress-Strain Model for Confined Concrete." *Journal of Structural Engineering-Asce* 114 (8): 1804–26. https://doi.org/10.1061/(ASCE)0733-9445(1988)114:8(1804).

Muntasir Billah, A.H.M., and M. Shahria Alam. 2015. "Seismic Fragility Assessment of Highway Bridges: A State-of-the-Art Review." *Structure and Infrastructure Engineering* 11 (6): 804–32. https://doi.org/10.1080/15732479.2014.912243.

"National Bridge Inventory - Bridge Inspection - Safety - Bridges & Structures - Federal Highway Administration." 2018 Accessed August 12, 2018. https://www.fhwa.dot.gov/bridge/nbi.cfm.

Nielson, Bryant G., and Reginald DesRoches. 2007a. "Analytical Seismic Fragility Curves for Typical Bridges in the Central and Southeastern United States." *Earthquake Spectra* 23 (3): 615–33. https://doi.org/10.1193/1.2756815.

Nielson, Bryant G., and Reginald DesRoches. 2007b. "Analytical Seismic Fragility Curves for Typical Bridges in the Central and Southeastern United States." *Earthquake Spectra* 23 (3): 615–33. https://doi.org/10.1193/1.2756815.

Nielson, Bryant G., and Reginald DesRoches. 2007c. "Seismic Performance Assessment of Simply Supported and Continuous Multispan Concrete Girder Highway Bridges." *Journal of Bridge Engineering* 12 (5): 611–20. https://doi.org/10.1061/(ASCE)1084-0702(2007)12:5(611).

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

Pan, Y., A. K. Agrawal, M. Ghosn, and S. Alampalli. 2010a. “Seismic Fragility of Multispan Simply Supported Steel Highway Bridges in New York State. I: Bridge Modeling, Parametric Analysis, and Retrofit Design.” *Journal of Bridge Engineering* 15 (5): 448–61. https://doi.org/10.1061/(ASCE)BE.1943-5592.0000085.

Pan, Y., A. K. Agrawal, M. Ghosn, and S. Alampalli. 2010b. “Seismic Fragility of Multispan Simply Supported Steel Highway Bridges in New York State. II: Fragility Analysis, Fragility Curves, and Fragility Surfaces.” *Journal of Bridge Engineering* 15 (5): 462–72. https://doi.org/10.1061/(ASCE)BE.1943-5592.0000055.

Park, Yj, and Ahs Ang. 1985a. “Mechanistic Seismic Damage Model for Reinforced-Concrete.” *Journal of Structural Engineering-Asce* 111 (4): 722–39.

Park, Yj, and Ahs Ang. 1985b. “Mechanistic Seismic Damage Model for Reinforced-Concrete.” *Journal of Structural Engineering-Asce* 111 (4): 722–39.

“PEER Ground Motion Database - PEER Center.” n.d. Accessed July 2, 2018. https://ngawest2.berkeley.edu/.

SAP2000, Version 17.3,2015, Computers and Structures, Inc.

“Seismic Retrofitting Manual for Highway Bridges. | National Technical Reports Library - NTIS.” n.d. Accessed August 12, 2018. https://ntrl.ntis.gov/NTRL/dashboard/searchResults/titleDetail/PB95212676.xhtml.

“Seismic Retrofitting Manual for Highway Structures: Part 1 – Bridges , January 2006 - FHWA-HRT-06-032.” 2006. January 2006. https://www.fhwa.dot.gov/publications/research/infrastructure/bridge/06032/.

“Seismic Retrofitting Manual for Highway Structures: Part 1 - Bridges: MCEER-06-SP10.” n.d. Accessed July 31, 2018. http://mceer.buffalo.edu/publications/catalog/reports/Seismic-Retrofitting-Manual-for-Highway-Structures-Part-1-Bridges-MCEER-06-SP10.html.

Tavares, Danusa H., Jamie E. Padgett, and Patrick Paultre. 2012. “Fragility Curves of Typical As-Built Highway Bridges in Eastern Canada.” *Engineering Structures* 40 (July): 107–18. https://doi.org/10.1016/j.engstruct.2012.02.019.

Toro, Gabriel R., Norman A. Abrahamson, and John F. Schneider. 1997. “Model of Strong Ground Motions from Earthquakes in Central and Eastern North America: Best Estimates and Uncertainties.” *Seismological Research Letters* 68 (1): 41–57.

“USGS Open-File Report 02-420: Documentation for the 2002 Update of the National Seismic Hazard Maps.” n.d. Accessed July 20, 2018. https://pubs.usgs.gov/of/2002/ofr-02-420/.

Vamvatsikos, D., and C. A. Cornell. 2002. “Incremental Dynamic Analysis.” *Earthquake Engineering & Structural Dynamics* 31 (3): 491–514. https://doi.org/10.1002/eqe.141.

VTrans Structures Design Manual, 5th Edition, SD5-2012.9.13, Structures Division, 2010

Wild, J.R., Eliassen, T.D.,McMahon, J.K., 2008, Investigation of Alkalai-Silica Reactivity in Portland Cement Concrete Phase II-Select Vermont Structures, Vermont Agency of Transportation, December 5, 2008

Zhong, Jinquan, Paolo Gardoni, and David Rosowsky. 2012. "Seismic Fragility Estimates for Corroding Reinforced Concrete Bridges." *Structure and Infrastructure Engineering* 8 (1): 55–69. https://doi.org/10.1080/15732470903241881.

APPENDICES

Historical Record of AASHTO Seismic Loading Requirements through 1983			
Year	**Reference**	**Section**	**Criteria**
1931	The American Association of State Highway Officials. (1931). *Standard Specifications for Highway Bridges and Incidental Structures,* 1st Ed., The Association of General Offices, Washington DC.	N/A	none (no mention of earthquakes)
1953	The American Association of State Highway Officials. (1953). *Standard Specifications for Highway Bridges,* 6th Ed., The Association of General Offices, Washington DC.	3.2.1.(design loads), 3.4.1.(unit stresses)	In both sections, earthquakes are mentioned but no quantifiable details are provided.
1961	The American Association of State Highway Officials. (1961). *Standard Specifications for Highway Bridges,* 8th Ed., The Association of General Offices, Washington DC.	1.2.20.	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)
1973	The American Association of State Highway Officials. (1973). *Standard Specifications for Highway Bridges,* 11th Ed., The Association of General Offices, Washington DC.	1.2.20.	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)
1977	The American Association of State Highway Officials. (1977). *Standard Specifications for Highway Bridges*, 12th Ed., The Association of General Offices, Washington DC.	1.2.20	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 = max 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
1981	Federal Highway Administration. (1981). Seismic Design Guidelines for Highway Bridges. Final Report. Federal Highway Administration, Washington DC.	4	Dependent on numerous classifications and factors.

Historical Record of AASHTO Seismic Loading Requirements through 1983			
Year	**Reference**	**Section**	**Criteria**
1983	The American Association of State Highway Officials. (1983). *Standard Specifications for Highway Bridges*, 13th Ed., The Association of General Offices, Washington DC.	3.21	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 = max 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

Ground Motion Summary Information (PEER, 2018)									
RSN	**Spectral Ordinate**	**Earthquake Name**	**Year**	**Station Name**	**Moment Magnitude**	**Mechanism**	**Rjb (km)**	**Rrup (km)**	**Vs30 (m/sec)**
98	H2	"Hollister-03"	1974	"Gilroy Array #1"	5.14	strike slip	9.99	10.46	1428.14
23	H2	"San Francisco"	1957	"Golden Gate Park"	5.28	Reverse	9.74	11.02	874.72
4312	H1	"Umbria-03	1984	"Gubbio"	5.6	Normal	14.67	15.72	922
4312	H2	"Umbria-03_ Italy"	1984	"Gubbio"	5.6	Normal	14.67	15.72	922
1649	H1	"Sierra Madre"	1991	"Vasquez Rocks Park"	5.61	Reverse	37.63	39.81	996.43
1649	H2	"Sierra Madre"	1991	"Vasquez Rocks Park"	5.61	Reverse	37.63	39.81	996.43
146	H1	"Coyote Lake"	1979	"Gilroy Array #1"	5.74	strike slip	10.21	10.67	1428.14
146	H2	"Coyote Lake"	1979	"Gilroy Array #1"	5.74	strike slip	10.21	10.67	1428.14
608	H1	"Whittier Narrows-01"	1987	"Carson - Water St"	5.99	Reverse Oblique	26.3	30.03	160.58
608	H2	"Whittier Narrows-01"	1987	"Carson - Water St"	5.99	Reverse Oblique	26.3	30.03	160.58
643	H1	"Whittier Narrows-01"	1987	"LA - Wonderland Ave"	5.99	Reverse Oblique	23.4	27.64	1222.52
643	H2	"Whittier Narrows-01"	1987	"LA - Wonderland Ave"	5.99	Reverse Oblique	23.4	27.64	1222.52
680	H1	"Whittier Narrows-01"	1987	"Pasadena - CIT Kresge Lab"	5.99	Reverse Oblique	6.78	18.12	969.07
680	H2	"Whittier Narrows-01"	1987	"Pasadena - CIT Kresge Lab"	5.99	Reverse Oblique	6.78	18.12	969.07
703	H1	"Whittier Narrows-01"	1987	"Vasquez Rocks Park"	5.99	Reverse Oblique	47.25	50.39	996.43
703	H2	"Whittier Narrows-01"	1987	"Vasquez Rocks Park"	5.99	Reverse Oblique	47.25	50.39	996.43

4083	H1	"Parkfield-02_ CA"	2004	"PARK-FIELD - TURKEY FLAT #1 (0M)"	6	strike slip	4.66	5.29	906.96
4083	H2	"Parkfield-02_ CA"	2004	"PARK-FIELD - TURKEY FLAT #1 (0M)"	6	strike slip	4.66	5.29	906.96
455	H1	"Morgan Hill"	1984	"Gilroy Ar-ray #1"	6.19	strike slip	14.9	14.91	1428.14
455	H2	"Morgan Hill"	1984	"Gilroy Ar-ray #1"	6.19	strike slip	14.9	14.91	1428.14
2715	H1	"Chi-Chi Taiwan 04"	1999	"CHY047"	6.2	strike slip	38.59	38.62	169.52
2715	H2	"Chi-Chi Taiwan-04"	1999	"CHY047"	6.2	strike slip	38.59	38.62	169.52
2753	H1	"Chi-Chi	1999	"CHY102"	6.2	strike slip	39.3	39.32	804.36
2753	H2	"Chi-Chi Taiwan-04"	1999	"CHY102"	6.2	strike slip	39.3	39.32	804.36
2955	H1	"Chi-Chi Taiwan 05"	1999	"CHY047"	6.2	Reverse	66.53	71.26	169.52
2955	H2	"Chi-Chi Taiwan-06"	1999	"CHY047"	6.2	Reverse	66.53	71.26	169.52
2989	H1	"Chi-Chi	1999	"CHY102"	6.2	Reverse	69.76	74.16	804.36
2989	H2	"Chi-Chi Taiwan-05"	1999	"CHY102"	6.2	Reverse	69.76	74.16	804.36
3251	H1	"Chi-Chi	1999	"TTN042"	6.2	Reverse	84.68	85.17	845.34
718	H1	"Supersti-tion Hills-01"	1987	"Imperial Valley Wild-life Lique-faction Ar-ray"	6.22	strike slip	17.59	17.59	179
718	H2	"Supersti-tion Hills-01"	1987	"Imperial Valley Wild-life Lique-faction Ar-ray"	6.22	strike slip	17.59	17.59	179
3282	H1	"Chi-Chi Taiwan-06"	1999	"CHY047"	6.3	Reverse	53.54	54.47	169.52
3282	H2	"Chi-Chi Taiwan-06"	1999	"CHY047"	6.3	Reverse	53.54	54.47	169.52
3302	H1	"Chi-Chi Taiwan 06"	1999	"CHY076"	6.3	Reverse	69.66	70.37	169.84
3302	H2	"Chi-Chi Taiwan-06"	1999	"CHY076"	6.3	Reverse	69.66	70.37	169.84
326	H1	"Coalinga-01"	1983	"Parkfield - Cholame 2WA"	6.36	Reverse	43.83	44.72	173.02
326	H2	"Coalinga-01"	1983	"Parkfield - Cholame 2WA"	6.36	Reverse	43.83	44.72	173.02
334	H2	"Coalinga-01"	1983	"Parkfield - Fault Zone 1"	6.36	Reverse	41.04	41.99	178.27
8167	H2	"San Sim-eon CA"	2003	"Diablo Canyon Power Plant"	6.52	Reverse	37.92	37.97	1100
729	H1	"Supersti-tion Hills-02"	1987	"Imperial Valley Wild-	6.54	strike slip	23.85	23.85	179

				life Liquefaction Array"					
729	H2	"Superstition Hills-02"	1987	"Imperial Valley Wildlife Liquefaction Array"	6.54	strike slip	23.85	23.85	179
80	H1	"San Fernando"	1971	"Pasadena - Old Seismo Lab"	6.61	Reverse	21.5	21.5	969.07
80	H2	"San Fernando"	1971	"Pasadena - Old Seismo Lab"	6.61	Reverse	21.5	21.5	969.07
3925	H1	"Tottori	2000	"OKYH07"	6.61	strike slip	15.23	15.23	940.2
3925	H2	"Tottori_ Japan"	2000	"OKYH07"	6.61	strike slip	15.23	15.23	940.2
3934	H1	"Tottori Japan"	2000	"SMN002"	6.61	strike slip	16.6	16.61	138.76
3934	H2	"Tottori Japan"	2000	"SMN002"	6.61	strike slip	16.6	16.61	138.76
3937	H1	"Tottori Japan"	2000	"SMN005"	6.61	strike slip	45.73	45.73	182.3
3937	H2	"Tottori Japan"	2000	"SMN005"	6.61	strike slip	45.73	45.73	182.3
3954	H1	"Tottori_ Japan"	2000	"SMNH10"	6.61	strike slip	15.58	15.59	967.27
3962	H1	"Tottori Japan"	2000	"TTR005"	6.61	strike slip	45.98	45.98	169.16
6212	H2	"Tottori Japan"	2000	"HRSH08"	6.61	strike slip	143.69	143.69	781.15
4203	H2	"Niigata Japan"	2004	"NIG013"	6.63	Reverse	38	40.59	174.55
4215	H1	"Niigata Japan"	2004	"NIG025"	6.63	Reverse	46.66	48.79	134.5
4215	H2	"Niigata Japan"	2004	"NIG025"	6.63	Reverse	46.66	48.79	134.5
962	H2	"Northridge-01"	1994	"Carson - Water St"	6.69	Reverse	45.44	49.81	160.58
1011	H1	"Northridge-01"	1994	"LA - Wonderland Ave"	6.69	Reverse	15.11	20.29	1222.52
1011	H2	"Northridge-01"	1994	"LA - Wonderland Ave"	6.69	Reverse	15.11	20.29	1222.52
1091	H1	"Northridge-01"	1994	"Vasquez Rocks Park"	6.69	Reverse	23.1	23.64	996.43
1091	H2	"Northridge-01"	1994	"Vasquez Rocks Park"	6.69	Reverse	23.1	23.64	996.43
5259	H1	"Chuetsu-oki Japan"	2007	"NIG013"	6.8	Reverse	27.92	29.8	174.55
5259	H2	"Chuetsu-oki Japan"	2007	"NIG013"	6.8	Reverse	27.92	29.8	174.55
5260	H2	"Chuetsu-oki Japan"	2007	"NIG014"	6.8	Reverse	21.37	27.09	128.12
5271	H1	"Chuetsu-oki Japan"	2007	"NIG025"	6.8	Reverse	28.3	28.59	134.5
5271	H2	"Chuetsu-oki Japan"	2007	"NIG025"	6.8	Reverse	28.3	28.59	134.5

5989	H1	"El Mayor-Cucapah Mexico"	2010	"El Centro Array #3"	7.2	strike slip	40.96	41.29	162.94
5989	H2	"El Mayor-Cucapah Mexico"	2010	"El Centro Array #3"	7.2	strike slip	40.96	41.29	162.94
1147	H1	"Kocaeli Turkey"	1999	"Ambarli"	7.51	strike slip	68.09	69.62	175
1209	H1	"Chi-Chi Taiwan"	1999	"CHY047"	7.62	Reverse Oblique	24.13	24.13	169.52
1209	H2	"Chi-Chi Taiwan"	1999	"CHY047"	7.62	Reverse Oblique	24.13	24.13	169.52